FIELD INSTRUMENTATION AND NUMERICAL ANALYSIS OF AN ELLIPTICAL SHAFT CONSTRUCTED FOR THE TEXAS SUPERCONDUCTING SUPER COLLIDER PROJECT

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ABSTRACT

This paper focuses on the field data interpretation and numerical analysis of an elliptical shaft constructed for the Texas Superconducting Super Collider Project. The instrumentation program consisted of measurements of lateral deformation, heave and pore pressure within the undoweded and predoweded reaches of the advancing shaft. The field measurements point to a blocky, post-peak response of the clay shale beneath the advancing shaft invert and indicate the importance of predoweling reinforcement in controlling progression of the blocky failure. The numerical analysis was performed based on an understanding of the rock mass response to excavation developed through analysis of field instrumentation data. The technique developed for simulating the impact of the blocky, post-peak response of the shale on the redistribution of stresses around the shaft opening was successful in describing the pore pressure response recorded beneath the shaft invert and has proven to be a useful tool in studying the redistribution of stresses in blocky materials.

Key words: deformation, field monitoring, numerical analysis, pore water pressures, shaft, soft rock (IGC: C7/G2)
PROJECT DESCRIPTION

The Texas Superconducting Super Collider (SSC) Project

The SSC, near Waxahachie, Texas (Fig. 1), was to be the world’s largest and most powerful particle accelerator. Atomic physicists, forever in search of an understanding of subatomic particles and of how gravity and magnetism work, conceptualized a particle accelerator system that could collide counter-rotating beams of protons at an energy of 40 trillion electron volts, TeV (20 times greater than possible at then existing facilities). Conventional construction facilities for the SSC (Fig. 2) were to consist of:

(a) An 87 km oval, main collider tunnel;
(b) Two large laboratory campus complexes, located on the east and west sides of the main collider tunnel;
(c) Approximately 15.4 km of booster tunnel (HEB, MEB, LEB and Lina);
(d) Four large underground experimental detector halls; and
(e) Twenty shaft service areas housing over sixty vertical shafts (N15 to N55 and S15 to S55).

In all, over 5.5 million m$^3$ of rock was to be excavated, approximately 1 million m$^3$ of concrete and shotcrete was to be placed, over 50 km of rock bolts were to be installed and over $10^8$ kg of structural and reinforcing steel were to be used (Clark, 1993).

However, on October 21, 1993, the United States Congress voted to suspend all then current and planned construction contracts associated with the SSC. At closeout, the status of the design and construction effort was as follows (Gilbert, 1993):

(a) Engineering design was approximately 70 percent complete;
(b) Construction of conventional facilities was 17 percent complete;
(c) 23.5 km of tunnel had been excavated; and
(d) 18 shafts had been completed.
**N15-Magnet Delivery Shaft**

The N15-MDS (Fig. 3) was one of the 18 shafts completed at the time of project closeout. Located over the main collider tunnel on the western edge of the 87 km collider ring, its purpose was twofold: (i) the shaft and starter tunnel were to serve as a staging area for future tunnel contracts; and (ii) the shaft was to provide access for lowering magnets into the collider tunnel following completion of construction.

The N15-MDS is an elliptical shaft, approximately 9 m x 18 m in cross-section from the ground surface to a depth of 58 m; changing to an ellipsoidal shaft from that point to the bottom of the shaft at a depth of 73 m (Fig. 4). Starter and tail tunnels for tunnel boring machine staging and assembly tie into the shaft in the northwest and southwest “corners” of the ellipse.

**Local Geology**

Boring log data obtained during the geotechnical investigation for the shaft indicates that residual clay soil with weathered limestone fragments extends from a depth of 0 to 0.3 m. The residual clay is underlain by soft to medium tan weathered limestone of the Austin Chalk formation to a depth of 3.6 m. A sharp contact to the fresh, medium to moderately hard, gray Austin Chalk is present at 3.6 m, and the fresh limestone extends to a depth of 60.6 m (see Fig. 4).

The Austin Chalk is primarily a light to medium gray, low-strength limestone with interbeds of calcareous shale. The calcium carbonate content is very high and averages approximately 85 percent. The last approxi-

![Fig. 4. Cross-section of the N15-magnet delivery shaft](image)
liner key at a depth of 58 m. The spacing of dowels was 1.2 × 1.2 m. The predoweling scheme was developed to control deformations beneath the shaft invert, and to help maintain the integrity of the shale until the final 3 m thick concrete shaft invert was completed.

GEOTECHNICAL INSTRUMENTATION

*Instrumentation Program*

The most significant aspects of the geotechnical instrumentation program included the measurement of horizontal and vertical deformations and pore pressure within the undowed and predoweded reaches of the advancing shaft. Figures 5 and 6 show the instrument locations at the N15-MDS site.

