Prediction and Interpretation of the Performance of a Deep Excavation in Berlin Sand

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Abstract: This paper describes the application of a generalized effective stress soil model, MIT-S1, within a commercial finite-element program, for simulating the performance of the support system for the 20-m-deep excavation of the M1 pit adjacent to the primary station “Hauptbahnhof” in Berlin. The M1 pit was excavated underwater and supported by a perimeter diaphragm wall with a single row of prestressed anchors. Parameters for the soil model were derived from an extensive program of laboratory tests on the local Berlin sands. This calibration process highlights the practical difficulties both in the measurements of critical state soil properties and in the selection of model parameters. The predictions for excavation performance are strongly affected by the vertical profiles of two key state parameters: the initial earth pressure ratio, $K_0$; and the in situ void ratio, $e_0$. These parameters were estimated from field dynamic penetration test data and geological history. The results showed good agreement between computed and measured wall deflections and tieback forces for three instrumented sections. Much larger wall deflections were measured at a fourth section and may be attributable to the spatial variability in sand properties that was not considered in the current analyses. The results of this study highlight the importance of basic state parameter information for the successful application of advanced soil models. DOI: 10.1061/(ASCE)GT.1943-5606.0000518. © 2011 American Society of Civil Engineers.

CE Database subject headings: Excavation; Germany; Constitutive models; Deformation; Finite element method; Diaphragm wall; Instrumentation.

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Introduction

Although finite-element analyses are routinely used in the design of excavation support systems and the interpretation of measured field performance, their predictive accuracy is often quite limited (e.g., Carter et al. 2000). In many cases, the analyses use simplified soil models or measurements from which to calibrate model parameters are unavailable. The geotechnical group at the Massachusetts Institute of Technology (MIT) developed a series of relatively complex elastoplastic soils (MIT-E3: Whittle and Kavvadas 1994; MIT-S1: Pestana and Whittle 1999) and have demonstrated their application for a number of well-instrumented excavation projects in clays (e.g., Whittle et al. 1993; Hashash and Whittle 1996, 2002; Jen 1997). In each of those projects, the numerical analyses were supported by site investigation and laboratory testing programs such that model parameters were well-calibrated and the role of the soil model clearly identified.

This paper follows a similar approach for simulating the performance of a deep excavation in sand. The effective stress-strain-strength properties of the sand were simulated by using the MIT-S1 model that had been integrated within the commercial finite-element program Plaxis. The MIT-S1 model incorporates a void ratio as a separate state variable, in addition to the state of stress, to simulate characteristic transitions from a dilative to a contractive response associated with increases in the formation void ratio and/or confining stress. The model also uses a new framework for describing the compression behavior of soils on the basis of the existence of the limiting compression curve (LCC) (Pestana and Whittle 1995), which provides the means for unifying the behavior of clays and sands within a single constitutive framework.

Model input parameters were calibrated through an extensive program of laboratory compression and triaxial shear tests on specimens of sand (Glasenapp 2002; Becker 2002), which were obtained from an excavation site in central Berlin (Savidis and Rackwitz 2004). The M1 site is one of a series of excavation pits that were used for the underground construction of a new multimodal transportation corridor through the center of Berlin, collectively referred to as the VZB project—Verkehrsanlagen im Zentralen Bereich. Fig. 1 shows that the M1 excavation pit is located to the north of the recently completed Lehrter Bahnhof station (Mönich and Erdmann 1997). The Berlin sands were found to be very different from the sands on which MIT-S1 was initially applied. This paper describes the challenges encountered in an independent model calibration for the Berlin sands and documents calibration approaches different than those initially established by Pestana (1994; Pestana et al. 2002). The model predictions were then compared directly with the field monitoring data, and parametric calculations were performed to understand the factors influencing wall deflections and tieback forces.
The German Society for Geotechnics has used another of the recent excavation projects in Berlin as the basis for a benchmark study to evaluate the accuracy of numerical analyses (Schweiger 2002). In contrast to the current study, the benchmark program provided minimal information on site-specific soil conditions or properties and found a large scatter in numerical predictions according to the selection of constitutive models and stiffness parameters. Indeed, many of the 17 predictors used the same soil models, such as the elastic-perfectly plastic, Mohr-Coulomb or Hardening Soil, integrated in the Plaxis software, but obtained widely varying predictions because of differences in the selection of model parameters. The study highlighted that more refined experimental investigations, including the measurement of stiffness at very small strains, should be employed to provide more reliable data for numerical analysis (Schweiger 2002). This paper offers such a complementary approach, providing a valuable database on Berlin sands and building the numerical model from laboratory measurements of the soil properties.

