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Control of Ground Movements for a Multi-Level-Anchored, Diaphragm Wall During Excavation

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CONTROL OF GROUND MOVEMENTS FOR A MULTI-LEVEL-ANCHORED, DIAPHRAGM WALL DURING EXCAVATION

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ABSTRACT

An excavation up to 23m deep for the Dana Farber research tower in the Longwood medical area of Boston, was supported by a permanent perimeter diaphragm wall extending into the underlying conglomerate and up to 6 levels of prestressed tiebacks anchored in the rock. The lateral earth support system was very successful in limiting wall deflections to less than ±15mm on each of the four sides of the excavation. However, ground surface settlements up to 65mm occurred on two sides and were attributed to ground losses that occurred when tiebacks were installed through overpressured sand layers at depths of 15-18m. Finite element simulations are able to describe consistently the effects of the different excavation and support sequences on each side of the project using backfigured soil properties, while surface settlements can be explained by including local ground losses within the analyses.

INTRODUCTION

The Dana Farber Research Tower (Smith Laboratory) is located in the Longwood Medical Area in Boston, Massachusetts. The tower has 14 above-ground stories devoted to office and research laboratory uses and five underground parking levels. Figure 1 shows the site plan, the building has a rectangular footprint (approx. 43m x 34m) and is bounded to the south by the MATEP (Medical Area Total Energy Plant) power plant, by Binney Street and Deaconess Road on the east and north sides, respectively, and by two low rise brick buildings to the west. In order to protect the delicate (and expensive) medical experiments carried out in the research tower, the basement structure is isolated from a permanent lateral earth support system comprising a 0.9m thick perimeter diaphragm wall braced by four to six levels of prestressed rock anchors.

This paper summarizes the performance of the lateral earth support system based on field monitoring data measured during excavation of the basement (January – October 1995). Back analyses are then used to evaluate and interpret the wall and ground movements.

SITE DESCRIPTION

The original site investigation comprised a series of 14 deep borings (with conventional blowcount data) within the footprint of the new building, supplemented with boring data from three adjacent structures. There was a very limited program of laboratory tests (index properties, water content and UU strength testing in the clay, and particle size distributions for the granular layers) while permeability properties were reported from borehole falling head tests.

Figures 2a and b summarize the soil profiles for the two orthogonal cross-sections A-A and B-B shown in Figure 1. The site is almost level with surface elevation El. 11.6m – 12.2m. The subsurface profile comprises 20m - 27m of post-glacial sediments overlying the Roxbury conglomerate bedrock. The bedrock was described as medium to hard, slightly weathered gray to purple, coarse-grained conglomerate with closely spaced dipping joints and RQD = 28% - 40%. The surface of the rock is quite irregular and dips from north to south across the site (Fig. 2a).
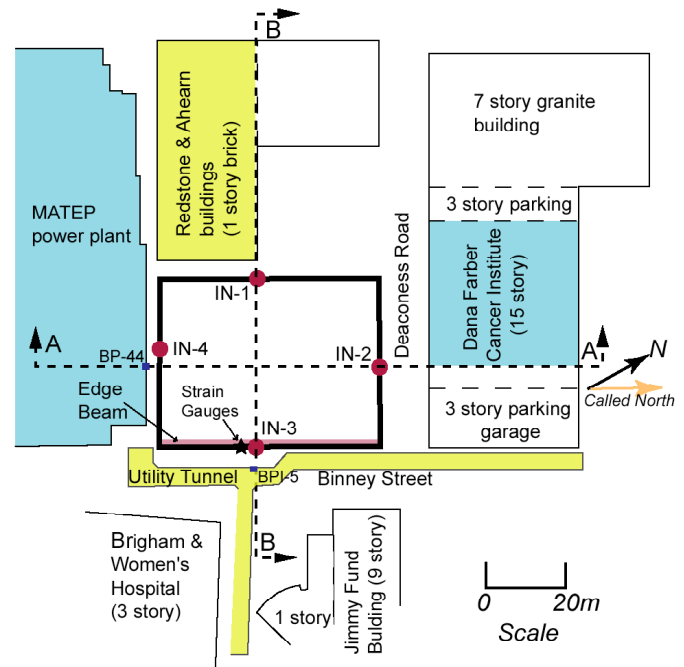

Fig. 1. Dana Farber research tower site plan

The overlying soils can be sub-divided into four main units: 1) surficial fill (up to 5m thick at the southern edge of the site), 2) low plasticity (I_p = 10—15%) marine clay (Boston Blue Clay) ranging in consistency and coloration and ranging in thickness from 10m – 17m (the upper clay contains discontinuous pockets of sand), 3) deposits of underlying sands and silts which taper to the north and south of the site but appear more continuous in the east-west plane; and 4) a drape of glacial till (0.3m – 3.0m thick) comprising very dense sand with silt and gravel.

The clay deposits range in consistency from a very stiff, oxidized crust (yellow coloration) to stiff (with gray coloration), and corresponding undrained shear strengths from UU tests, s_u = 90kPa, 60kPa, respectively.   The clays are clearly overconsolidated, but no consolidation tests were performed for this project. The underlying silts, sands and till layers were found to have relatively high permeability in the range, k = 0.3 – 3.0m/day. The 'silty fine sand' and glacial till layers were classified as very dense layers based on SPT data. In some locations (Fig. 2) the sand there are looser deposits of 'silt with sand' (N = 12 – 37bpf) directly beneath the marine clay.