Instruments installed from the ground surface prior to shaft excavation included four vibrating wire piezometers (designated P1, P2, P3 and P4), two pneumatic piezometers (designated PP1 and PP4) and four inclinometers (designated MDS-1 to MDS-4). The piezometers were spaced vertically to ensure an accurate understanding of the groundwater response to the shaft excavation as shown in Fig. 6. This included one piezometer (i.e., P2) in the Austin Chalk above the regionally traceable 0.2 to 0.3 m thick "Bentonite Marker Bed", which is nominally 21 m above the Austin Chalk/Eagle Ford Shale contact, and two piezometers (i.e., P1 and PP1) in the Austin Chalk below the bentonite but just above the Eagle Ford Shale contact. The three remaining piezometers installed from the ground surface (i.e., P3, P4 and PP4) were spaced vertically within the Eagle Ford Shale, just outside the perimeter of the ellipsoidally shaped shaft bottom. P3 was installed at an elevation of 164.3 m and P4 and PP4 were paired and installed at an elevation of 149.1 m. The paired vibrating wire and pneumatic piezometers were used to provide redundancy in the event of gauge failure.

The instruments shown inside the shaft limits (i.e., piezometers P5-P8 and heave gauges HG1-HG2) were installed from within the shaft at the base of the shaft's lower liner key. When shaft construction had progressed to a depth of 58 m (approximately 3 m above the Austin Chalk/Eagle Ford Shale contact) excavation was suspended for a period of 19 days in order to finish construction of the shaft's lower liner key. From this point, the contractor was required to predowel the shale beneath the future final shaft invert (from the final invert elevation to approximately 15 m below the final invert) and to drill holes for installation of the two heave gauges (i.e., HG-1 and HG-2) and the four additional vibrating wire piezometers (i.e., P5, P6, P7 and P8). These instruments were to be used to measure the heave and pore pressure response of the shale, directly beneath the shaft, as the excavation progressed. The locations and depths of the instrumentation described above and of the convergence anchors and multiple position borehole extensometers (MPBXS) that were also installed are presented in the profile of the shaft instrumentation in Fig. 6.

The following presents the record of deformation and pore pressure response observed in the Austin Chalk and Eagle Ford Shale during shaft construction.
Deformation Measurements

During shaft excavation in the Austin Chalk, only small displacements around the large elliptical shaft opening were recorded as the excavation progressed. Inclinometers indicated that the maximum amount of deformation into the shaft in the Austin Chalk was approximately 3 mm and was recorded in inclinometer MDS-2B just above the Austin Chalk/Eagle Ford Shale contact. Convergence gauge anchors confirmed the movement recorded with the inclinometers, and indicated that deformations on the order of 1 to 3 mm were occurring in the Austin Chalk around the perimeter of the shaft.

In the Eagle Ford Shale, the inclinometer casings installed from the ground surface (Fig. 7) and the heave gauge casings installed from the base of the lower liner key (Fig. 8) provided the most complete record of deformations. The inclinometers were read for the last time prior to completing construction of the shaft’s lower liner key, on May 29, 1992 (i.e., 74 days following the start of the excavation). The readings revealed no measurable horizontal deformation in the Eagle Ford Shale at that time.

On June 13, 1992 (i.e., 89 days following the start of excavation) the lower liner key was completed and shaft excavation recommenced. By day 91, the shaft had progressed to a depth of 59.7 m (225 ft), approximately 1.9 m below the lower liner key, but still 0.9 m above the Austin Chalk/Eagle Ford Shale contact. Inclinometer readings and heave gauge readings taken at that depth revealed that the shaft had begun to deform and respond to the approaching shaft bottom.

The inclinometer readings indicated an abrupt increase in horizontal deformation at a depth of approximately 60.6 m, corresponding with the Austin Chalk/Eagle Ford Shale contact. Increased horizontal deformation continued to a depth of approximately 70.1 m, or approximately 10.4 m ahead of the shaft bottom at a depth of 59.7 m. The heave gauge readings as shown by solid squares in Fig. 8 confirmed this response with a maximum vertical displacement of 2.2 cm recorded within the undoweled shale at HG-1. The extent of vertical deformation was considerable, with movement occurring to a depth of 87.8 m, or just 0.6 m above the bottom of the predefined zone.

The inclinometers suggested that the magnitude and depth of horizontal deformations continued to increase as the excavation progressed. The increase revealed a “stepped” pattern of horizontal deformation, indicative of deformations occurring along bedding planes or other discrete planes of weakness within the shale (see Fig. 7). This deformation mechanism was observed during the reading of the heave gauge instruments as well. Visual field observations revealed horizontal offsets, in the heave gauge casings, of up to 2.5 cm within 3.0 m of the advancing shaft bottom. In two cases, these offsets prevented the instruments from being read. HG-2 could not be read on day 99 due to bending of the heave gauge casing north (i.e., into the middle of the shaft) by approximately 2.5 cm at a depth of 2.4 m below the shaft bottom at a depth of 59.7 m (Shannon and Wilson, 1993); and a final reading of HG-1 was not possible due to a 2.5 cm horizontal offset in the casing at approximately 1.5 m below the final shaft invert.