Site Characterization

The geology of the central area of Berlin is characterized by saturated deposits of quaternary age, reflecting three different glacial periods (Savidis and Rackwitz 2004). The glacial sediments are highly irregular in their horizontal and vertical distributions and also vary widely in their composition, which consists of tills, sands, gravel, and boulder clays. The typical vertical profile at the M1 pit, Fig. 2(b), includes 3–4 m of fill overlying three primary sandy till units: (1) S0, upper Holocene sand, approximately 6 m with a lower 1-m-thick organic soil unit, O; (2) S1 glacial sands from the late Pleistocene period (Weichsel glaciation) that are typically 10 m thick; and (3) S2 glacial sands from the early Pleistocene (Saale glaciation) that are encountered approximately 22 m below the ground surface. The characteristic engineering properties of these sand units have been reported by Arbeitsgemeinschaft Umweltgeologie und Geotechnik (GuD/DMT) (1994) and Borchert and Richter (1994), principally on the basis of empirical correlations from dynamic probing tests [DPH; after Deutsche Forschungsgesellschaft für Bodenmechanik (Degebo) 1993]. These correlations suggest design friction angles, \( \phi' = 31, 34, \) and \( 37.5^\circ \) for the S0, S1, and S2 units, respectively.

The local ground water table is located 2 m below the ground surface. Given the high permeability of the surrounding sandy soils (i.e., in the range \( 10^{-3}–10^{-4} \) m/s), underwater excavation was considered the only practical construction method because dewatering would affect a large area, have a significant environmental impact, produce significant settlements, and potentially cause damage to historical buildings (Savidis and Rackwitz 2004). The M1 excavation pit, shown in Fig. 2(a), is supported by a 1.2–1.5-m-thick, reinforced-concrete diaphragm wall that extends around the perimeter of the site, measuring approximately 300 m long and 25 m wide. The wall panels extend to depths ranging from 25 to 31 m, corresponding to toe embeddings of 6.8–7.8 m below formation level. The wall is supported by a single row of prestressed tieback anchors located 2–3 m below the ground surface with spacing ranging between 1.0 and 1.5 m. These are installed with dip angles ranging from 25–35° and 8 m fixed (i.e., grouted) anchor lengths within the S1 or S2 sand units (free lengths range from 26–40 m). Each tieback typically has eight or nine strands of a high-strength steel (i.e., grade 270) tendon.

After installing the diaphragm wall and tieback anchors, excavation was performed underwater by using a pontoon-mounted...
crane to an average final formation grade 20.2 m below the initial ground surface. Before dewatering, the base of the excavation was sealed by a 1.5-m-thick underwater concrete slab supported by an array of tension piles. The design for the M1 pit used a system of H-piles installed by a vibratory driver and grouted to ensure a good connection with the surrounding sand (RI system). The installation of the RI piles produced significant additional movements of the diaphragm walls. Schran (2003) attributed this behavior, in part, to the presence of light cementation within the deeper S2 sand unit. This study focuses on the performance of the support system during the underwater excavation phase and does not deal directly with the subsequent construction of the anchor piles or base slab. In addition, the current analyses use properties of reconstituted sands and therefore, do not resolve the possible role of cementation on wall movements. The performance of the excavation was monitored through a series of inclinometers installed within the diaphragm walls [Fig. 2(a)] and the load cell measurements of forces in the tieback anchors. The uplift of the base slab was later monitored with horizontal inclinometer tubes [Savidis and Rackwitz 2004; Fig. 2(a)].

**Soil Properties and Model Parameters**

The effective stress-strain-strength properties for the three sandy till units were modeled by using the MIT-S1 model (Pestana and Whittle 1999) under the assumption that all three units had similar material properties but differed principally in their in situ state (i.e., stress conditions, void ratio, and stress history). The MIT-S1 formulation was built on the incrementally linearized theory of rate-independent elastoplasticity (e.g., Prévost 1978) and incorporates void ratio as a separate state variable to describe peak friction angles and dilation rates as functions of the in situ void ratio and effective stress state. The primary features of the model can be summarized as follows:

- Large-strain shearing is controlled by an isotropic, critical state frictional failure criterion.
- Shear behavior is described by a single anisotropic bounding surface that is a function of the effective stresses and current void ratio.
- The density hardening of the bounding surface is controlled by the compression behavior of freshly deposited soils represented by the LCC (Pestana and Whittle 1995), whereas rotational hardening accounts for the evolution of anisotropic properties.
- The small strain nonlinearity in shear and stress-strain responses in unload-reload cycles is described through a perfectly hysteretic formulation.

The MIT-S1 model requires 13 material parameters to characterize the behavior of freshly deposited, uncemented clean sands. Pestana et al. (2002) detailed the selection of these parameters for Toyoura sand, a standard test material whose behavior has been extensively documented in the literature over the last 25 years. For this material, model parameter selection was greatly facilitated by the availability of high-quality laboratory test data including high-pressure consolidation tests and extensive programs of triaxial shear tests.

No comparable test database was available for the Berlin sands; and hence, the authors initiated a laboratory test program on reconstituted test specimens (Glansenapp 2002; Becker 2002). Samples were obtained from the VZB excavation pit M1 and were blended and mixed to obtain an average set of physical properties (Table 1).

Berlin sand is a poorly graded, fine-medium sand with rounded particles, which are associated with fluvio-glacial deposition. When compared with other natural sands of similar particle size, shape, and grading (e.g., Pestana and Whittle 1995) it is apparent that Berlin sand exhibits very low formation void ratios (i.e., $e_{\text{max}} = 0.59$ and $e_{\text{min}} = 0.39$) and has a small range of formation conditions ($\Delta e = 0.20$, Table 1).
Maximum void ratio, \( \varepsilon_{\max} \) tests were also conducted on specimens formed at void ratios associated with the LCC used in the MIT-S1 model. Nineteen triaxial minimum void ratio, \( \varepsilon_{\min} \) tests were performed by Becker (2002), with respective). A subsequent program of 16 triaxial shear tests, and standard drained compression modes (CIUC and CIDC, stresses \( \sigma \)).