Groundwater conditions were measured by a series of piezometers screened within the underlying sands, till and rock layers. These data consistently show the groundwater head at El. +3m. There is a perched groundwater table in the overlying fill which is typically 1m - 2m below the ground surface.
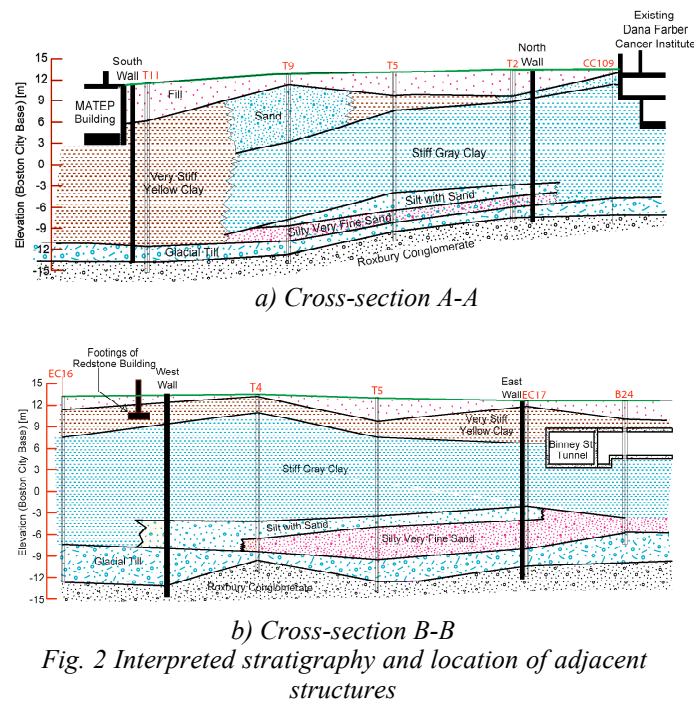

a) Cross-section A-A

b) Cross-section B-B
Fig. 2 Interpreted stratigraphy and location of adjacent structures

Figure 1 shows that the research tower site is surrounding by existing structures. The designers made a detailed assessment of the vulnerability of each structure to excavation-induced ground movements. The most vulnerable are the utility tunnel beneath Binney Street and the Redstone and Ahearn buildings. The concrete utility tunnel has a 4.6m square section with soffit at El. +3m -+4m and is located approximately 0.6m from the property line along the northern half of the East slurry wall. It was originally built by a cut-and-cover technique. The Redstone and Ahearn buildings are one and two story brick

buildings that are within 2m of the West slurry wall. These structures were underpinned (along their southern edge) during construction of the MATEP power plant.

The MATEP power plant, immediately adjacent to the South slurry wall, is founded on a 1.5m thick concrete mat foundation at El. +2.7m. This foundation was expected to distribute the ground movements and hence, was less likely to suffer any damage during excavation. Similarly, minimal effects of the excavation were anticipated for the existing Dana Farber Cancer Institute, located 12m from the North slurry wall and founded on a shallow mat at El. 9.1m.

LATERAL EARTH SUPPORT SYSTEM

The permanent lateral earth support system comprising a 0.9m thick perimeter slurry wall with up to six levels (P1 – P6) of rock anchors, was designed to resist the lateral earth and pore water pressures, seismic loads and surcharge loads from construction equipment and adjacent structures. The tieback anchors were also specified with zero tolerance against water leakage in order protect the steel tendons from long term corrosion problems. This design requirement presented a particular challenge for the three lowest levels of anchors (P4 – P6) which were installed through the highly permeable sand and silt layers with an artesian/over pressure condition (i.e., the anchor head elevation is less than the piezometric head within these layers). The final anchor design used three levels of waterproofing including a flexible water-swelling seal (Shields, *pers. comm.*), further details can be found in Konstatakos (2000).

The perimeter slurry wall was constructed in a series of 6m long slurry panels each extending a minimum of 0.6m into the underlying Roxbury conglomerate. The tieback anchors were installed through steel sleeves cast into the slurry wall. Each anchor was inclined at 45° with minimum fixed anchor lengths of 6m in the bedrock. Horizontal spacing of the anchors ranged from 1.65m to 3.2m and each tendon comprised from 9 to 16 strands of 1.5cm diameter high tensile strength steel. Table 1 summarizes the average axial stiffness, free-length and measured lock-off loads for each level of anchors (per unit length along the wall) around the excavation. It should be noted that the top two levels of anchors (P1, P2) were absent along the South wall, due to the proximity of the MATEP mat foundation. The top of the East wall was braced by an edge beam cast at grade in order to avoid interference with the Binney Street utility tunnel.

The excavation performance was monitored by 1) weekly optical surveys of surface and building settlements (accurate to within 3mm), 2) inclinometer measurements of wall deflections from a series of 4 inclinometers (IN1 – IN4, Fig. 1) installed through the slurry wall and extending 5m into the underlying rock (these are accurate to within 2.5mm at ground surface), and 3) vibrating wire piezometer measurements of water pressures below the excavation in the underlying rock (2 locations), and outside the property line in the glacial till and sand layers (3 locations). Axial forces and bending moments in the slurry wall were also interpreted from two clusters of strain gauges and pressure transducers attached to the steel reinforcement at El. –22.5m on the East and North slurry walls (close to IN-3 and IN-2, respectively).