Further analysis of the heave gauge data revealed that as the excavation progressed, approximately 3.5 to 4.7 cm of vertical deformation consistently occurred in the undoweled shale within 0.6 to 1.2 m of the advancing shaft bottom. The data also suggested a blocky post-peak response of the undoweled shale to excavation. The vertical boundaries of the blocks can be identified in Fig. 8 by locating adjoining heave gauge rings with ab-
normally large vertical displacements between them. These boundaries can be readily identified at depths of 64.3 m, 68.6 m, and right at the top of the predoweded zone at 73.2 m. This ground response is seen again within the predoweded zone at a depth of 76.2 m and may be indicative of yield occurring in the dowels within the top 3 m of the predoweded zone.

A similar ground response was recorded in the HG-2 field data (not shown) as well. In the undowedel shale, the blocky separations appeared at depths of 64.9 m, 67.0 m, and 70.1 m. There also appeared to be a separation of the undowedel shale from the predoweded shale when the excavation reached a depth of 71.3 m. The failure of portions of the dowels within the predoweded zone appeared to have occurred at a depth of approximately 77.1 m; however, it occurred only when the excavation had progressed to within 1.8 m of the final shaft bottom.

The predewedeling appears to have had an impact on horizontal deformations beneath the advancing shaft invert as well. When the inclinometers were read on day 92 (diamonds in Fig. 7), at a shaft bottom depth of 59.7 m (or 225 ft.), the data revealed that horizontal deformations were occurring to a depth of approximately 70.1 m, or 10.4 m ahead of the shaft bottom. Two days later, on day 94 (crosses in Fig. 7), the excavation had progressed to a depth of 61.3 m (232 ft) and the data revealed that horizontal deformations were occurring to a depth of approximately 73.2 m, or 11.9 m ahead of the shaft bottom at a depth of 61.3 m and corresponding with the top of the predoweded zone. As the excavation progressed, the magnitude of horizontal deformations continued to increase, but the deformations themselves were contained between the Austin Chalk/Eagle Ford Shale contact and the top of the predoweded zone. The data suggests that the predowels beneath the shaft invert maintained the integrity of the shale by controlling deformations and preventing the mechanism of blocky shale failure from developing.

The above response was recorded in all of the inclinometers except MDS-1 until some point after day 120. From day 116 to day 146 the shaft bottom depth was constant at 68.9 m while construction of the headings for both the starter and tail tunnels leading out of the shaft were completed. Inclinometer readings after day 128 revealed that the effect of horizontal containment due to the predowedeling reinforcement had begun to diminish and that the depth of horizontal deformations had extended into the predoweded zone. Once the initial horizontal deformations within the upper portions of the predoweded zone had begun, it is unclear as to the impact, if any, the dowels had on containing the magnitude of horizontal deformation within those portions of the invert. It is reasonable to assume however, that at that point, the shale may have actually moved “up and through” the patterned predowels.

The final inclinometer readings indicated that horizontal deformations of 2.5 to 12.7 mm occurred in the top 6 m of the predowedel invert with the deformations decreasing to less than 1.3 mm at a depth of 80 m.

Fig. 9. Piezometer readings beneath the shaft invert at the N15-MDS site, adapted from Shannon and Wilson, 1993

Horizontal deformations were not stabilized until the pouring of the 3 m thick concrete shaft invert. Inclinometer readings before and after the concrete pour revealed that the shale in the walls of the shaft and in the predoweded zone beneath the shaft had stabilized significantly following the concrete pour.

**Piezometric Measurements**

As a means of clarifying the overall mechanism of shale response to excavation, the measurement of pore pressure during construction was vital and the measured data are shown in Fig. 9. When the excavation reached a depth of 57.9 m and stopped for construction of the shaft’s lower liner key, the piezometers in the Eagle Ford Shale installed from the ground surface (i.e., P-3, P-4, and PP-4) registered a piezometric elevation of approximately 177.7 m, or 64.4 m above the Austin Chalk/Eagle Ford Shale contact. This value fluctuated slightly during construction of the shaft’s lower liner key, but was fairly consistent with the invert piezometers installed within the shaft at this point (i.e., P-5 to P-8), which indicated a piezometric elevation range of 173.6 to 178.1 m and an average piezometric elevation of 175.8 m. The piezometric elevation of 178.1 m was registered by the shallowest invert piezometer, P-5, and the piezometric elevation of 173.6 m was recorded by the deepest invert piezometer, P-7. With the range of piezometric elevations, it is unknown if all of the piezometers beneath the shaft invert reached equilibrium with the true ground water level before shaft excavation resumed below the lower liner key.