The MIT-S1 model assumes that sand specimens compressed from different initial formation densities approach a unique response at high stress levels, referred to as the limiting compression curve. For 1D compression tests, the behavior in the LCC regime is characterized by a linear relationship in the \( \log(e) = -\rho_c \log\left(\frac{\sigma'_c}{\sigma''_c}\right) \) space:

\[
\log(e) = -\rho_c \log\left(\frac{\sigma'_c}{\sigma''_c}\right)
\]

where \( \rho_c \) is slope of the LCC curve; and \( \sigma'_c = \) vertical effective stress at a reference void ratio, \( e = 1.0 \).

Fig. 3 shows data from four 1D compression tests on Berlin sand, each from a different formation void ratio. The data clearly support the LCC concept, with slope \( \rho_c = 0.34 \) and \( \sigma'_c/\rho_a = 25.5 \), where \( \rho_a \) is the atmospheric pressure. The reference pressure for Berlin sand is substantially smaller than expected from empirical correlations on the basis of the mean particle size, \( d_{50} \), and angularity. For \( d_{50} \approx 0.4 \) mm, the data compiled by Pestana and Whittle (1995) show \( \sigma'_c/\rho_a \) increasing from 30 for angular particles (e.g., ground quartz) to 80 for rounded particles (e.g., Ottawa sand). This very interesting observation echoes earlier findings of De Beer (1965) who suggested that Berlin sand is more sensitive to particle splitting than those of other similar sand deposits, such as Mol sand, and attributed this behavior to impurities in the particles.

The MIT-S1 model assumes that the compressibility parameter, \( \rho_c \), is independent of the lateral earth pressure ratio, \( \sigma'_c/\sigma''_c \). However, a fixed spacing exists between the LCC regimes measured in 1D (i.e., \( K_0 \)-LCC) and hydrostatic (i.e., I-LCC) compression. The model (Table 2) actually uses the reference mean effective stress, \( \sigma''_c \), corresponding to hydrostatic compression as an input parameter. Following Pestana and Whittle (1999), this can be obtained from

\[
\frac{\sigma'_c}{\sigma''_c} = \left(1 + 2K_{\text{ONC}}\right)\left[1 + \left(\frac{6}{\alpha^2}\right)\left(1 - K_{\text{ONC}}\right)^2\right]
\]

where \( \alpha^2 = 24\sin^2\phi'/\left(3 - \sin^2\phi'_c\right)^2 \leq 1 \).

No direct measurements of the earth pressure coefficient exist for compression of Berlin sand in the high-pressure LCC regime. Instead, the current analyses assumes \( K_{\text{ONC}} = 0.5 \), which is consistent with empirical correlations (Jaky 1944), assuming a

Table 2. Input Parameters for MIT-S1 Model

<table>
<thead>
<tr>
<th>Parameter/symbol</th>
<th>Physical contribution/meaning</th>
<th>Berlin sand</th>
</tr>
</thead>
<tbody>
<tr>
<td>( \rho_c )</td>
<td>Compressibility of sands at large stresses (LCC regime)</td>
<td>0.34</td>
</tr>
<tr>
<td>( \sigma'_c/\rho_a )</td>
<td>Reference stress at unit void ratio for conditions of hydrostatic compression in the LCC regime</td>
<td>23.5</td>
</tr>
<tr>
<td>( \theta )</td>
<td>First loading curve in the transitional stress regime</td>
<td>0.25</td>
</tr>
<tr>
<td>( h )</td>
<td>Irrecoverable plastic strain, OC(^a)</td>
<td>—</td>
</tr>
<tr>
<td>( K_{\text{ONC}} )</td>
<td>In the LCC regime</td>
<td>0.50</td>
</tr>
<tr>
<td>( \rho_0 )</td>
<td>Poisson’s ratio at load reversal</td>
<td>0.28</td>
</tr>
<tr>
<td>( \omega )</td>
<td>Nonlinear Poisson’s ratio; 1D unloading stress path</td>
<td>1.00</td>
</tr>
<tr>
<td>( \phi'_{cs} )</td>
<td>Critical state friction angle in triaxial compression</td>
<td>31.0(^b)</td>
</tr>
<tr>
<td>( \phi'_{ma} )</td>
<td>Controls the maximum friction angle as a function of 12.5(^b)</td>
<td></td>
</tr>
<tr>
<td>( p )</td>
<td>formation density at low effective stresses</td>
<td>2.7</td>
</tr>
<tr>
<td>( m )</td>
<td>Controls the cap geometry of the bounding surface</td>
<td>0.42</td>
</tr>
<tr>
<td>( \varepsilon_i )</td>
<td>Small strain (&lt; 0.1%) nonlinearity in shear</td>
<td>4.0</td>
</tr>
<tr>
<td>( \psi )</td>
<td>Rate of evolution of anisotropy; stress-strain curves</td>
<td>10</td>
</tr>
<tr>
<td>( C_b )</td>
<td>Small strain stiffness at load reversal</td>
<td>950</td>
</tr>
</tbody>
</table>

\(^a\)Parameter not needed in current study.

