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Design, Construction, and Performance of a Deep Braced Excavation

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ABSTRACT

The engineering approach to design and specification development for a deep excavation is presented along with construction instrumentation data that illustrates the concepts, criteria, and performance of the excavation shoring system. The project included an excavation of up to about 20 m depth, over 650 m long, and 20 m wide made through generally competent glacial overburden with 46 structures located immediately adjacent to the excavation. Excavation support was achieved using a braced soldier-pile and lagging wall system. A detailed instrumentation program was undertaken by the owner to monitor contractor compliance with ground and structure movement criteria. Semi-empirical and theoretical concepts related to earth pressure diagrams and soldier-pile design "reduction factors" are explored in detail, with particular emphasis on contract provisions for specifying design of excavation support. The deformation performance, structural design, and construction pre-loading are shown to be directly linked an alternative approach is presented for future design and specification of excavation support.

INTRODUCTION

Construction of a new subway structure required an excavation in an urban area close to many buildings. Empirical data suggested that a suitably designed soldier-pile and lagging system should be able to control movements within acceptable limits given the reasonably competent soils at the site. To limit damage to the adjacent structures, an iterative process was used during the design stage of the project so that specifications could be developed that would assist in achieving damage control (e.g. Boone et al. 1998 and 1999a). Typical local practice for large public infrastructure contracts is to prepare a performance specification along with a number of minimum design and construction criteria, principally consisting of minimum design earth pressures and maximum permissible displacements. Such criteria are included to give the owner a reasonable degree of assurance that the performance objectives can be met without limiting the contractor's ingenuity or cost-competitiveness within certain bounds. Detailed design of the shoring is then left to the contractor's engineer. Public construction contracts in Toronto are typically awarded on a low-bid basis. However, some shoring design practices that would produce the least costly final design conflict with the goal of limiting movement. Some research has indicated that while apparent or conventionally derived active earth pressures are suitable for designing wall supports (struts, tie-backs, deadmen, etc.), reductions in the cost of the wall components (soldier-piles, sheet-piles, etc.) can be achieved if a "reduction factor" is applied to the earth loads or calculated bending moments. Qualitative indications have been provided in the literature regarding the good practice of strut pre-loading and the associated benefits in limiting wall movement; however, little quantitative information is available (e.g. Peck et al. 1973, O'Rourke 1981, Boone et al. 1999b). The use of load reduction factors and the need for pre-loading of supports has remained subject to considerable judgement and, in some cases, confusion. The design process, final shoring design, and construction performance of this project quantitatively illustrate these issues and provide insight for future projects.

PROJECT DESCRIPTION

A deep braced excavation, over 650 m long, from 9 m to 20 m deep, and over 20 m wide in places, was made for construction of a new section of subway structure for the Toronto Transit Commission (TTC). The new structure included a long (about 230 m) single track section, that opened into a 2 track section, that opened once again into a 3 track wide structure, thus allowing for storing a train and switching train directions. The central track in the 3-track section dropped in elevation relative to the other two tracks so that it could pass beneath the abutting subway station and connect to a subway built at a lower elevation. The central excavation made for the lower track

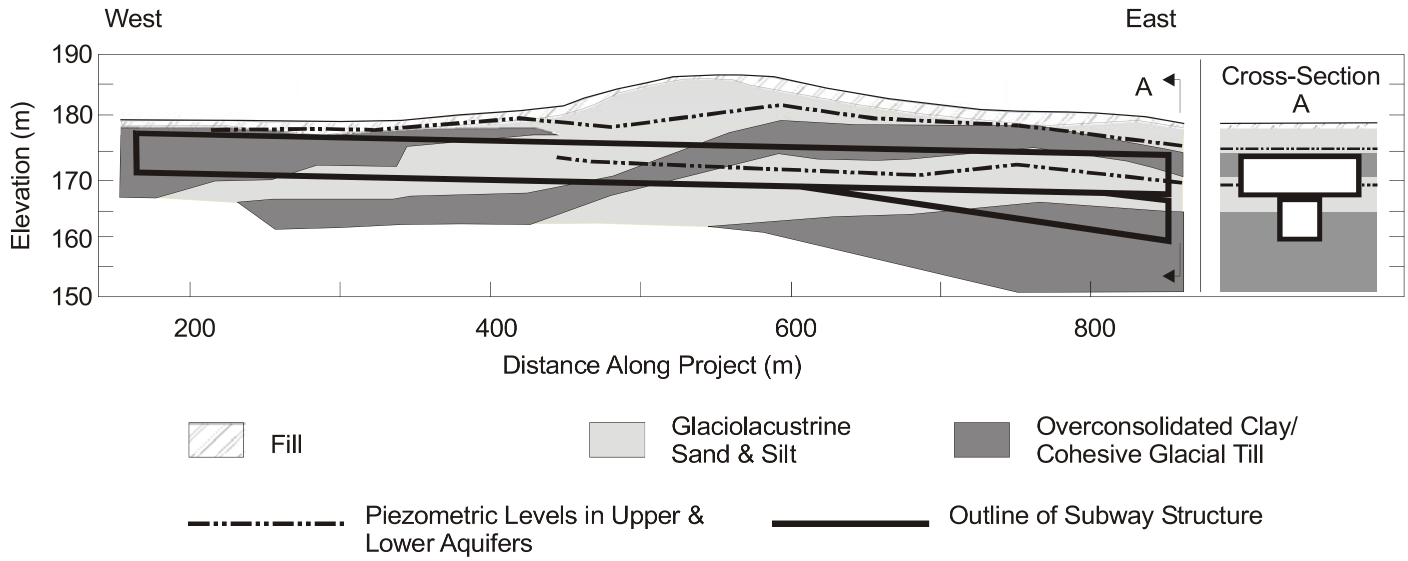

Figure 1. Subsurface conditions along excavation.

varied up to about 7.5 m deep at the east end where it was in the center of the 20.4 m wide excavation. The subway excavation was made through glacial till and highly over-consolidated glaciolacustrine sand, silt, and clay deposits (see Fig. 1). Groundwater levels, observed in two distinct aquifers, ranged from near the ground surface to about 4 m above the base of the main excavation. Geotechnical properties of the various strata are provided in Table 1.