The response of the invert piezometers to the continued shaft excavation does however appear reasonable as shown in Fig. 9. All the piezometers beneath the shaft show a large decline in pressure with the advance of the excavation into the shale. The piezometric level recorded by the deepest piezometer, P-7, appears to follow the elevation of the excavation most closely. Nevertheless, when the shaft reached a depth of 68.6 m, or elevation 163.2 m, approximately 28.6 m above the P-7 tip elevation, the piezometric level moved below the bottom of the shaft invert, indicative of negative excess pore pressure.
response to the unloading excavation.

This negative excess pore pressure response was most evident in the piezometers closest to the excavated surface. P-5 and P-8, the two shallowest invert piezometers, recorded approximately the same negative excess pore pressure response for the first month following recommencement of shaft excavation. These two instruments actually recorded piezometric elevations below the instrument tip elevations. The data reveals that pressures less than zero gauge pressure were recorded at P-5 and P-8 on day 101 and day 116, respectively, prior to gradually returning to zero at the instrument tip elevation over a period of several weeks. The data is unclear as to whether this is the true piezometric response, or whether cavitation or breakdown in the pressured piezometer system occurred.

Overall, the piezometric field data indicates that a zone of negative excess pore pressure developed beneath the shaft invert as the excavation proceeded below the lower liner key and into the Eagle Ford Shale. This response appeared to be very tightly constrained to an area just beneath the shaft, as the piezometers outside the shaft perimeter (i.e., P-3, P-4, and PP-4) did not record the same response. Rather, they revealed a slight decrease in the piezometric elevation as the excavation progressed below the lower liner key and into the shale, but reached an elevation of approximately 170.4 m, or 0.8 m below the Austin Chalk/Eagle Ford Shale contact, and remained there throughout the rest of the construction period.

**Interpretation of Results**

From the recorded displacement and pore pressure instrumentation data, an initial understanding of the response of the Eagle Ford Shale to shaft excavation was developed. Horizontal deformations were not recorded in the Eagle Ford Shale until the shaft bottom was approximately 0.9 m above the Austin Chalk/Eagle Ford Shale contact. At that point, both the inclinometer and heave gauge readings revealed that the shale had begun to deform and respond to the approaching shaft bottom.

As the excavation progressed through the contact and into the shale, the piezometers beneath the shaft invert responded with a large drop in the recorded pore pressure. The inclinometers recorded movement along the Austin Chalk/Eagle Ford Shale contact, and suggested a stepped pattern of horizontal deformation in the undoweled shale below the contact. At the same time, the heave gauge data suggested that vertically, the undoweled shale separated and failed in blocks as the excavation progressed. Figure 8 indicates that the vertical thickness of the blocks ranged from approximately 4-5 meters in the undoweled zone to approximately 3 meters in the predoweled zone. Features of blocky failure were also observed in the field as vertical cracks and irregular surface heave at the base of the excavation. A schematic of this response based on field observation and instrument data interpretation is presented in Fig. 10.

**NUMERICAL ANALYSIS OF SHAFT CONSTRUCTION**

From the recorded displacement and pore pressure data, an initial understanding of the rock mass response to excavation was developed and a numerical analysis of the shaft construction process was performed. The numerical analysis focused on identifying the mechanism of shale failure and the impact of failure on the measured pore pressure response. The predictions of displacements and pore pressure obtained through the analysis were used to examine the validity of the suggested mechanisms of shale response formulated through analysis of the field data.

**Effective Stress Analysis**

Undrained, effective stress 2D and 3D models were used to investigate the ability of the effective stress approach to predict the ground and pore pressure response of the Eagle Ford Shale. The numerical modeling was carried out using the finite difference codes FLAC (Fast Lagrangian Analysis of Continua) and FLAC3D developed by the Itasca Consulting Group (Itasca, 2000a and 2000b).

In the 2D analysis, the rock mass was discretized using a finite difference grid of 34 by 61 zones, representing a 230.4 m wide by 231.8 m deep section. The dimensions
Table 1. Mechanical and index properties of Austin Chalk and Eagle Ford Shale