The test program performed by Glasenapp (2002) included a series of four one-dimensional (1D) consolidation tests that were carried to high confining stresses to identify the parameters associated with the LCC used in the MIT-S1 model. Nineteen triaxial tests were also conducted on specimens formed at void ratios \( \varepsilon_0 = 0.43–0.60 \) that were hydrostatically consolidated to effective stresses \( \sigma'_c = 100, 500, \) and 800 kPa and sheared in both undrained and standard drained compression modes (CIUC and CIDC, respectively). A subsequent program of 16 triaxial shear tests, performed by Becker (2002), with \( \varepsilon_0 = 0.43–0.57, \) \( \sigma'_c = 100, 800 \) kPa, used more refined testing procedures including reduced-friction end-platen and local strain measurements to enable the more reliable interpretation of large-strain, critical state conditions and nonlinear stiffness properties at small shear strains.

Table 2 summarizes the input parameters used by the MIT-S1 model together with their physical meaning and the values ultimately selected for Berlin sand. The following section provide additional details about the parameter selection.

**Compression Behavior**

The MIT-S1 model assumes that sand specimens compressed from different initial formation densities approach a unique response at...
friction angle $\phi_o' = 31^\circ$ for shear strength at high confining pressures (Table 2). Substituting into Eq. (2), $\sigma'_f/\sigma_{cr}' \approx 0.92$ and $\sigma'_f/p_a = 23.5$.

The MIT-S1 model introduces a parameter, $\theta$, to describe the progressive breakage of particles as specimens are compressed. Larger values of $\theta$ cause a more gradual transition to the LCC regime; whereas low values of $\theta$ represent materials with well-defined yield points associated with particle breakage (typically observed in tests on very uniform materials such as glass ballast). Fig. 3 shows that the measured compression behavior of Berlin sand is well represented by $\theta = 0.25$. This is consistent with the expected behavior from empirical correlations between $\theta$ and the uniformity coefficient; $\theta \approx 0.1 C_u \approx 0.3$ for rounded particles, presented by Pestana and Whittle (1995, 1999).

**Small Strain Stiffness Properties**

The model parameters $\mu_0'$ and $C_b$ define the elastic Poisson’s ratio and bulk modulus that control the stiffness of sand immediately upon load reversal (Pestana 1994):

$$C_b = \frac{K_{\text{max}}}{p_a} \left( \frac{e}{1+e} \right) \left( \frac{\sigma'}{\epsilon'} \right)^{-1/3}$$

where $e =$ void ratio; $\sigma'$ = mean effective stress; and $K_{\text{max}} =$ small strain elastic bulk modulus. The small strain elastic shear modulus, $G_{\text{max}}$, can then be derived from $C_b$ and $\mu_0'$:

$$2G_{\text{max}} = \frac{3(1-2\mu_0')}{1+\mu_0'}$$

The model parameters $C_b$ and $\mu_0'$ were derived from local strain measurements in the triaxial shear tests performed by Becker (2002).

The tangential elastic moduli and, hence, Poisson’s ratio during unloading, are updated as a function of stress variations and a parameter, $\omega$. This parameter captures the nonlinearity in the effective stress paths during unloading ($\omega = 0$ would yield a linear relationship between $K_b$ and OCR). In principle, $\omega$ can be interpreted from the unloading effective stress path in a rigid-walled, 1D compression device, requiring very precise measurements of lateral stresses, or through very accurate small strain measurements in both vertical and radial directions during unloading in a triaxial cell. No such measurements were recorded for Berlin sand, and instead, $\omega = 1.0$ was selected on the basis of the recommendations of Pestana from typical data reported in the literature (Pestana et al. 2005). This value yields nonlinearity in the effective stress path even for OCR $\approx 1.5$. Nonlinear behavior at relatively small shear strain levels (i.e., less than 0.1%) is controlled by a second parameter $\omega_1$ that, in principle, can be fitted to local strain data on modulus degradation.

**Shear Behavior**

In the previous formulation of MIT-S1 for Toyoura sand, Pestana et al. (2002) tried to develop procedures that could provide the unambiguous estimate of the remaining six model input parameters as follows: (1) the large strain friction angle, $\phi_o'$, measured in either drained or undrained shear tests; (2) the peak friction angle measured in drained shear tests on dense specimens to enable the selection of parameters $\phi_o'$ and $p$ (Table 2); (3) the effective stress paths in undrained shearing to enable the selection of parameters $m$ and $\psi_s$; and (4) the small strain nonlinear stiffness used to define $\omega$. This approach also minimizes the need to measure critical state conditions in the laboratory tests.

For Berlin sand, it has proven difficult to follow such a simple procedure because of the uncertainties in the critical state and the variability in the peak friction angles, as shown in Figs. 4–6.

Fig. 4 shows typical drained shear tests on Berlin sand at three different confining pressures and formation void ratio, $e_0 = 0.51$. As expected, the measured peak friction angle decreases with the level of confining pressure, and the three tests converge to a unique stress ratio at large shear strains, corresponding to a friction angle of approximately $31^\circ$. This angle is assumed to be the critical state friction angle in the MIT-S1 model ($\phi_o'$, Table 2), although it is not clear from the volumetric strain data if the samples have actually
achieved critical state conditions (i.e., a zero rate of volumetric strain) at the end of each test with shear strains exceeding 20%.