Over 50 buildings were in the vicinity of the project and 46 of these were within the most critical "zone of influence" of the excavation; i.e. where the front of the structure was within a distance equal to or less than the depth of the excavation. Some buildings were less than 2 m from the excavation face. Most of the structures were between 1 and 3 stories high with a shallow basement and were constructed of masonry load-bearing exterior walls and wood framing within.

Table 1. Generalized geotechnical properties of deposits.

Parameter	West End	Middle & East End
Total Unit Weight, γ (kN/m^3)	Fill – 18 Sand/Silt – 22 Clay/Till – 21	Fill - 18 Sand/Silt - 22 Clay/Till - 22
Effective Friction Angle – Granular Soils, ϕ'	Fill – 25^o Sand/Silt – 38^o	Fill – 25^o Sand/Silt – 36^o
Undrained Shear Strength - Cohesive Soils, S_u (kPa)	150 in upper part of deposit	400
Average SPT "N" Value (blows/0.3 m)	20 to 60	40 to 70

DEVELOPMENT OF PROJECT SPECIFICATIONS

In the 1960's, design of shoring systems for Toronto's subway construction was left primarily to the contractors. Reviews of construction records from this time indicated that this "hands-off" approach was sometimes less than successful. In some cases damage to adjacent buildings was severe enough to warrant the demolition and reconstruction of the structures. In 1967, the TTC completed a study of earth pressures on shoring systems. In the study, strut loads were measured at various sections during construction with the loads then distributed as a pressure on the back of the wall. This study adopted the "apparent earth pressure" concept outlined by Terzaghi and Peck (1947), later described by Peck (1969). During the 1960's and 1970's, shoring systems were specified to be designed using diagrams provided directly on the contract drawings (see Fig. 2). In general, these diagrams utilized one of several "earth pressures" for different soil categories as defined by the 1967 study. The earth pressure was then utilized for specification of soldier-pile design bending moments assuming that the pile acted as a simple, uniformly-loaded span between strut locations; i.e:

$$M_{max} = wL^2/8 \quad (1)$$

where M_{max} is the maximum design bending moment, w equals the uniformly distributed load on the beam, and L is the maximum unsupported span of the beam (between struts in this case). In areas where damage to buildings was of particular concern a contiguous caisson wall (drilled secant piles) was specified, rather than permitting a conventional and less expensive soldier-pile and lagging wall. This latter approach worked reasonably successfully through the last period of major urban subway construction in Toronto.

In the early 1990's the TTC embarked on a subway expansion program and this project was one of the first to be designed and constructed. Considering the history of TTC projects, ground conditions, and local shoring design practice the geotechnical consultant provided an apparent earth pressure diagram for the outline design and assessment of shoring systems (Fig. 3). Figure 3 illustrates the effect of beam continuity assumptions. Assuming that the beam is continuous between supports (1990s practice) effectively reduces the design pressure (or bending moment) by about 20% compared to the simple beam assumptions (late 1960s practice).

An empirical relationship between ground conditions and settlements adjacent to braced excavations was used during the project design stage as a tool for evaluating the potential performance of soldier-pile and lagging excavation support systems (see Fig. 4). For these early evaluations, deformations associated with contiguous caisson walls were considered to be about half those illustrated in Fig. 4. Using these principles, the generalized expected movements of the surrounding structures were assessed in a two-step risk-evaluation process (e.g. Boone et al. 1998 and 1999a). Categorization of building damage potential was based on the criteria provided by Boscardin and Cording (1989). If ground movements from a conventional soldier-pile and lagging shoring system were judged to be too severe in the first step, the assessment process was repeated using incrementally more sophisticated soil-structure interaction models and stiffer wall systems to select the best balance of cost and risk for the project. Based on these analyses, it was considered that if a number of provisions were made in the contract, adequate control of ground movements could be achieved with a soldier-pile and lagging excavation support system in most areas. It was considered that if damage could be limited to "slight" or less the cost-risk of making minor decorative repairs to adjacent buildings would be far less than the cost of constructing more robust shoring systems that, in any case, may not have been able to eliminate all damages. Requiring use of a secant pile wall for the entire project could have resulted in an approximate 60% to 100% increase in excavation support costs (between 10% and 15% of the total project cost).