Table 1. Summary of rock anchors

Anchor	Parameter	North	South	East	West
P1 El. 9.0m	EA (x10^5 kN/m)	0.79	–	0.016[2]	0.79
	L_f (m)	24.4	–	–	32.9
	F_L (kN/m)	385	–	250[2]	397
P2 El. 5.9m	EA (x10^{5}kN/m)	0.96	–	0.96	0.96
	L_f (m)	20.1	–	25.0	28.0
	F_L (kN/m)	435	–	426	438
P3 El. 2.9m	EA (x10^{5}kN/m)	1.66	1.08	1.66	1.66
	L_f (m)	15.9	24.4	20.7	23.8
	F_L (kN/m)	795	510	1349	829
P4 El. −0.2m	EA (x10^{5}kN/m)	2.04	1.45	2.04	2.84
	L_f (m)	11.6	19.5	16.7	19.5
	F_L (kN/m)	965	695	1052	1486
P5 El. −3.3m	EA (x10^{5}kN/m)	1.14	1.95	1.14	2.30
	L_f (m)	7.0	15.2	12.2	15.2
	F_L (kN/m)	550	905	820	1024
P6 El. −6.3m	EA (x10^{5}kN/m)	1.14	2.72	1.14	2.30
	L_f (m)	4.6	11.0	7.9	9.1
	F_L (kN/m)	550	1135	867	1310

Notes: 1. F_L – lock-off load,
L_f – estimated free-length of anchor
2. Edge beam installed at ground surface (Fig. 1).

MEASURED PERFORMANCE DURING EXCAVATION

Figure 3 summarizes the timeline of excavation and anchor prestressing for average conditions along each of the four sides of the excavation (following prior slurry wall installation over a period of approximately 5 months). The accumulated time in this figure refers to the completion of each excavation/prestressing phase. It should be noted that there was no dewatering carried out during the excavation although there was a passive relief system of two wells installed in the till within the footprint of the excavation. Total water inflows ranged averaged 75 l/min, and maximum inflows (190 l/min) occurred during the first stages of excavation. Piezometers outside the excavation measured less than 0.6m change in head throughout the excavation.

Figures 4 and 5 compare the measured wall deflections and surface settlements at the South and North walls (i.e., section A-A, Fig. 1, 2a) at selected stages of the excavation. The results show a small (5mm) inward cantilever deflection of the North wall (Stage L1, Fig. 4b). However, subsequent excavation and prestressing of anchors causes a reversal in wall deflections such that there is a small net outward movement (up to 6mm at El. 5m) at the end of the excavation (stage L7, Fig. 5b). The toe of the wall rotates but shows no net displacement. The behavior of the South wall is strongly influenced by the proximity of the MATEP foundation mat. There is no space for the P1 and P2 anchors. However, the inward cantilever deflections (6mm) at L3 are similar in magnitude to those found for the North wall at L1. This behavior can be attributed to the stiffening effects from grouting that was carried out to mitigate shallow soil cave-ins that occurred during installation of the South wall. Prestressing of anchors P3 – P6 is able to control subsequent inward wall deflections to less than 18mm. The proximal edge of the MATEP power plant settled a similar amount (approx. 15mm) and there is less than 10mm of differential settlement across of the foundation mat. Settlements up to 20mm occurred behind

the North wall, but movements of the Dana Farber Cancer Institute were less than 10mm.
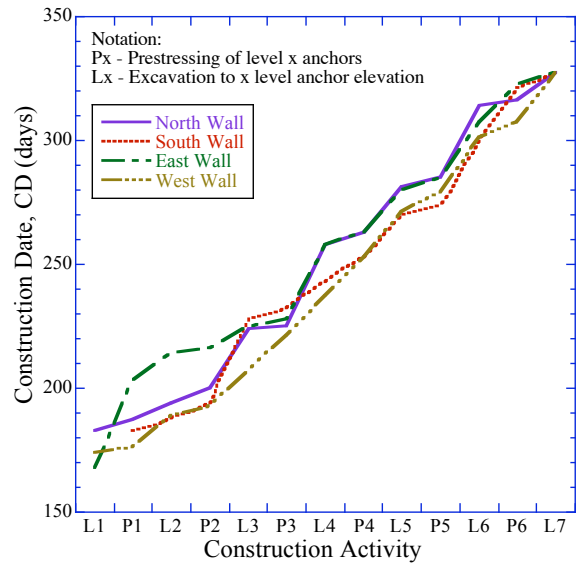

Fig. 3 Summary of construction activities

Figure 6 and 7 show analogous results for the East and West walls (i.e., section B-B, Fig. 1, 2b). Both walls show excellent toe fixity and both are prestressed such that there are maximum net outward movements up to 15mm (and quite similar deflection mode shapes). Settlements behind the East wall were measured at a series of reference points inside the utility tunnel (Figs. 6b, 7b). These data show very small settlements through the early phases of excavation (less than 3mm at stage P3). However, there are large increments of settlements associated with installation of tieback anchors at level P4 and further increases during the remainder of the excavation. Maximum settlements of the utility tunnel beneath Binney Street exceeded 70mm by the end of excavation (L7; Fig. 7b). There was a similar pattern of measured surface settlements for the Redstone Building, with maximum settlements up to 64mm.