<table>
<thead>
<tr>
<th></th>
<th>Austin Chalk</th>
<th>Eagle Ford Shale</th>
</tr>
</thead>
<tbody>
<tr>
<td>1. General Characteristics</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Dry unit weight</td>
<td>19.5 kN/m³</td>
<td>18.4 kN/m³</td>
</tr>
<tr>
<td>Total unit weight</td>
<td>22.0 kN/m³</td>
<td>21.8 kN/m³</td>
</tr>
<tr>
<td>Density</td>
<td>0.024</td>
<td>0.024</td>
</tr>
<tr>
<td>Liquid limit</td>
<td>NA</td>
<td>90%</td>
</tr>
<tr>
<td>Plastic limit</td>
<td>NA</td>
<td>30%</td>
</tr>
<tr>
<td>2. Effective stress strength parameters</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Cohesion (k)</td>
<td>4540 kPa</td>
<td>235 kPa (peak) and 0 kPa (residual)</td>
</tr>
<tr>
<td>Friction angle (ϕ)</td>
<td>30°</td>
<td>20° (peak) and 12° (residual)</td>
</tr>
<tr>
<td>Tension cut-off</td>
<td>NA</td>
<td>138 kPa</td>
</tr>
<tr>
<td>3. Deformation characteristics</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Bulk Modulus for Elastic Model</td>
<td>1760 MPa</td>
<td>232 MPa</td>
</tr>
<tr>
<td>Shear Modulus for Elastic Model</td>
<td>1270 MPa</td>
<td>199 MPa</td>
</tr>
<tr>
<td>Swelling Index Cs</td>
<td>NA</td>
<td>0.018 - 0.018</td>
</tr>
<tr>
<td>4. Hydraulic Conductivity</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Parallel to bedding</td>
<td>5 x 10⁻⁶ m/s</td>
<td>1 x 10⁻⁶ m/s</td>
</tr>
<tr>
<td>Normal to bedding</td>
<td>5 x 10⁻⁶ m/s</td>
<td>1 x 10⁻⁶ m/s</td>
</tr>
<tr>
<td>5. In-situ Ko ratio</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Maximum (direction)</td>
<td>2.1 (N318°)</td>
<td>1.5 (N18°)</td>
</tr>
<tr>
<td>Minimum (direction)</td>
<td>1.5 (N85°W)</td>
<td>1.5 (N118°)</td>
</tr>
</tbody>
</table>

Material Properties

Material property data used as a starting point in the numerical analysis was developed from various sources associated with the SSC project (Table 1). This data includes laboratory and field test results obtained during the feasibility level, and initial and final design level geotechnical exploration programs for both the Austin Chalk and Eagle Ford Shale. Specifically, a large portion of the index testing of the Eagle Ford Shale for the SSC was performed at the University of Texas at Austin (Olson et al., 1993; Brouillette et al., 1993; Hsu, 1996; Lai, 1997). One-dimensional compression and swelling tests, direct shear tests and triaxial compression and extension tests (both drained and undrained) were also performed. Lai (1997) reported that in undrained compression tests, samples of Eagle Ford Shale generated positive excess pore pressures, whereas in triaxial extension, samples generated negative excess pore pressures. This suggests that stiffness anisotropy can be an important aspect of the deformation characteristic (i.e. Graham and Houlsby, 1983; Yimsiri and Soga, 2002). In the analysis for this paper, the data from triaxial extension tests were essential for modeling the deformation characteristics of the shale.

Estimates of hydraulic conductivity were developed from field packer testing. The results indicated that the hydraulic conductivities ranged from 7.8 x 10⁻⁷ to 2.7 x 10⁻⁵ m/s in the Austin Chalk and 2.1 x 10⁻⁸ to 4.3 x 10⁻⁹ m/s in the Eagle Ford Shale. Lai (1997) measured the hydraulic conductivities of the Eagle Ford Shale using the constant head technique and reports the values ranging from 5 x 10⁻¹¹ m/s to 5 x 10⁻¹² m/s. The significantly lower laboratory values may be indicative of the lack of discontinuities present in intact laboratory samples. However, based on both laboratory and field permeability values, the response of the Eagle Ford Shale during construction is considered to be essentially an undrained condition.

Pressuremeter and hydraulic fracturing testing was performed to estimate the in-situ stress conditions. Given, σ₁, the maximum effective horizontal stress, σ₃, the minimum effective horizontal stress and σ₃, the effec-
Table 2. Mechanical and hydraulic parameters for effective stress analyses using elastic, isotropic and Mohr-Coulomb plasticity models

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Austin Chalk</th>
<th>Eagle Ford Shale</th>
</tr>
</thead>
<tbody>
<tr>
<td>Density (kg/m^3)</td>
<td>1987</td>
<td>1875</td>
</tr>
<tr>
<td>Bulk modulus (Pa)</td>
<td>1.76 x 10^9</td>
<td>2.32 x 10^8</td>
</tr>
<tr>
<td>Shear modulus (Pa)</td>
<td>1.27 x 10^9</td>
<td>1.99 x 10^8</td>
</tr>
<tr>
<td>Cohesion (Pa)</td>
<td>--</td>
<td>2.35 x 10^5</td>
</tr>
<tr>
<td>Angle of friction (deg.)</td>
<td>--</td>
<td>12-20</td>
</tr>
<tr>
<td>Tension cut-off (Pa)</td>
<td>--</td>
<td>1.38 x 10^5</td>
</tr>
<tr>
<td>Porosity</td>
<td>0.24</td>
<td>0.32</td>
</tr>
</tbody>
</table>

tive vertical stress, the σ_i/σ_j stress ratio in the Austin Chalk was 1.2 to 2.2 and the σ_i/σ_j stress ratio was 0.9 to 1.8. In the Eagle Ford Shale, the σ_i/σ_j stress ratio was determined to be 1.2 to 1.6. The testing also indicated that the ratio of maximum to minimum horizontal stresses, the σ_i/σ_j ranged from 1.0 to 1.4, suggesting that the horizontal stress state within the shale is fairly uniform (Teetes, 2001).