Where ground movements are of concern it has been considered

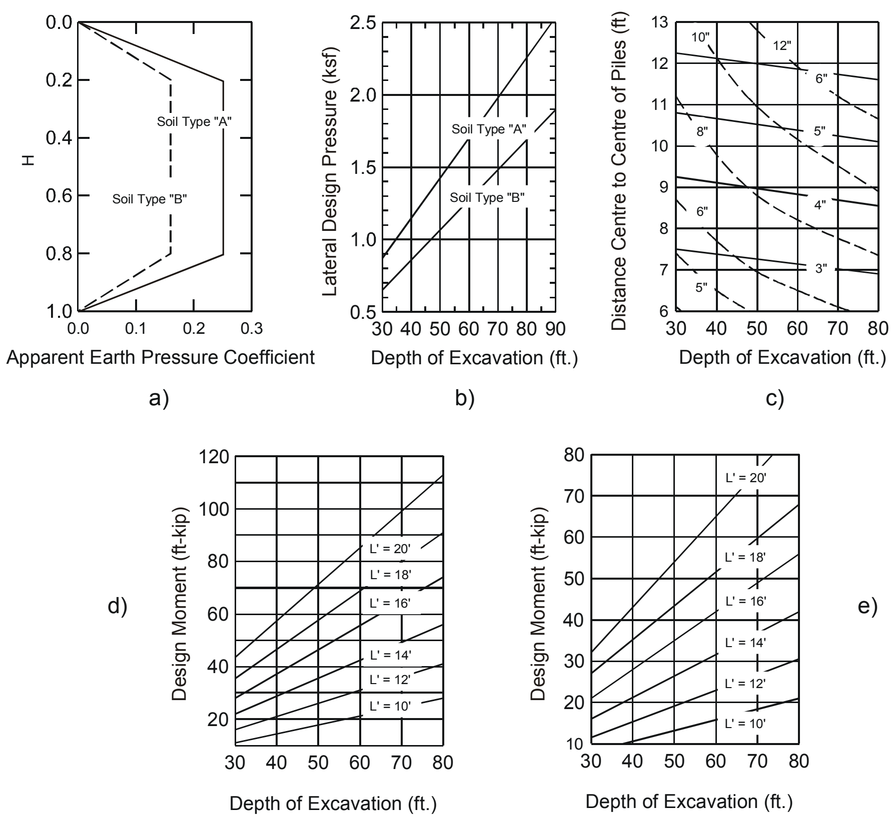

Figure 2. Requirements for design of soldier-piles and lagging excavation support for Sheppard Station in 1969: a) shape of apparent earth pressure diagram; b) apparent earth pressures; c) contours of lagging thicknesses for soil types "A" and "B"; d) bending moments for pile design with spans of length L and soil type "A"; e) bending moments for soldier-pile design, soil type "B" (from TTC 1969).

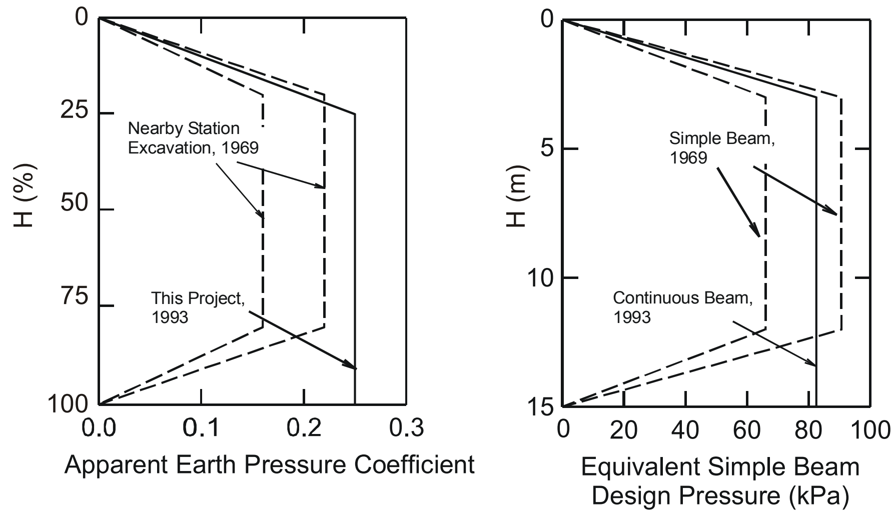

Figure 3. Comparison of earth loads between 1969 and 1993: specified apparent earth pressure coefficients and distributions (left) and resulting pile design pressures (right).

good practice over the past 20 to 30 years to "pre-load" the struts during installation (e.g. Peck 1969, Peck et al. 1973, Goldberg et al. 1976, O'Rourke 1981, Mana and Clough 1981, CNBC 1990). Pre-loading generally has two effects: 1) to compress the "slack" in the connection between the strut and wall; and 2) to restore or increase confining stresses within the retained earth mass prior to further excavation so that ground deformations are reduced. The final contract requirements specified the use of the full apparent earth pressure (Fig. 3) for final design of the soldier-pile and lagging wall system, and that struts were to be pre-loaded to a minimum of 50% of this design load. As a number of factors controlling ground deformation depend on workmanship, maximum ground displacement criteria derived from the design assessments were also included

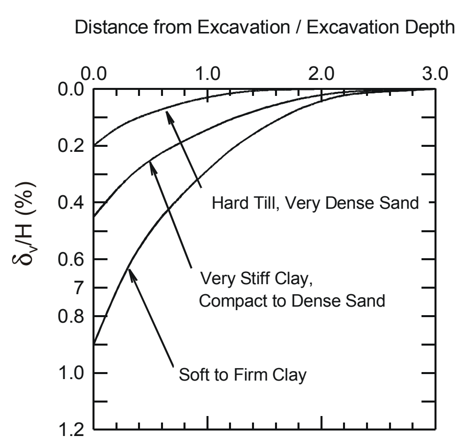

Figure 4. Empirical relationship between ground type and anticipated displacement due to deep excavations.

in the contract so that reasonable and clear performance targets were established.