A relatively small range in peak friction angles was measured in the CIDC shear tests (Fig. 5) (i.e., $\phi_{\text{peak}}^i = 32^\circ$–$40^\circ$), and significant variability, up to $2^\circ$, among tests performed under nominally identical formation conditions. The peak friction angles are lower than would be expected for other quartzitic sands, at a similar range of confining pressures (i.e., $100$ kPa) but was in good agreement with data for $\sigma_s' = 500$, 800 kPa. The computed critical state line had a critical void ratio $e_{\text{crit}} = 0.6$ (i.e., $e_{\text{crit}} \approx e_{\text{max}}$) at a low effective stress (Fig. 6). According to Ishihara (1993), sands with a state index $I = [\psi_{\text{crit}} - e]/(e_{\text{crit}} - e_0)] < 0$ will collapse during undrained shearing with zero residual strength.

The final model input parameter, $\psi$, controls the rotational hardening of the yield surface in MIT-S1 and, hence, characterizes the evolution of anisotropic deformation and strength properties. In previous studies, $\psi$ has been calibrated from the stress-strain response measured during undrained shearing to large strains. For example, Fig. 7 illustrates the selection of $\psi$ for one undrained shear test. The parameter has minimal effect on the predicted response until the mobilized friction exceeds $\phi_i'$. For Berlin sand, a strong cross-coupled effect of $\psi$ with $\omega_1$ exists, which was not previously found for Toyoura sand. Fig. 8 shows that $\omega_1$ has a very similar effect as $\psi$ on the undrained stress-strain response at large shear strains and also influences the initial effective stress path.

Fig. 9 compares the model predictions for $\psi = 10$ and 25 with the measured shear stress-strain behavior from CIDC tests on Berlin sand consolidated to $\sigma_s' = 800$ kPa from different formation void ratios. Although the results show that the model tends to underestimate the initial shear stiffness and peak shear resistance of the densest specimens (i.e., $e_0 = 0.462, 0.491$), the general trends in behavior were well described by the model with $\psi = 10$ and other input parameters listed in Table 2.

**Initial Soil State Parameters**

To apply MIT-S1 for simulations of excavation performance for the M1 pit, it is first necessary to establish the ranges of two key state variables, $e_0$, the in situ void ratio; and $K_o$, the lateral earth pressure coefficient at rest. No direct measurements of these parameters exist. The only in situ data are from dynamic probing (DPH) tests performed in conjunction with the boreholes shown in Fig. 2(a). The DPH $N_{10}$ blowcount data can be correlated with relative density, $D_r$:

$$D_r = 0.23 + 0.38 \log(N_{10})$$  

Eq. (4) follows DIN 4094-3 (2002) and uses the laboratory values of $e_{\text{max}}$ and $e_{\text{min}}$ for Berlin sands (Table 1). Fig. 10 summarizes the resulting profiles of the estimated void ratio from four typical locations around the M1 pit [Fig. 2(a)]. Although the results do show a trend of increased density with depth, considerable scatter is present in the estimated void ratio at any selected depth. The data suggest that the upper sand unit, S0, is in a loose state with $e_0 \approx 0.6$ (i.e., the upper 8 m), whereas the lowest unit, S2, is very dense with $e_0 \approx 0.3$–0.4; the intermediate unit, S1, has $e_0 \approx 0.5$–0.6. The authors have not found any clear spatial pattern in the data and, hence, assume the same void ratio profile in the analyses at each of the instrumented sections.

The in situ $K_o$ values should be strongly influenced by the geological history. In principle, the heavily precompressed Pleistocene units (i.e., S1 and S2) can be expected to have higher values of $K_o$ than the recent Holocene unit, S0. On the basis of this reasoning, and in the absence of any direct measurements, the authors have assumed default values $K_o = 0.5$ and 1.0 for the S0 and S1/S2 units, respectively.

An alternative method for estimating the void ratio is through the empirical correlations used for the mobilized friction angles in each of the three sand units. According to GuD/DMT (1994), $\phi_i' = 31$, 34, and 37.5° for the S0, S1, and S2 units, respectively. Assuming that these friction angles are to be correctly represented by the MIT-S1 model, a consistent set of in situ void ratios can be
obtained from the model predictions relating the peak friction to the void ratio and effective confining stress (cf. Fig. 5). This procedure is illustrated in Fig. 11. The soil profile is approximated by the three sand units, ignoring secondary details such as the fill and organic layers, as shown in Fig. 11(a). For the upper S0 sand, \( \phi = \phi' \) and, hence, \( e_0 \geq 0.6 \). For S1, the in situ stress ranges from 135 to 265 kPa and, hence, \( e_0 = 0.51-0.53 \), consistent with \( \phi' = 34^\circ \). By a similar procedure, \( e_0 = 0.40-0.45 \) in S2. These results suggest higher in situ void ratios than those derived directly from DPH correlations (Fig. 10).