The utility tunnel suffered some cracking as a result of these settlements and only slight damage was reported for the Redstone building. Nevertheless, the measured settlements exceeded prior predictions by more than 40mm and 50mm for the Redstone and utility tunnel, respectively. The behavior on the East wall was attributed, in large part, to ground losses that occurred during installation of the P4 anchors. Air pressures used during drilling of one anchor forced sand and water out through an adjacent anchor hole (soil was ejected approximately 5m according to the construction records). The ground was then stabilized by grout injections and subsequent drilling was performed in a more controlled fashion. Although, this event can explain some of the observed settlements for the East wall, there were no comparable incidents on the West wall. It is unclear why much larger surface settlements occurred for the East and West walls compared to the North or South walls (even allowing for the presence of the MATEP foundation adjacent to the South wall). It seems likely that small ground losses occurred for all anchors drilled through the overpressured silt and sand layers. These layers thin out to the North of the project site and were not present beneath the Dana Farber Cancer Institute (Fig. 2a) and this may explain the more satisfactory performance noted in Figs. 4b and 5b.
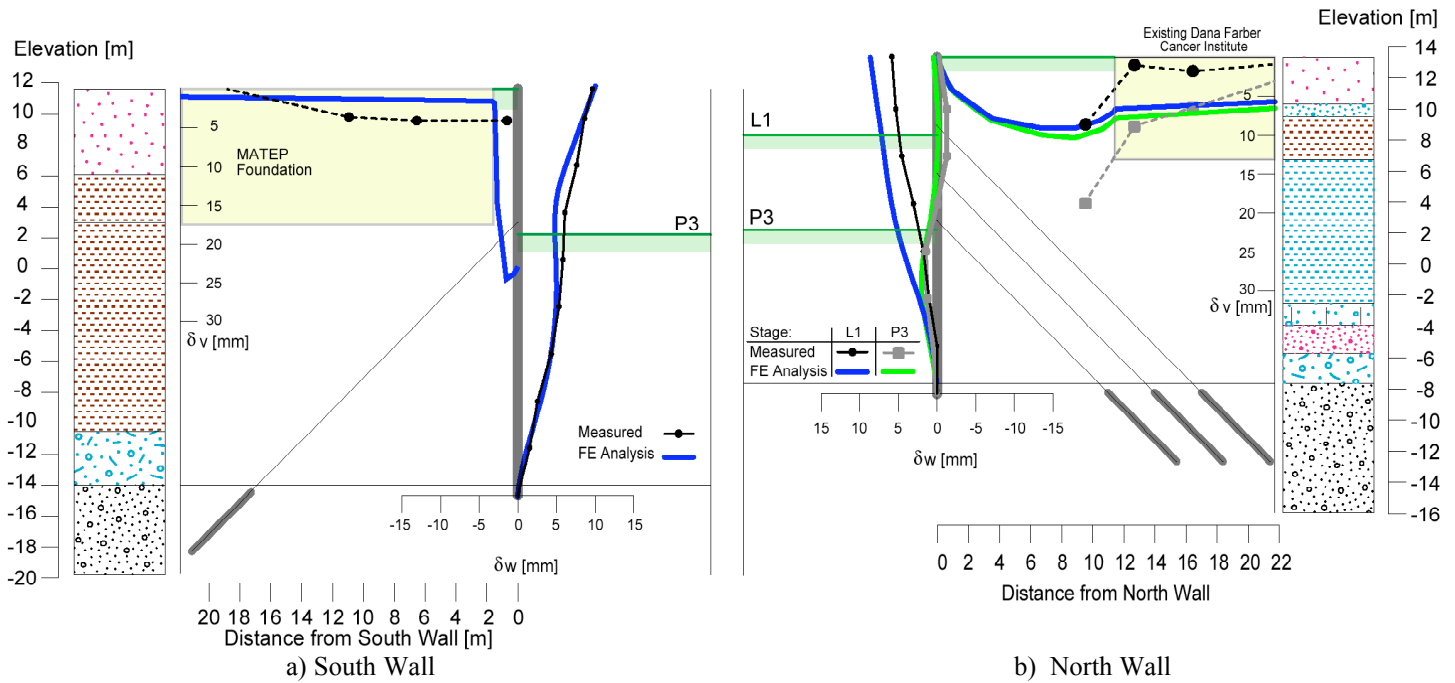
a) South Wall   b) North Wall

Fig. 4 Measured and predicted behavior of Section A-A walls at early stages of excavation