MONITORING

A detailed instrumentation program was undertaken to monitor the contractor's compliance with the ground and structure movement criteria. Instrumentation relevant to this project included:

- 18 inclinometers in the ground behind the shoring system;
- 92 ground monitoring points consisting of steel rods with their ends grouted 1.8 m below the ground surface;
- 237 structure monitoring points; and
- 79 vibrating wire strain gauges installed in arrays where each strut and deck beam in a vertical section of shoring was instrumented.

Surveying of building and ground movement was completed with electronic levels achieving a typical accuracy of ±1 mm. Typically, the instruments were read on a daily or weekly basis (depending on the instrument) when excavation or strut removal/backfilling work was occurring within 50 m of the instrument. When such work was paused or had been completed, reading frequency decreased to approximately 1 reading per month.

DETAILED SHORING DESIGN

During the early stages of construction, an alternative shoring scheme was proposed by the contractor’s designer. This alternative scheme was based on several design assumptions:

1) where the excavation was less than 12 m deep, the contractor proposed utilizing a single-strut system with a sliding deck-beam connection;
2) the contractor judged that, by virtue of using a single support, a conventional "active" earth pressure distribution could be utilized for design;
3) to limit the cost of piling, the contractor also elected to apply a "reduction factor" of 0.67 to the earth pressures used for design of the piles (e.g. Peck et al. 1974); and
4) since struts were to be directly welded to the piles (no wales were to be used), the contractor considered that pre-loading the struts was unnecessary.

The reduction of design earth pressures for shoring walls has been advocated for sheet-pile bulkheads by Rowe (1952, 1957), and for excavations with multiple levels of support by Peck et al. (1973) and others. Reduction factors as low as 0.67 (Peck et al. 1973) to 0.8 (e.g. Goldberg et al. 1976) are used depending on the particular situation. Reducing the load assumes that the wall between the supports will deform sufficiently to allow "arching" to occur, thereby shedding the load to the struts. Depending on the soil conditions, the degree of arching and load sharing between the wall and struts is assumed to achieve equilibrium at some undefined level of deformation. While this general approach to retaining system design may be adequate to satisfy ultimate stability, the geotechnical consultant, the project designer, and the owner judged that this approach was not suitable for an area where control of deformations was of concern and did not accept the alternative proposal.

The final excavation support system design generally consisted of wide-flange steel beams (soldier-piles) placed in pre-bored holes with wood lagging installed between the piles as excavation progressed. Soldier-piles were installed with a 3 m center-to-center spacing. Pile toe depths (below excavation level) were typically 2.5 m for excavation of about 9 m deep to about 3.5 m at the section where the excavation was about 20 m deep. Horizontal restraint was provided by deck beams and pre-loaded pipe struts connected to each pile. Each strut was connected to the piles at either end by a wide plate-steel flange welded directly to the piles similar to shoring of the Berlin subway excavations (e.g. Muller-Haude and Von Scheiber 1965). Pre-loading of struts to 50% of the strut design load was accomplished by inserting a flat-jack into a notch cut within the flange, jacking in the required load, welding the connection and removing the jack. The vertical spacing of struts generally ranged between 2.4 and 5.8 m, resulting in each pile pair being restrained by the deck beam and two to three struts below. Because the excavation was made beneath a street, it was fully decked during construction, except for small openings for removing spoil and equipment and lowering lagging, bracing, and other construction materials. The interior excavation was also supported by soldier-piles and lagging with a pipe strut generally 1 to 2 m below the main excavation bottom and pile toe depths of between 2.5 and 3 m.

CONSTRUCTION PERFORMANCE

Two major construction events discussed below, coupled with construction choices made early in the project resulted in a variety of conditions that serve to illustrate important design and construction principles.