The MIT-S1 model simulates nonlinear stress-strain behavior from small levels of shear strain. Fig. 12 illustrates the profile of the small strain shear modulus, \( G_{\text{max}} \), computed for the M1 site on the basis of laboratory stiffness parameters and the assumed profiles for \( K_0 \) and \( e_0 \) [Eqs. (3a) and (3b); Table 2]. These results are in very good agreement with well-known empirical correlations for \( G_{\text{max}} \) of sands such as those proposed by Hardin and Richart (1963), which are also included in the recommendations of the German Society for Geotechnical Engineering (DGGT). In principle, these results should match closely the values of \( G_{\text{max}} \) from
measurements of the cross-hole wave velocity, $v_s$, reported in the preliminary site investigation work by GuD/DMT (1994). However, Fig. 12 shows that the cross-hole $G_{\text{max}}$ data are much lower than expected. Indeed, the cross-hole values of $G_{\text{max}}$ were actually lower than empirical estimates of the “reload modulus” used in the original wall design methods for the VZB pits. The source of this discrepancy is not known but the small strain stiffness used by MIT-S1 is higher than the modulus values from the previous empirical correlations in Berlin.

### Numerical Model for M1 Excavation

The two-dimensional finite-element analyses of the M1 excavation pit were performed by using the commercial finite-element software, Plaxis. The MIT-S1 model was integrated within this code through a “user-defined” constitutive model interface. The analyses focus on four half-sections through the excavation pit [all similar to Fig. 2(b)] that correspond to the locations of inclinometers MQ2–MQ5 [Fig. 2(a)]. The characteristics of the cross sections are summarized in Table 3. The ground surface on the west side of the M1 pit is 1.5 m lower than on the east side, and the excavation progressed northwards. The final depth in MQ2 was reached more than a month after MQ4. The soil is represented by six-node plane strain elements, the tiebacks by using a combination of node-to-node anchor and “geotextile” elements for the free and fixed anchor lengths, respectively, and the diaphragm wall by using elastic Mindlin-beam elements. The analyses assumed that the wall was “wished-in-place” and hence, did not consider local changes in stresses or soil properties associated with trench excavation and concreting. The analyses simulated the initial excavation to 30.5 m, followed by tieback installation and prestress, then by four stages of underwater excavation to the final formation level (no quantitative data about the underwater excavation stages were available).

### Results

Initial parametric analyses were performed assuming a uniform soil profile (single sand unit) at a reference section, MQ3, to investigate the effects of the in situ state parameters $e_0$ and $K_0$. Fig. 13 summarizes the measured wall deflections and tieback loads immediately after prestress and at the final formation stage. The measured data were compared with finite-element simulations for a constant void ratio (i.e., $e_0 = 0.5$) and three possible values of $K_0 = 0.5$, 0.75, and 1.0. The results showed that higher $K_0$ values generated larger wall deflections and anchor loads at the end of excavation. The measured data were within the midrange of the computed maximum wall deflections (i.e., 1.5–2.8 cm) whereas the tieback force was in close agreement with results for $K_0 = 1.0$. However, the analyses generally underestimated the wall pull-back upon the initial application of the prestress and overestimated deflections at the top of the wall during excavation.

Fig. 14 shows a further set of calculations for a constant $K_0 = 0.5$ and three possible values of $e_0 = 0.4, 0.5$, and 0.6. The in situ void ratio had a minimal effect on the wall deflections at prestress or on the values of the tieback force at the end of excavation. However, wall deflections during excavation were very strongly influenced by $e_0$. The maximum wall deflection increased from 1.0 to 4.0 cm as $e_0$ increased from 0.4 to 0.6. The movements at the top and toe of the wall were little affected by changes in $e_0$ between 0.4 and 0.5 and were generally in close agreement with the measured data.

The parametric analyses highlighted the need to subdivide the vertical profile and corroborate the variation of state variables.
discussed previously. A third set of analyses for MQ3, Fig. 15, considered a more realistic profile represented by three sand units with $e_0 = 0.60, 0.53,$ and $0.40$ and $K_0 = 0.5, 1.0,$ and $1.0$ in $S_0,$ $S_1,$ and $S_2,$ respectively. The overall pattern of predictions was much improved for this case. The numerical analyses were in excellent agreement with the movements at the top and toe of the wall at the final formation grade but underestimated the maximum wall deflection by 0.5 cm. The bending of the wall was much better described than in either of the two preceding sets of analyses with homogeneous state variables. The model predicted very small surface settlements (i.e., up to 0.2 cm) in the retained soil, and 1.5 cm of heave below the base of excavation. Unfortunately no data was available to evaluate those results.

Figs. 16–18 summarize further computations and measurements for three independent cross sections [i.e., MQ5, MQ4, and MQ2, respectively, cf. Fig. 2(a)]. Table 3 summarizes the differences in the support systems and the excavation depths for each of these sections.

Section MQ5, Fig. 16, is immediately opposite MQ3 but is supported with a thinner diaphragm wall section (i.e., 1.2 versus 1.5 m), less steeply inclined anchor (i.e., 25 versus 35° dip angle) and lower prestress load. The measured data showed slightly higher maximum wall deflections (i.e., 2.7 versus 2.1 cm at MQ3) and movements at the top of the wall (i.e., 1.1 versus ~0.2 cm at MQ3) that were consistent with these differences in support conditions. The measured toe movements were almost the same at both MQ3 and MQ5 (i.e., 0.3 cm).