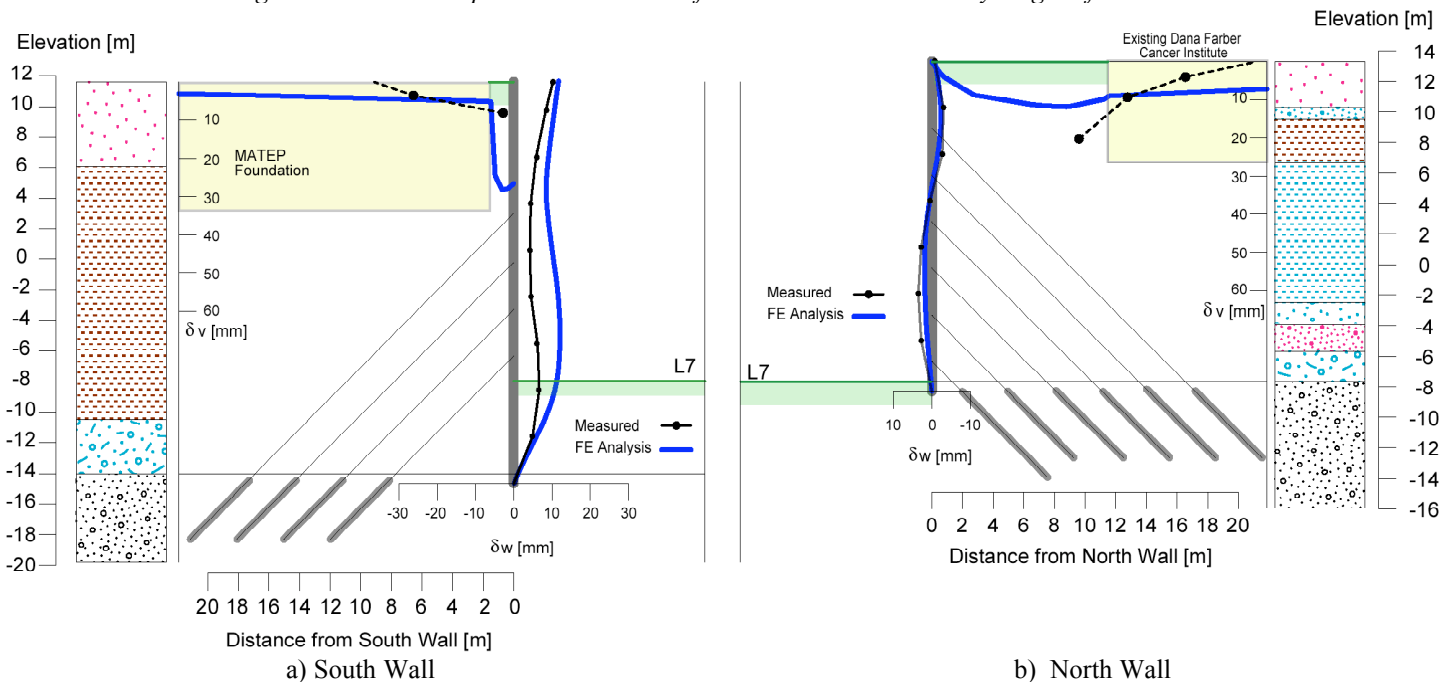
a) South Wall   b) North Wall

Fig. 5 Measured and predicted behavior of Section A-A walls at end of excavation

FINITE ELEMENT SIMULATIONS

A series of finite element simulations have been carried out to obtain better insight into the performance of the excavation support system for the Dana Farber research tower. The calculations have been carried out using the Plaxis finite element code (Brinkgreve, 2002) and comprise a series of four 2-D, plane strain models representing each of the four sides of the excavation. Given the almost complete lack of site specific data on soil deformation and shear strength properties, these parameters have been estimated based on prior experience, published correlations and case studies in the Boston area (e.g., Duncan et al., 1980; Johnson, 1989; Ladd et al., 1999; Altabba & Whittle, 2001; Hashash & Whittle, 1996). The soil

parameters have subsequently been refined in back-analyses for the North wall section. Each of the soil layers has been simulated using the Hardening Soil (HS) model (Schanz et al., 1999). This model represents an updated version of the well known Duncan-Chang model (Duncan et al., 1980), formulated using elasto-plasticity. The non-linear shear-stress strain behavior in loading is represented by a hyperbolic function (with average secant modulus, E_{50}, Fig. 8); while a much stiffer linear response in unloading is described by the parameter, E_{ur}. The shear strength is characterized by conventional Mohr-Coulomb parameters (c', ϕ'). The HS model enables a realistic description of the stiffness of the retained soil relative to the excavated material with minimal additional parameters.

Table 2. Model parameters assumed in finite element analyses

Layer	Model[1]	γ_t [kN/m³]	c' (s_u) [kPa]	ϕ' (°)	E_{50}[2] [MPa]	E_{ur}[2] [MPa]	m[3]	K_0	k [m/day]
Fill	HS	19.0	25	34	20	100	0.5	0.4	3.0
Sand	HS	20.0	–	34	30	90	0.5	0.4	0.1
Yellow Clay	HS(U)	20.0	90	–	25	100	1.0	0.8	5×10^{-4}
Gray Clay	HS(U)	18.5	60	–	25	100	1.0	0.6	5×10^{-5}
Silt & Sand	HS	20.0	–	35	35	105	0.5	1.0	0.001
Silty sand	HS	20.0	–	40	55	165	0.5	1.0	0.1
Glacial Till	HS	22.0	–	43	60	180	0.5	1.0	0.01
Bedrock	MC	23.6			2.8×10^4	$\nu = 0.15$	–	1.0	0.01