Shoring Wall Stiffness

Prior to resolving the contractor's desire to use an alternative

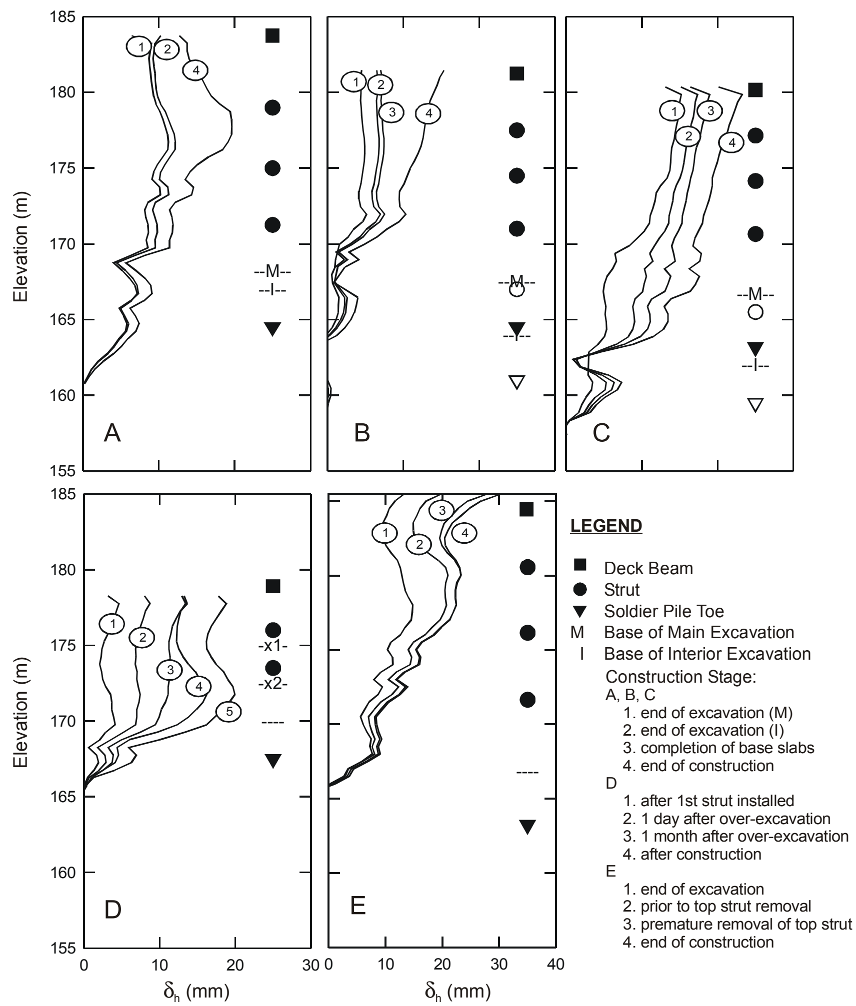

Figure 5. Example lateral displacement performance relative to construction stage and incidents.

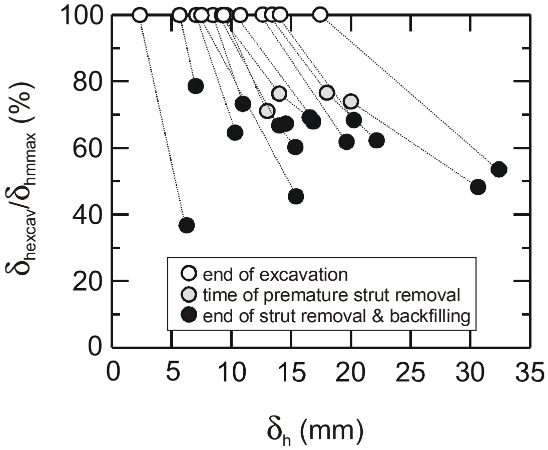

Figure 6. Proportion of maximum lateral displacement, δ_{hmax}, occurring during various construction stages.

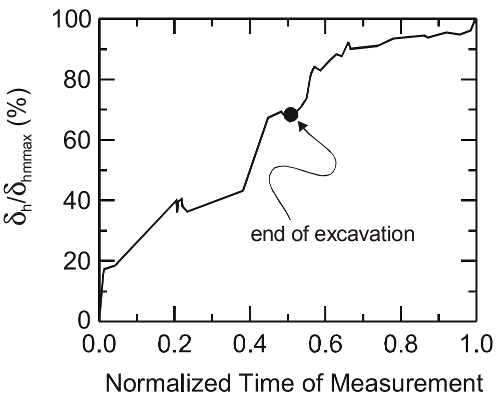

Figure 7. Lateral displacement occurring during life of project (normalized time = days elapsed/total days until excavation fully backfilled).

shoring design, pile installation proceeded. By rejecting the alternative shoring design, the contractor then included additional struts in the final design to account for the flexibility of piles in the eastern and western ends of the project. The middle section, designed last, was designed based on full application of the apparent earth pressure diagram within a beam-on-elastic foundation model (thus considering the beam continuous between strut locations).

Following the work of Mana and Clough (1981) and Clough et al. (1989), the relative, non-dimensional stiffness (S_r) of a particular vertical section of a soldier-pile and lagging wall can be estimated using:

$$S_r = EI/(\gamma h^4) \qquad (2)$$

where E is the elastic modulus of steel, I is the moment of inertia of the steel section *per unit length* of the wall, γ is the total unit weight of the soils, h is the average vertical distance

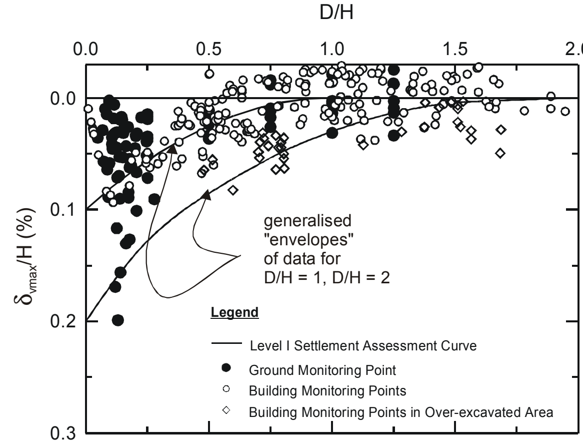

Figure 8. Summary of relative vertical displacements compared to design envelope.

between strut/support locations with the bottom of the excavation considered a support location. The shoring designs used for the project produced S_r values ranging from less than 1 (initial proposal) to greater than 20 (as-built, east and west ends).