Table 3. Properties of Excavation Support Structures at Inclinometer Locations

<table>
<thead>
<tr>
<th>Location</th>
<th>Excavation depth (m)</th>
<th>Thickness (m)</th>
<th>Height (m)</th>
<th>Spacing (m)</th>
<th>Free length (m)</th>
<th>Dip angle (°)</th>
<th>Number of tendons</th>
<th>$A_s$ (cm²/m)</th>
<th>Prestress (kN/m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>MQ2</td>
<td>23.30</td>
<td>1.5</td>
<td>31.05</td>
<td>1.0</td>
<td>40.0</td>
<td>38</td>
<td>10</td>
<td>18.15</td>
<td>400</td>
</tr>
<tr>
<td>MQ3</td>
<td>21.40</td>
<td>1.5</td>
<td>28.70</td>
<td>1.0</td>
<td>34.5</td>
<td>35</td>
<td>8</td>
<td>14.52</td>
<td>540</td>
</tr>
<tr>
<td>MQ4</td>
<td>18.00</td>
<td>1.2</td>
<td>24.80</td>
<td>1.5</td>
<td>30.5</td>
<td>35</td>
<td>8</td>
<td>9.68</td>
<td>213</td>
</tr>
<tr>
<td>MQ5</td>
<td>20.10</td>
<td>1.2</td>
<td>27.20</td>
<td>1.2</td>
<td>26.5</td>
<td>25</td>
<td>9</td>
<td>13.61</td>
<td>292</td>
</tr>
</tbody>
</table>

Note: Wall: elastic properties, $E = 30$ GPa, $\nu = 0.15,$ $\gamma_c = 24$ kN/m³; tiebacks: head at elevation +31 m, fixed anchor length, $L_a = 8.0$ m, 15.2-mm-diameter steel tendons, modulus, $E = 210$ GPa.
For this section, the numerical predictions were in excellent agreement with the measured top, toe, and maximum wall deflections and anchor forces at the final formation level. The analyses also predicted much larger settlements at MQ5 (i.e., 1.2 versus 0.2 cm for MQ3) because of differences in anchor location and prestress, whereas predictions of heave inside the excavation were almost the same for both MQ5 and MQ3.

Section MQ4, Fig. 17, used the same diaphragm wall section as MQ5 but had a shallower dip of the tieback anchors and was designed with lower anchor stiffness and prestress (Table 3).
The excavation was also 1.8 m shallower at MQ4. Numerical predictions for MQ4 were consistent with expected behavior on the basis of these perturbations of support conditions. The computed maximum wall deflection (i.e., 1.9 cm) was smaller than that found at MQ5 (i.e., 2.5 cm), whereas computed movements at the top of the wall were larger (i.e., 2.0 versus 1.2 cm for MQ5). Although excellent agreement existed between the computed and measured top-of-wall deflection and anchor load, the numerical analysis significantly underestimated the measured toe movement (i.e., 0.2 versus 0.9 cm) and, hence, underestimated the measured maximum wall deflection (i.e., 2.6 cm). These discrepancies are not easily explained from the results at the previous sections of MQ5 or MQ3. Although the borehole data do indicate a thicker zone of organic materials in the vicinity of MQ4, no evidence suggests...

Fig. 15. Predicted excavation performance for MQ3 from best estimate of state parameters

Fig. 16. Predicted excavation performance for MQ5
high void ratios in the sands from DPH soundings at B1129 [cf. Figs. 2(a) and 10]. However, construction problems were associated with the diaphragm wall panel installation in this area (by using a hydrofraise), and it is possible that this may be associated with the local loosening of the soil at the toe of the wall.

Finally, results for MQ2 in Fig. 18 are most directly comparable to the conditions at the reference section MQ3 (Fig. 15). These two used the same diaphragm wall section (ie., 1.5 m) and had similar anchor inclinations (38 versus 35° for MQ3), but the excavation was almost 2 m deeper at MQ2. The wall deflection data from MQ2 differed significantly from any of the three preceding sections. It was the only section in which significant inward wall deflections were measured at the prestress stage, up to 0.7 cm at the middepth of the wall. At the end of excavation, the maximum measured wall deflection was approximately 5.2 versus 2.1 cm at MQ3. This difference in measured performance was certainly
Discussion

The preceding numerical analyses have shown that it is possible to obtain reasonable predictions of wall deflections and tieback forces by using a constitutive model that is calibrated to the results of laboratory tests on reconstituted sand specimens. The MIT-S1 model was able to describe variations realistically in the shear strength and stiffness parameters measured at different confining stresses and void ratios by using a single set of input parameters. However, further judgment was needed in the selection of in situ state variables $e_0$ and $K_0$.