Notes:
1. HS – Hardening soil; (U) – 'undrained' capability to develop excess pore pressures during shearing
   Default parameters used: $E_{oed} = E_{50}$, $R_f = 0.9$ and $\nu_{ur} = 0.2$
   MC – linearly elastic perfectly plastic with Mohr-Coulomb yield
2. Stiffness parameters defined at reference pressure, $p^{ref} = 100$kPa
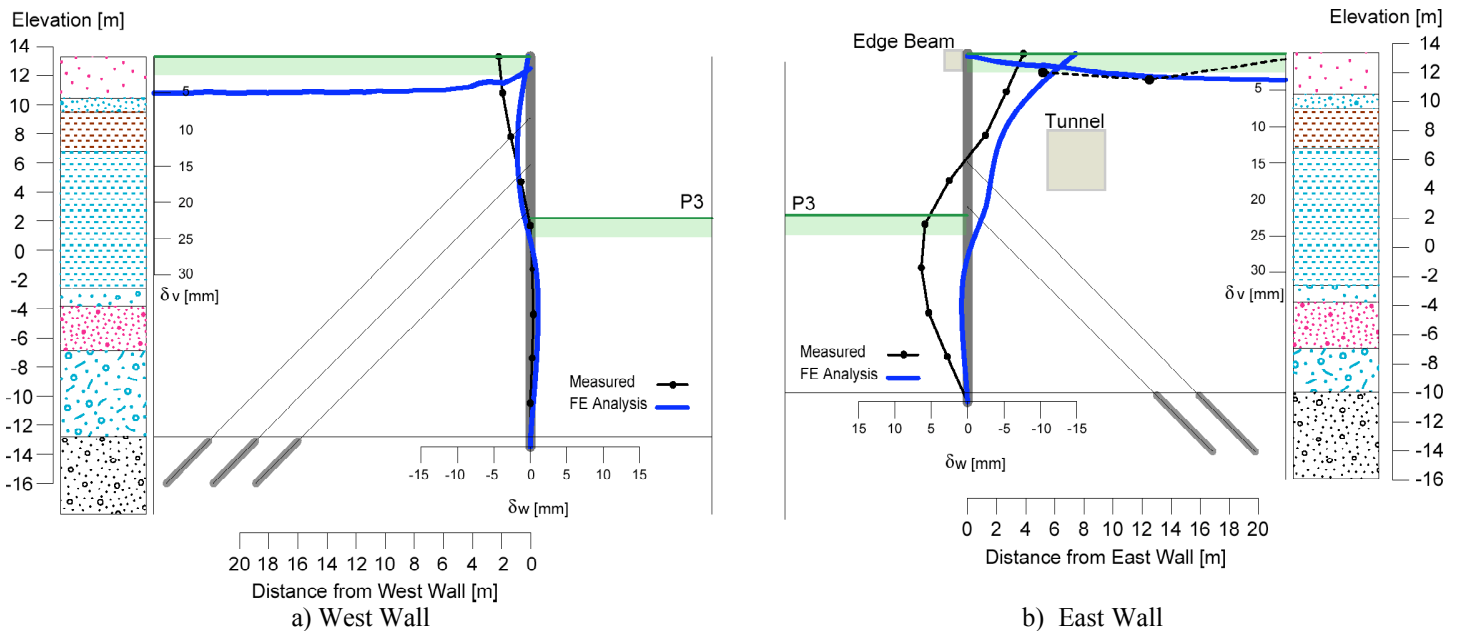3. All stiffness parameters $E \sim (\sigma'_3/p^{ref})^m$

a) West Wall

b) East Wall

Fig. 6 Measured and predicted behavior of Section B-B walls at early stages of excavation
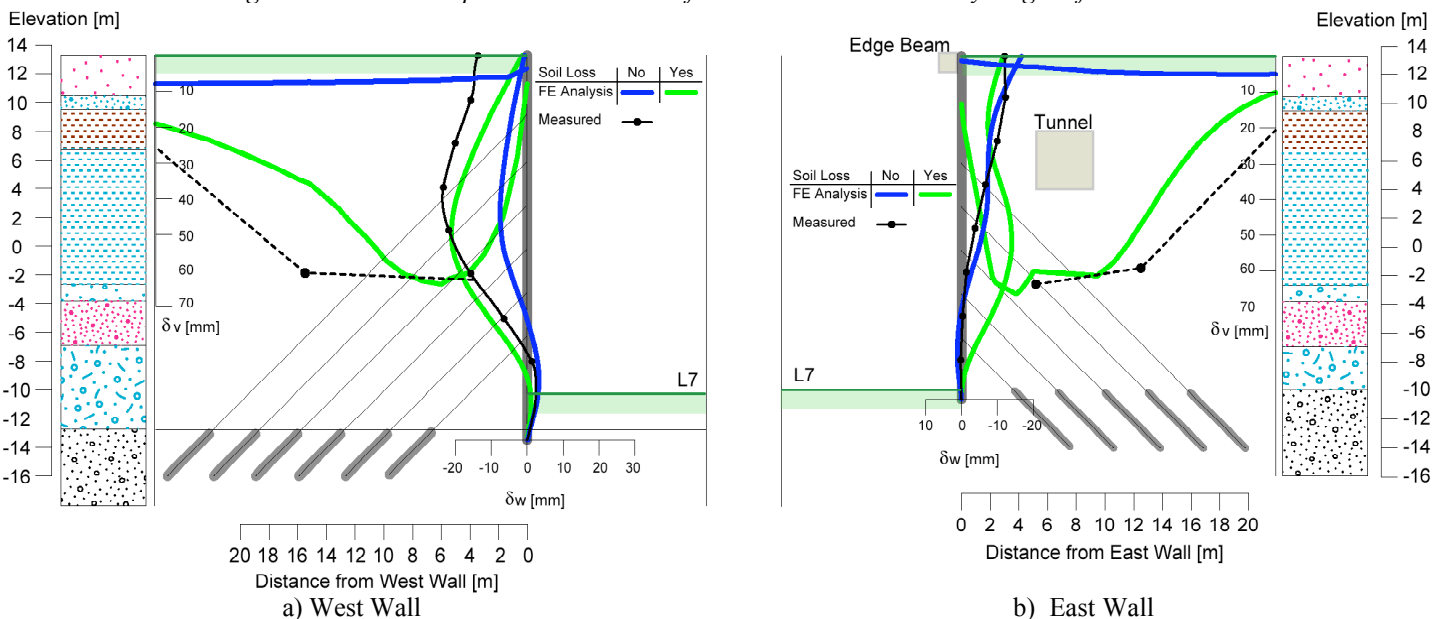