Lateral Ground Movements

Lateral movements, δ_h, during construction of a braced excavation occur primarily as a result of: 1) deformation prior to strut installation at or below each excavation/bracing stage; 2) compression of the struts and connections; and 3) deformations as the struts are removed during backfilling. Other sources of ground movements include disturbances during pile or wall installation and ground losses and stress relief during excavation for and installation of lagging for soldier-pile and lagging walls. Figure 5 illustrates the general development of lateral movements during excavation, until the structure invert was placed, and subsequent movements during strut removal and backfilling. Figures 6 and 7 suggest that an average of approximately 60% of the maximum movement occurred during the excavation stage with the remaining displacement occurring during strut removal and backfilling. Near the middle and east end of the project, a number of struts were prematurely cut during backfilling. The premature removal of struts caused additional ground movements that were readily quantifiable as illustrated by Figures 5 and 6. Near the west end of the project, an area was over-excavated below a planned strut level between a Friday and the following Monday. The over-excavation resulted in an approximately 4.5 m span below the installed strut (about double the span called for in the final shoring design). This condition persisted for about two weeks prior to strut installation. Figure 5d illustrates inclinometer movements in this area where it can be seen that approximately 58% of the total movement occurred because of this incident. At this particular location, the relative stiffness for the contractor's initially proposed design (without the second strut and with a sliding

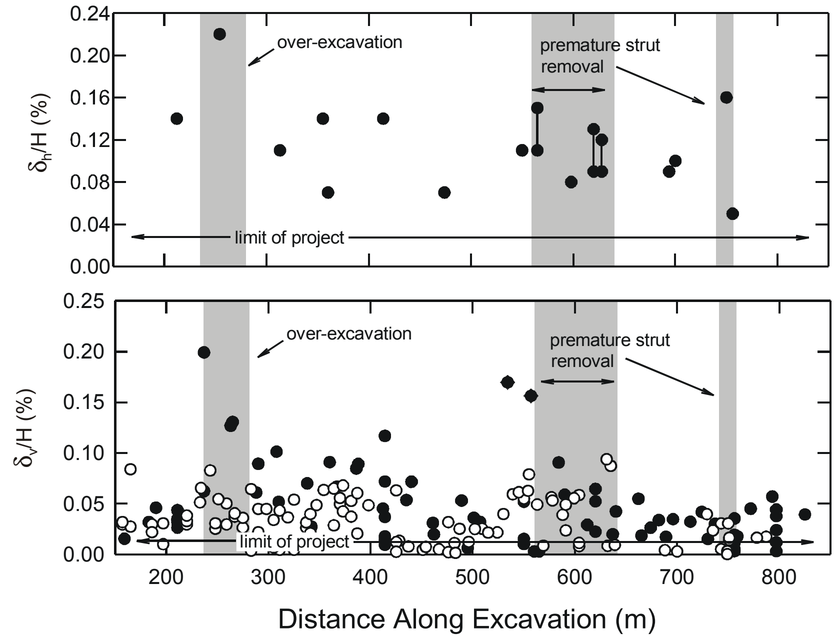

Figure 9. Summary of lateral (top) and vertical (bottom) displacements relative to position along excavation (open symbols represent measurements of building displacement.

deck beam) would have resulted in $S_r \approx 0.9$. The relative stiffness at the time of over-excavation was about 10 and the final S_r of other nearby areas constructed according to the final design was about 24. The over-excavation provides insight into the ground deformation that might have resulted if the contractor's proposed design had been adopted.

The lateral movement patterns exhibited by the inclinometers were primarily cantilever in shape but also included some "bulging" between the supports (Fig. 5). Final lateral movements along the excavation, relative to the excavation depth, are illustrated in Fig. 9 along the entire excavation. In this figure, the effects of both the over-excavation and premature strut removal are evident. From Fig. 5 it can be seen that movement occurred below the toe of the piles but the interior excavation had little influence on the total movements experienced at each inclinometer location. Strut pre-loading workmanship was also a factor in lateral movements as discussed in more detail below.

Vertical Ground Movements

Ground settlements, δ_v, of up to 31 mm were measured as summarized in Figures 8 and 9. The locations of over-excavation and premature strut removal are clear in these graphs. Maximum measured vertical displacements were consistent with the maximum measured lateral displacements along the excavation, as suggested by Figures 8 and 9. Figure 10 illustrates the patterns of lateral and vertical displacement at several monitoring sections. Figure 10d compares the shape of the vertical displacement profiles with the equation suggested by Bowles (1996). For all monitoring stations, the equation adequately described the shape of the displacement profile, except at one location where the measurements were so small that they were likely influenced to a large degree by survey error (δ_{vmax} at this location was less than 5 mm, open symbols in Figure 10d). The "zone of influence", or D_{max}/H, for each section was typically close to 1.0, though where settlements approached 0.2%H, the measured zone of influence varied between 1.5 and 2.0.

Strut Pre-loading

The struts were pre-loaded according to the general procedure outlined in the design as discussed above. Strain gauge readings were generally taken immediately prior to pre-loading to provide a "zero" reading while the strut sat on its end supports. Readings were subsequently taken at full jack load and immediately following removal of the jack. It became evident, however, that loads were being lost during the pre-loading process. Fig. 11 illustrates a typical plot of compression load from the strain gauges from the initial reading until the gauge was removed from the strut. It was determined that the welds at the connection between the strut and pile were only being partially completed prior to removing the jack in many instances. The combination of weld quality, weld area, and remaining gap between the pile and strut plate-flange allowed compression to take place and subsequent relaxation of the strut pre-load. After recognizing this workmanship issue, the contractor made efforts to ensure adequate load transfer. The last part of the excavation was made near the middle of the project and load-transfer during pre-loading was optimum and