Material Models

The Austin Chalk was modeled in this analysis using an elastic, isotropic model with constant stiffness. The elastic properties used are listed in Table 2. The decision to describe the Austin Chalk behavior with this model was based on two premises: first, the shear strength in comparison to the range of stresses encountered was relatively high; and second, a desire to simplify where possible the numerical analysis process.

In contrast, four different constitutive models were used in an attempt to describe the Eagle Ford Shale response. First, an elastic, isotropic model with a constant stiffness; second, a Mohr-Coulomb plasticity model with the shear yield surface corresponding to a Mohr-Coulomb criterion, and either dilatant or non-dilatant post-peak behavior; third, the isotropically hardening/softening elastoplastic Modified Cam Clay model; and fourth, the same Cam Clay model but with yield described via a Hvorslev strength criterion. The model parameters used for the Modified Cam Clay and Cam Clay/Hvorslev models are listed in Table 3. As presented above, the material properties of the models were determined using undrained triaxial extension test data reported by Lai (1997).

Due to markedly different physical properties of the Austin Chalk and Eagle Ford Shale, an attempt was made to model the contact between the two formations with an interface as shown in Fig. 11. The interface was assumed to behave according to a linear Coulomb shear strength criterion, and it was given a friction angle of 10 degrees.

Table 3. Mechanical parameters for effective stress analyses using modified Cam Clay and Hvorslev strength criterion models

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Modified Cam Clay</th>
<th>Hvorslev</th>
</tr>
</thead>
<tbody>
<tr>
<td>M_{ext} (θ)</td>
<td>0.39-0.61 (12-20)</td>
<td>0.39-0.61 (12-20)</td>
</tr>
<tr>
<td>κ (C_s)</td>
<td>0.012 (0.028)</td>
<td>0.012 (0.028)</td>
</tr>
<tr>
<td>δ (C_s)</td>
<td>0.033 (0.076)</td>
<td>0.033 (0.076)</td>
</tr>
<tr>
<td>v (e)</td>
<td>1.471 (0.471)</td>
<td>1.471 (0.471)</td>
</tr>
<tr>
<td>p'</td>
<td>20.7 MPa</td>
<td>8.6 MPa</td>
</tr>
<tr>
<td>p'_i</td>
<td>2.1 MPa</td>
<td>2.1 MPa</td>
</tr>
<tr>
<td>G</td>
<td>1.99 x 10^9 Pa</td>
<td>1.99 x 10^9 Pa</td>
</tr>
<tr>
<td>ν</td>
<td>0.3</td>
<td>0.3</td>
</tr>
</tbody>
</table>

Ground Support

In a 2D axisymmetric model, patterned dowels in the wall of a shaft can be included in an axisymmetric analysis by factoring the input properties of the dowels by the dowel spacing, and in effect "smearing" the support along the entire shaft. The technique however, is not plausible for representing dowels in the invert of a shaft. To represent the predoweled zone beneath the shaft invert in the axisymmetric analyses, an equivalent material model was developed (Teetes, 2001). The equivalent material model accounted for the impact of the predowels numerically in the continuum model by an increase in the Young’s modulus and cohesion of the material. For the equivalent material, the Young’s modulus was assumed to increase most significantly in the direction parallel to the dowels, while the cohesion was assumed to increase most significantly in the direction normal to the dowels.

In the 3D model, shotcrete and cement grouted rock dowels were installed in accordance with the actual construction sequence. Prior to each numerical excavation stage, support was installed up to the bottom of the advancing shaft. The thickness of the mesh representing the concrete and shotcrete support was maintained at a constant dimension. However, in order to model the change in actual lining thicknesses from the Austin Chalk (0.15 m) to the Eagle Ford Shale (0.30 m) and resulting change in bending stiffness, the stiffness of the lining was varied at different rock sections based on the 28-day Young’s modulus for concrete.

Over 1600 rock dowels installed during construction were incorporated in the model and this number includes the approximately 100 predowels that were installed below the final shaft invert while the shaft excavation was still in the Austin Chalk formation. The lengths, spacing and angle of installation varied by formation and by depth as shown in Fig. 12. The predoweled zone consisted of 15 m long, No. 11 bars (0.0349 m diameter) grouted into 30 m long vertical and sub-vertical holes drilled from the base of the shaft’s lower liner key. No. 9 bars (0.0289 m diameter) were used for the other rock dowels.
Table 4. Rock dowel specifications

<table>
<thead>
<tr>
<th>Shaft depth</th>
<th>Rock type</th>
<th>Length (m)</th>
<th>Bar size</th>
<th>Hole dia. (m)</th>
<th>Spacing (m)</th>
</tr>
</thead>
<tbody>
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The steel bars have elastic modulus of $2.1 \times 10^5$ GPa and tensile yield strength of $1.25 \times 10^5$ kN. The pattern spacing and lengths of the rock dowels are presented in Table 4. It was assumed in the model that the bars and the surrounding rock were tied tightly together by the grout.