a) West Wall

b) East Wall

Fig. 7 Measured and predicted behavior of Section B-B walls at end of excavation

Table 2 summarizes the input parameters used to model the soil layers. Values of hydraulic conductivity were derived from field pumping tests, while undrained shear strength parameters for the clay were based on UU triaxial shear test data. Each of the four FE models considers a profile of horizontal soil layers (cf. profiles shown in Figs. 2a, b) with initial K_0 conditions listed in Table 2. The perimeter slurry wall was modeled using elastic beam elements (with axial and bending stiffnesses; EA = 2.52×10^7 kN/m and EI = 1.7×10^6 kNm^2/m, respectively), while elastic properties and prestress loads for the rock anchors are given in Table 1.
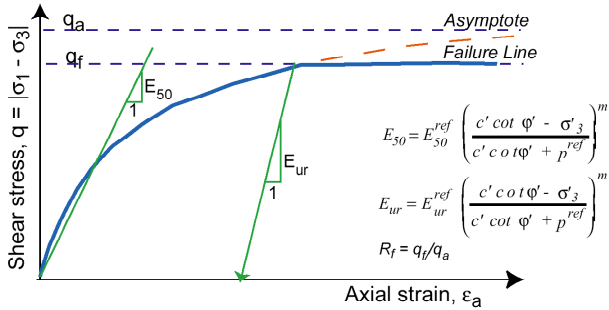

Fig. 8 Shear stress-strain behavior of Hardening Soil (HS) model (Schanz et al., 1999)

The FE models simulate closely the recorded construction history shown in Figure 3. Each of the phases of construction is modeled assuming coupled flow and deformation within the clay layers, while fully drained shear conditions occur in the rock and cohesionless soil layers.

The FE analyses generally overestimate the initial cantilever deflections of the North wall (with maximum wall deflections, $\delta_w \approx 10$m; Fig. 4b), but appear to describe quite closely the deflection mode shape during the subsequent stages of the excavation (e.g.. stages P3, Fig. 4b; L7, Fig. 5b). The computed settlements are also in good agreement with the measured deformations of the existing Dana Farber Cancer Institute, but tend to underestimate surface settlements at points on the Deaconess Road. Figure 9 shows a more detailed comparison of the computed and measured bending moments in the North wall, based on sister-bar strain gauges installed at El. −6.9m (data are presented for two pairs of gauges). The measured data show negligible bending of the wall at this elevation until prestressing of the 4th level of anchors (P4), suggesting that there is rigid body rotation of the wall at early stages of the excavation. The FE analyses overestimates the measured bending moments during the first part of the excavation, due to the assumed rotational fixity of the wall within the underlying conglomerate. However, there is very good agreement with the bending moments measured after stage P4.

The magnitudes of lateral wall deflection are very well predicted for the South Wall (Figs. 4a, 5a), with maximum inward deflections up to 10mm at the top of the wall. However, the analysis does not describe the measured rotations of the wall within the glacial till and rock layers. The analysis also gives very reasonable predictions of settlements for the MATEP mat foundation (grouting of surficial soils immediately behind the wall prevented large local settlements reported from the analysis).