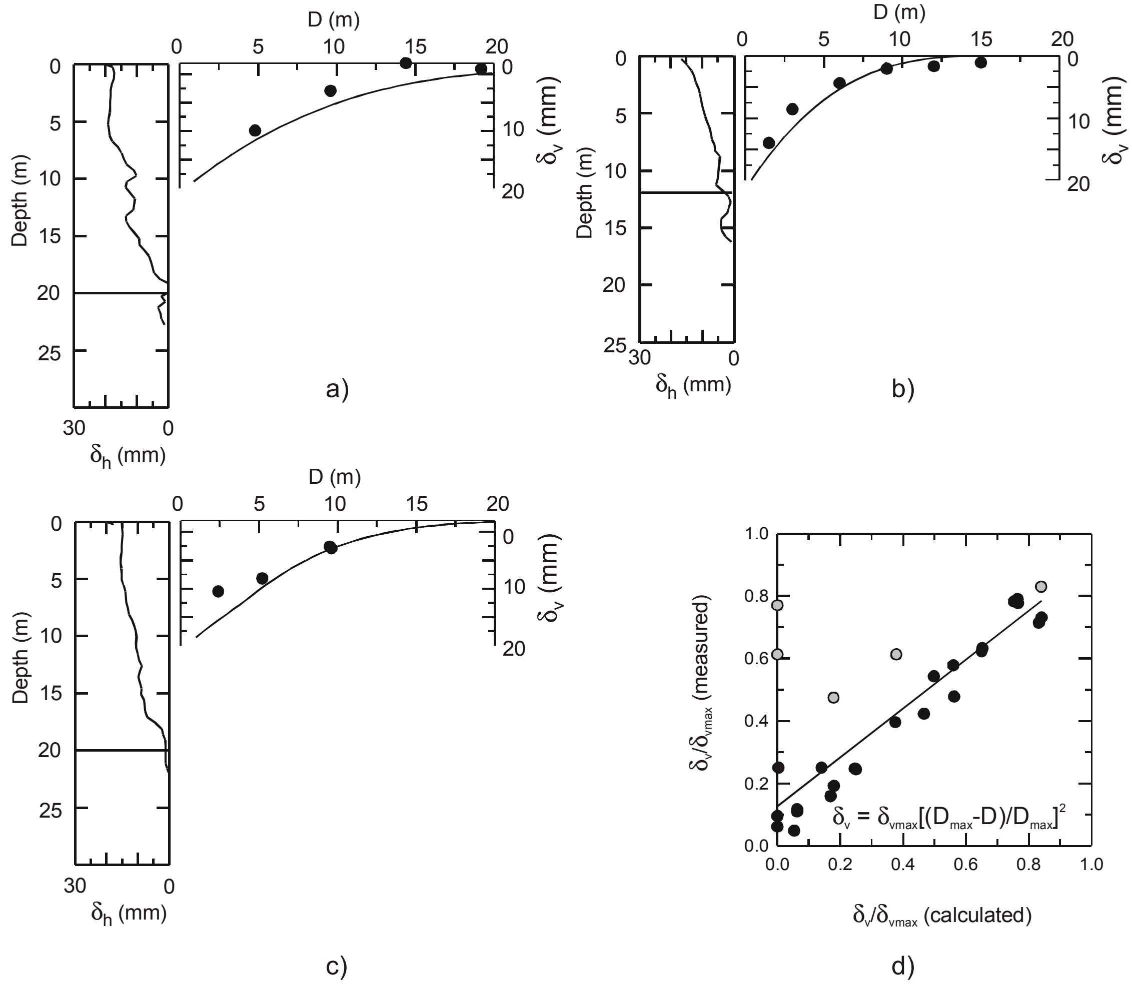

Figure 10. Patterns of lateral and vertical displacement at monitoring stations.

ground movements were minimized in this area. Boone et al. (1999b) and Bidhendi et al. (2000) provide further examination of pre-loading methods, pre-load losses, and their effect on lateral shoring deformations having accounted for the effects of premature strut removal and over-excavation. Figure 12 illustrates the influence of pre-loading on controlling lateral displacement.

Maximum Strut Loads

Following pre-loading and subsequent pre-load loss, it was observed that all struts exhibited an increase in compression loading, illustrating that the combination of earth and temperature loads exceeded the realized pre-load (as illustrated by Fig. 11). Strut load data was compared to the apparent earth pressure diagram used for strut design as illustrated in Figure 13. On average, the strut loads were about 73% of the maximum load that could be indicated by the design diagram. One strut exhibited significant corrosion and therefore, the thickness was likely reduced leading to the interpreted high stresses and load indicated by the point in Fig. 13 where an earth pressure coefficient of about 0.45 is shown. The large amount of strain gauge data also allowed detailed

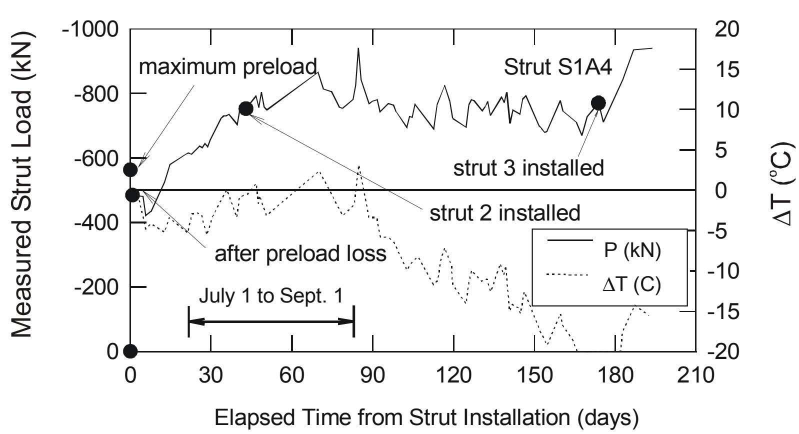

Figure 11. Measured load on one strut illustrating immediate loss of pre-load following jack removal and showing influence of temperature on strut load.