Modeling Approach

Two methods for analyzing the deformational post-peak response of the shale observed during construction were considered. In the first, “conventional” method, excavation of the shaft was based as closely as possible on the actual construction sequence and focused on the ability of each of the four chosen constitutive models to predict the response of the shale as shown in Fig. 13(a). In these analyses, the model simulation consisted of excavation of the shaft collar to a depth of approximately 11 m, followed by 39 excavation stages, ranging from 1.0 to 2.2 m each, to the bottom of the shaft at a depth of 73 m.

In the second method, a technique was developed to replicate the impact of the blocky failure on the redistribution of stresses beneath the shaft. The instrumentation data suggests that failure and fragmentation of the Eagle Ford Shale occurred as the shaft excavation progressed through the Austin Chalk/Eagle Ford Shale contact and into the brittle, over-stressed, undowedeled shale beneath the contact. Based on the progression of the blocky failure captured in the instrumentation data, the excavation sequencing was modified as shown in Fig. 13(b). The simulation proceeded as described above until just above the Austin Chalk/Eagle Ford Shale contact. The modified simulation then involved the removal of the last zone of Austin Chalk above the contact and the replacement of the first four zones of undowedeled Eagle Ford Shale with an applied vertical stress equivalent to the weight of the removed zones. The next step was a repeat of the first, with the remainder of the undowedeled shale being removed and a vertical stress representing the remaining zones being applied to the shaft invert, and in effect, simulating the progression of the blocky failure. The final step in the model involved removal of the remaining applied vertical stress, and replacement of the stress with zones representing the final concrete shaft invert.

Both methods were applied to the 2D model primarily to examine the differences in both displacement and pore pressure response with different soil models. Only the conventional method was applied to the 3D model primarily to investigate whether the detailed modeling of rock dowels improved the displacement prediction.

Numerical Analysis Results

The 2D and 3D numerical modeling results using the conventional approach indicate that each of the four constitutive models, to varying degrees of accuracy, is able to predict the displacements occurring around the elliptical N15-MDS. Figure 14 presents a horizontal prediction of displacement for the four constitutive models evaluated. The elastic, isotropic model for the shale predicts the lowest maximum horizontal displacement, while the Cam Clay/Hvorslev model predicts the highest maximum horizontal displacement. Figure 15 shows the vertical displacement predictions for the four constitutive models evaluated. Both the elastic, isotropic and Mohr-Coulomb models closely predicted the shape and magnitude of the vertical displacement curve developed from the instrumentation data within the predowedeled zone, but underpredicted the vertical displacements in the undowedeled zone. Both critical state models overpredicted the magnitude of the vertical displacements within the predowedeled zone and within the undowedeled zone.

The Modified Cam Clay model predicts horizontal and vertical displacements within 1 mm and 14 mm of the measured field data; however, the stress state remains inside the yield surface throughout the shaft excavation. Hence, the observed difference between the isotropic elastic and Cam Clay models is essentially due to the difference in models (constant elastic stiffness versus pressure dependent stiffness) and the material properties adopted. Both Mohr-Coulomb and Cam Clay/Hvorslev models showed some yielding at the invert and sidewalls. Hence, when the results of the isotropic elastic model and the Cam Clay model are compared to those of the Mohr- Coulomb model and Cam Clay/Hvorslev model, respectively, larger horizontal displacements were calculated.
for the latter models. However, it is difficult to conclude whether any of the models performed better than the others.

The pore pressure predictions using the Cam Clay model are shown in Fig. 16. The solid lines represent the measured pore pressure response at each of the four piezometers beneath the shaft, and the dashed lines represent the model predictions of pore pressure response at the corresponding locations in the numerical model. The other models showed similar pore pressure response; pore pressure response remained fairly constant until 150 days following the start of excavation and then a sudden drop was observed at the shallowest piezometer depths (P5 and P8 in Fig. 16). Although the dilative behavior upon yielding by the Cam Clay/Hvorslev model contributed to additional decrease in excess pore pressures at the later stages of excavation (not shown), none of the four constitutive models chosen to describe the shale behavior were capable of capturing the large initial drop in pore pressure beneath the shaft after 90 days following the start of excavation.