It is certainly plausible to achieve comparable agreement between computed and measured behavior by using simpler constitutive soil models. However, the difficulty lies in the rational selection of input parameters. None of the “simple models” used in current practice can describe the full range of stiffness and shear strength properties measured in the laboratory tests on Berlin sand. Instead, it is more effective to consider optimizing the selection of key input parameters for these models and then to compare the optimized parameters with the results of the laboratory tests. For example, the authors optimized the selection of shear strength and stiffness parameters for the Hardening Soil Model (Schanz et al. 1999) within the Plaxis software at section MQ3. This was accomplished by using genetic algorithms similar to those described by Levasseur et al. (2008) and by optimizing the selection of two model input parameters, $E^\text{ref}_{50}$ and $\phi'_\text{peak}$. The objective function was set to minimize the differences in the computed and measured tieback forces and lateral wall deflections, over the full depth of the wall, at the preload stage and at the end of excavation. Table 4 summarizes the input for the Hardening model, including the selected range for the optimizing parameters. Fig. 19 plots the predictions obtained by the best-fit Hardening Soil Model parameters at MQ3. Good agreement exists between the computed and measured maximum wall deflections at the end of excavation. However, the model overestimated the inward movements at the toe of the wall and,

Table 4. Input Parameters Used in Generic Algorithms for Hardening Soil Model

<table>
<thead>
<tr>
<th>$E^\text{ref}_{50}$ (MPa)</th>
<th>$E^\text{ref}_{50}$ (MPa)</th>
<th>$E^\text{ref}_{50}$ (MPa)</th>
<th>$\nu_{ur}$</th>
<th>$m$</th>
<th>$\rho_{ref}$ (kPa)</th>
<th>$\phi'_\text{peak}$ (°)</th>
<th>$\psi_d'$ (°)</th>
</tr>
</thead>
<tbody>
<tr>
<td>[20–200]</td>
<td>$bE^\text{ref}_{50}$</td>
<td>$3E^\text{ref}_{50}$</td>
<td>0.25</td>
<td>0.5</td>
<td>100</td>
<td>[30–40]</td>
<td>Calculated</td>
</tr>
</tbody>
</table>
| Note: In Plaxis, the following relationships are used to calculate $E_{50}$ and the dilation angle $\psi_d'$:

$$E_{50} = E^\text{ref}_{50} \left( \frac{\sigma_0^c}{\rho_{ref}} \right)^m$$

$$\sin \psi_d' = \frac{(\sin \phi'_\text{peak} - \sin \phi_{cv}')}{(1 - \sin \phi'_\text{peak} \sin \phi_{cv}')}$$

where $\phi_{cv}' = 31°$; $b$, and $K_0$ vary with depth and are shown in Fig. 19.

Fig. 19. Predicted excavation performance for MQ3 by using the Hardening Soil Model; input was optimized by using genetic algorithms (Table 4)
compared with the MIT-S1 predictions, yielded larger deformations below the base of excavation. Moreover, it predicted heave behind the wall, an improbable response for the retained soil. The benchmark study on a similar Berlin excavation (Schweiger 2002) also reported heave predictions, illustrating the inadequacy of some of the models used and the lack of calibration data for the Berlin sands. The backfitted value for the peak angle, $\phi'_{\text{peak}} = 36.6^\circ$, was in reasonable agreement with the friction angle measured at $\sigma'_{1} = 100$ kPa (test #570, Fig. 4). The predicted dilation angle, $\psi_d = 6.8^\circ$ was at the upper limit of dilation angles measured in the laboratory triaxial tests (cf., #570, Fig. 4). However the elastic moduli, $E = 50$–140 MPa for $\sigma'_{1} = 100$–800 kPa were significantly lower than the stiffness values measured in the corresponding test (i.e., $E_{50} \approx 140$–245 MPa, respectively). These results suggest the need for further refinement in the selection of the Hardening Soil Model parameters for the lower sand unit S2 but give no insight into the broader applicability of the laboratory test results.

## Conclusions

This paper described the application of a generalized effective stress soil model, MIT-S1, for predicting the performance of deep excavations in Berlin sand. The model was calibrated by using data from an extensive laboratory program of tests on reconstituted sand specimens. The calibration process proved quite challenging because of the variability in the peak friction angles with small perturbations in the formation void ratio and uncertainties in the interpretation of critical state conditions.

The model was used in finite-element simulations of the underwater excavations at a series of instrumented sections in the M1 pit near the Lehrter Bahnhof in central Berlin. Site investigations for this project showed that the vertical profile included three primary sand units, whereas in situ density and shear strength were estimated by using correlations to DPH $N_{10}$ blowcount data. The measured data showed significant variability in the estimated in situ void ratio. The current study assumed a single average profile and used the DPH correlations and background information on the geological history to estimate the in situ void ratio, $e_0$, and earth pressure coefficients, $K_o$.

The numerical simulations were in very good agreement with measured diaphragm wall deflections and forces in the single row of tieback anchors for three of the four instrumented sections considered in this study. The measured data for a fourth section (i.e., MQ2) showed much larger wall movements than predicted or expected on the basis of the design of the lateral earth support system, whereas unusual wall-toe movements occurred at a second section (i.e., MQ4). Although these deviations in behavior were mostly probably caused by the spatial variations in soil properties, no supporting evidence exists from the local DPH data.

The study showed that realistic predictions of excavation performance can be achieved through the careful site-specific calibration of sand behavior and by using a constitutive model capable of representing variations in stress-strain-strength properties as functions of the confining stress and void ratio. This approach provides a more consistent method of model validation than generic benchmark studies that use aggregated soil properties. However, further work is needed to address the effects of spatial variability in site-specific applications.

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