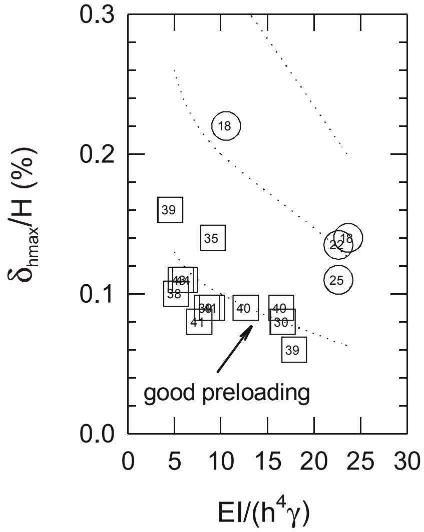

Figure 12. Effectiveness of pre-loading in control of lateral displacements. Numbers indicate measured pre-load expressed as a percentage of design load.

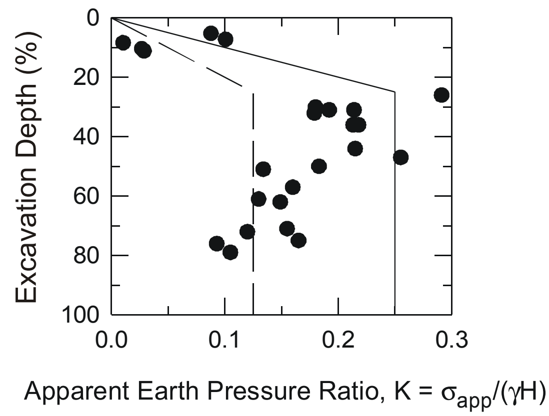

Figure 13. Comparison of maximum measured strut loads to specified apparent earth pressure diagram. Dashed line indicates minimum pre-load requirement.

analysis of important strut load dependency on temperature (Boone and Crawford, 1999).

Building Damage

The vertical and lateral ground displacements caused a number of the nearby buildings to suffer some degree of damage. Most of the building met the design criteria and suffered only "negligible" to "slight damage". Several of the buildings near the deepest section of the excavation, however, suffered "moderate" damage. The extent of the damages and displacements are described in detail by Boone et al. (1999a). In this area of the project, the shoring was designed by applying the full apparent earth pressure for pile design, and pre-loading of the struts was reasonably well controlled. Ground displacements in this area were close to but slightly above the threshold of displacements judged tolerable at the outset of the project design. Overall, the full cost of damages buildings, including all insurance, labor, and physical repairs was less than about 30% of the cost of installing more robust shoring systems in the affected areas.

DISCUSSION

The observed performance of this braced excavation highlighted a number of important elements of shoring design and performance. The variation in wall stiffness and pre-loading workmanship and their effect on ground deformations were quantitatively illustrated by field results from a detailed instrumentation program. Lateral ground movements were directly related to shoring wall stiffness even for the generally competent and stratified soils found at the project site. Measured strut load behavior demonstrated that strut pre-loading is an effective aid in controlling ground movements, regardless of the strut-wall connection detail. Field data indicate that considering maximum vertical movements (settlements) equal to maximum lateral movements was a reasonable design assumption for this case. Measurements of settlements at various positions away from the excavation edge illustrate that the settlement zone was approximately equal to the excavation depth and its approximate shape can be described by a parabolic equation.

CONCLUSIONS

It is well recognized that apparent earth pressure diagrams do not necessarily bear any relationship to the final distribution of earth pressures on any one particular shoring system. Selection of apparent earth pressure diagrams may be either arbitrary or highly dependent upon local experience, and subject to considerable debate. Although valid in some cases, the application of load reduction factors was shown to be unsuitable for design for this project as they lead to a more flexible wall system. Based on the experiences drawn from this project, other projects have subsequently been specified and designed on the basis of a minimum relative stiffness value, using earth pressure values only as a check on the ultimate stability and safety of the structural members. In such cases, the displacement of the wall and ground, having satisfied safety concerns, are the primary criteria related to control of urban excavations and their effects on neighboring facilities. Since the relative stiffness criterion is well-defined and independent of debatable variables, its use in specifying minimum design criteria for shoring design can assist in limiting disputes during construction. This practice is in general keeping with the principles of the TTC practice during the 1970's and 1980's of specifying minimum bending moments for the design of soldier piles to have better control over the displacement performance of the shoring. Specification of minimum relative stiffness values provides a simple measure to accomplish the same goal and is also in keeping with more recent research and case histories in which displacement is of critical concern (e.g. Clough et al. 1989, Koutsoftas 1999). Although it has been stated that for stiff or dense soils elastic ground responses govern displacements and little can be done to control these (e.g. O'Rourke 1981), this project clearly demonstrates that the wall design plays an integral role in limiting ground movement.

Since design of excavation systems often relies on assumed loading and assumed beam behavior (i.e. simple beam or a continuous beam), both the specifier and designer of shoring systems need to be aware of the pitfalls of combined assumptions when designing for deformation control. The use of apparent earth pressure diagrams for both specification and design, while typical in practice, can lead to disputes related to the details of the design and have little true relationship to displacement criteria. One of the most important elements of this project, however, is that a detailed instrumentation program was implemented for evaluation of performance and contractor compliance with the contract. In addition, the data has subsequently permitted continued re-examination of fundamental principles related to shoring design.

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