The modified method was used to capture the impact of the failure and fragmentation of the Eagle Ford Shale. The pore pressures were predicted using the Cam Clay/Hvorslev model and the computed results at the four piezometer locations beneath the shaft are shown in Fig. 17. The figure indicates that the technique is capable of reproducing the large drop in pore pressure recorded in the field. Furthermore, the variability in magnitude of the pore pressure response with depth is reproduced. Although the predicted trends are accurate, the magni-
Fig. 18. Axisymmetric and 3D total stress model predictions of horizontal displacements

tude of the final predicted response at the P5 and P8 piezometer locations may be exaggerated. It should be noted that, in an effort to better evaluate the entire pore pressure response beneath the shaft, the water was given a high tension limit value in order to prevent cavitation from occurring.

The numerical analysis also attempted to investigate the sensitivity of the model prediction to model geometry. Additional analyses focusing mainly on comparing the 2D and 3D model predictions of horizontal and vertical displacements were performed. As presented earlier, the equivalent circular radius was represented in the 2D axisymmetric analyses by an average axis radius of the ellipse (i.e., radius = \(a + b)/2\)).

Figure 18 is a plot of the instrumentation field data from inclinometer MDS-1 and the axisymmetric and 3D total stress predictions of horizontal displacements. Both analyses assumed a Mohr–Coulomb plasticity constitutive model for the shale, and the same strength and deformation input parameters were used. Compared to the field data, both the axisymmetric and 3D predictions of maximum horizontal displacements were fairly accurate. The inclinometer recorded a maximum horizontal displacement of 22.4 mm while the axisymmetric model predicted 19.0 mm, or a difference of 3.4 mm. The 3D model predicted 24.4 mm of horizontal displacement, or a difference of 2.0 mm.

Although including the actual invert predowels in the axisymmetric model was not plausible, a comparison of axisymmetric and 3D vertical displacement predictions is presented as a general comparison of the models’ ability to predict the vertical response beneath the shaft (Fig. 19). The 3D model prediction is closer to the measured field data in the undoweded shale (between the predoweling and the bottom of the advancing shaft at a depth of 68.6 m) than the axisymmetric model prediction, however, the 3D model prediction is less accurate than the axisymmetric model prediction throughout the predoweled zone and below. Hence, it was difficult to conclude whether the 3D model that included the detailed modeling of rock dowels was able to capture the observed deformation behavior better than the 2D model. Further investigation is needed.

Overall, the numerical analysis results indicate that mechanistic application of a simple best-fit computer model to predict ground response may lead to serious misconceptions regarding the mechanism of ground behavior. In addition the results indicate that the technique developed for simulating the impact of the blocky, post-peak response of the shale on the redistribution of stresses around the shaft opening was successful in describing the pore pressure response beneath the shaft invert, and has proven to be a useful tool in studying the redistribution of stresses in blocky materials. However, further investigation of the effect of cavitation on soil behaviour and its modelling are necessary to achieve a better simultaneous evaluation of pore pressure and deformation response during excavation.

CONCLUSIONS

A general understanding of the impact of predoweling reinforcement on the behavior of the shale was obtained
through the instrumentation data obtained during this project. The data suggests that the predowels were effective at controlling both horizontal and vertical displacements and maintaining the integrity of the shale as the excavation progressed. The predowels were able to control displacements until the point, at which the shear strength along portions of the predowels was exceeded. At that point, the heave gauge data indicated that portions of the predowels failed and blocks of shale moved up and through the predowels into the shaft.

Failure in the Eagle Ford Shale was due primarily to the low shear strength of the cemented clay shale relative to the range of stresses encountered. In the field, failure occurred along bedding planes and other planes of weakness resulting in the blocky invert conditions, which were captured in the instrumentation data and described herein. Given the blocky, post-peak response, the pattern of stress redistribution was directed further ahead of the excavation, resulting ultimately in the progression of the blocky failure into the predoweded zone. The removal of the Austin Chalk, the subsequent failure of the undoweded shale, and the resulting change in the state of stress around the excavation combined to create the drop in pore pressure that was recorded at the four piezometers beneath the shaft.

Four constitutive models were investigated for predicting and describing the Eagle Ford Shale response to excavation. The numerical results indicate that each of the four constitutive models, to varying degrees of accuracy, was able to predict the displacements occurring around the elliptical N15-MDS. However, they were incapable of successfully predicting the post-peak, blocky behavior of the shale and the resulting pore pressure response, which was recorded as the excavation progressed through the Austin Chalk into the Eagle Ford Shale. A technique was developed for simulating the impact of the blocky, post-peak response of the shale by replacing zones of undoweded shale in the model with vertical stresses equivalent only to the weight of the replaced material was performed. The numerical results indicate that the technique was successful in describing the pore pressure response beneath the shaft invert, and has proven to be a useful tool in studying the redistribution of stresses in blocky materials. Pore pressure data was used to substantiate the appropriateness of the numerical analysis approach, and clearly indicates the importance of an effective stress analysis in providing a rational explanation of the behavior of saturated soft rocks.

REFERENCES

13) SSCL (1992): Laboratory status report, Superconducting Super Collider Laboratory, Dallas, Texas.