Figures 6 and 7 show similar comparisons of predicted and

measured wall deflections for the West and East walls at representative stages of the excavation (after 3rd level prestressing, P3, Fig. 6; and at the end of excavation, L7, Fig. 7). Note that deflections in Figure 7 are shown at a reduced scale. Again the overall magnitudes of wall deflections are well estimated by the FE analyses, while differences in the deflection modes shapes can be explained by simplifications in modeling (such as the representation of the edge beam for the East wall) or uncertainties in the selection of soil parameters. As expected, the predicted surface settlements remain very small (less than 10mm) through the end of the excavation.
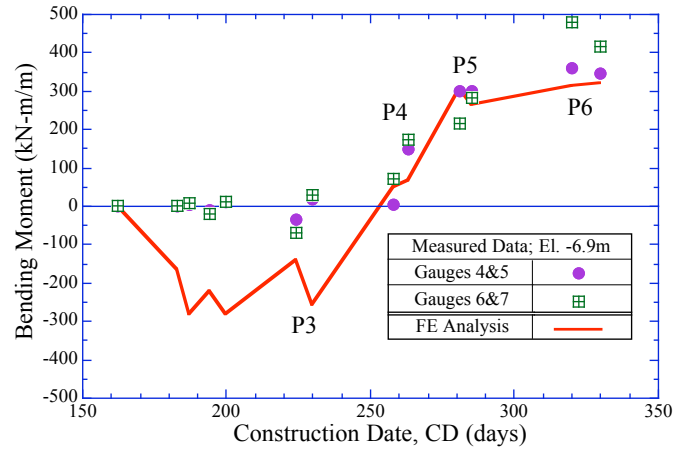

Fig. 9 Comparison of computed and measured bending moments in North wall
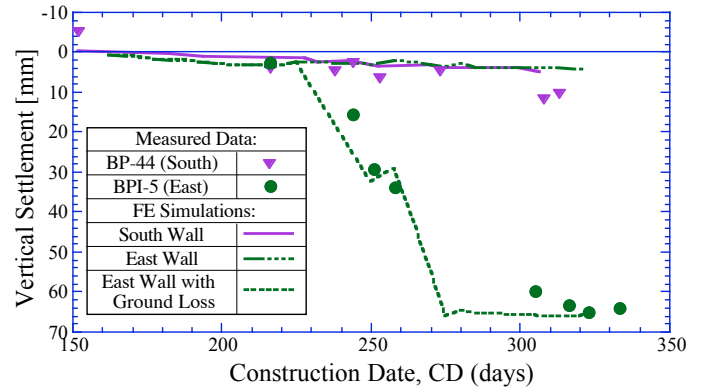

Fig. 10 Effects of ground loss in simulations on surface settlements

Further analyses have been performed to evaluate the possible effects of ground loss (associated with tieback installation) on the predicted wall deflections and ground movements. The analyses are performed by specifying a volumetric strain within particular clusters of finite elements (Brinkgreve, 2002), a technique that has previously been applied to model processes such as compensation grouting around tunnels (Schweiger & Falk, 1998). Calculations for the East wall assume that ground loss occurs within the silt and sand layers (but not in the clay or glacial till) during installation of the 4th and 5th level tiebacks. Figure 10 compares the finite element analyses with the timeline of settlements measured at a reference point BPI-5 on the utility tunnel (Fig. 1). The original analysis is in close agreement with the measured settlements through CD230 (L3 excavation, Fig. 3) and entirely consistent with the performance measured a similar distance behind the South wall (BP-44). By

then assuming a 2% volume strain during anchor installation (prior to stages P4 and P5), the analyses achieve very good matching with the large settlements measured in the utility tunnel (71mm at end of excavation, L7). This simulation is equivalent to a ground loss of 0.52 m^3/lin-m (which can be compared with the 1.5m^3 of grout injected to remediate the one recorded occurrence of ground loss).

Figure 7b shows that the effects of the local ground loss (occurring within 6-7m of the wall) affects the surface settlements up to 20m behind the East wall. The ground loss accounts for an additional 60mm of surface settlement but less than 10mm of incremental wall deflection by the end of excavation (Fig. 7b). A similar procedure has been used to simulate ground losses for the West wall. In this case, smaller volume strains have been included prior to prestressing for P3 through P6 (the results in Fig. 6a assumed 1%, 1%,1.3% and 1.5% following excavation of stages L3 through L6, respectively, amounting to a ground loss of 0.36m^3/lin-m). The computed surface settlement beneath the Redstone Building increases from 8mm to 65mm as a result of these assumed ground losses, producing very good matching with the measured data. The simulated ground losses also produce a net outward deflection of up to 14mm of the wall, leading to much improved agreement with the data measured by inclinometer IN-1.

CONCLUSIONS

Deep excavations for the Dana Farber research tower were supported by a permanent lateral earth support system comprising a perimeter diaphragm wall embedded in the underlying conglomerate bedrock and 4 to 6 levels of prestressed rock anchors. The lateral earth support system was very successful in controlling lateral wall movements to less than ±15mm, however, surface settlements exceeded 65mm – 71mm occurred on two sides of the excavation causing minor damage to adjacent structures. These effects were attributed to local ground losses during anchor installation through overpressured sand and silt deposits.

Back-analyses of the excavation performance using 2-D finite element analyses were able to give consistent estimates of the measured wall deflections on each of the four sides of the excavation. Ground losses have also been simulated in the FE analyses by including local volumetric strains in clusters of soil elements around the tiebacks. These simulations are able to replicate the measured surface settlements with relatively small volume strains, generating additional surface settlements in the range, 50mm – 60mm. The simulated ground losses also cause the perimeter wall to deflect 10mm –15mm from the excavation under the action of the applied anchor pre-stress loads. This result improves the overall agreement between the computed and measured behavior. The results appear to confirm the hypothesis that local ground losses during anchor installation can explain the unexpectedly large surface settlements that occurred on two sides of the excavation.

ACKNOWLEDGEMENTS

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