Soil characterization for the north transversal tunnel on Grenoble area

J. Monnet  
LIRIGM, Université Joseph Fourier, BP 53, 38041, Grenoble, France

C. Chapeau  
CETE-Lyon, 25 av. François Mitterrand, Case 1, 69674, Bron Cedex

G. Godard  
DDE, 17 bld Joseph Vallier, 38000, Grenoble

ABSTRACT: The tunnel construction in urban area is a particular sensitive problem, by the careful control of the security requirements to prevent failure, but also to control displacements so that damage should be avoided. For the preliminary design, it is necessary to study each alternative plants of the project and to propose the best adaptation of the construction to the soil. The determination of the geomechanical characteristics is essential.

The preliminary laying out of the tunnel of the North transversal road of Grenoble begins from the Sablon district, crosses under river Isére, passes under the Ile Verte district, crosses a second time under river Isère to join the motorway A48 at Porte de France. The geology of the layout includes in the first part alluvial soil with silty sands and clays, then after the second crossing of the Isère river it includes limestone of Chartreuse. The selected geotechnical investigation uses boreholes to study the soil at the level of the construction that is 10 to 20m depths for the soil description, classification, and behaviour along the tunnel construction.

Sampling of silty sands under the water table is very difficult so that the deformation and failure parameters are measured by pressuremeter tests. The geomechanical data variability is taken into account for the geomechanical parameters for the project and for the tunnel drilling. The project should be constructed in the first decade of this new century.

1 INTRODUCTION

Tunnel construction in urban area is very difficult to overcome, as there are a lot of safety rules to apply and prevent failure, and an accurate control of surface displacements to make. From an economical and technical point of view, the design must be adapted to the soil, which had to be well known. The usual method for geotechnical investigations is to use samples of soil, which are tested on laboratory equipments. Unfortunately, sandy and silty soil lower than water table cannot be taken out without remoulding. Thus, pressuremeter analysis is provided, by its ability to measure, in a borehole, mechanical characteristics of undisturbed soil. It is the technique used in Grenoble geotechnical campaign.

2 PROJECT DESCRIPTION

The preliminary laying out of the tunnel of the North transversal road of Grenoble begins from the Sablon district, crosses under river Isère, passes under the Ile Verte district, crosses a second time under river Isère to join the motorway A48 at Porte de France. A complete analysis on different drawings of the north transversal road (Figure 1), leads to a layout, which crosses over « Ile Verte » district for technical facilities, construction techniques, disturbance of surface traffic, and coast. Then, outline of the tunnel were studied, as well as longitudinal profile. The size of the vehicles was taken into account (Figure 2). Two maximal heights (2m and 2.7m) and two longitudinal profiles were studied. The main benefits of these limits are the increase of the tunnel slopes and approaches, a best fitting of the project in urban area, the ability to construct a cloverleaf for the Michallon hospital, and the prohibition from truck traffic so that security is improved by removal of the fire risk. On the opposite side, smokes do not stay under ceiling, rescue vehicles must be adapted, and vehicles must be sort out at tunnel entrance.

The 2m height allows 93% of vehicles to pass through as, car, trading vehicle, pick-up. On the opposite side, coach, bus, rescue vehicle, are not allowed. The 2.7m height (Figure 2) allows 96% of vehicles to pass through, as lowered bus, ambulance. But rescue vehicle must be adapted to the size of the tunnel. This height is 15% more expensive than the
previous one. The total coast is estimated between 378M € and 509M €.

- Stratum rock, under Bastille district, with calcareous and lime marl.

3.2 Geotechnical description

Geotechnical campaign was carried out in 1996-1997 in the Ile Verte district, with 4 destructive boreholes, 8 boreholes with sampling (30m to 40m depth), 15 pressuremeter boreholes with standard tests (NF P 94-110-1, 2000) and cyclic tests, triaxial tests on undisturbed samples, cyclic triaxial tests, oedometric tests, physical identification. The results of the exploration are shown on Table 1. Alluvial deposits are made of 8 families of soils, which are:
- Family F1, Surface filling, 1 to 4m depth,
- Family F2, Plastic clay, under family F1,
- Family F3, Surface brown and grey silt, 3 to 7m thick, under family F1,
- Family F4, Sand and gravels, 3 to 20m thickness, under F3. Gravels are smaller than 100mm and the mean value is 20mm.
- Family F5, Grey clayey silt, replaces family F4 on the eastern entrance of the tunnel,
- Family F6, Grey sand and silt, under family F4, with pockets of silt and clay,
- Family F7, Deep sandy grey silt, on the western entrance of the tunnel,
- Family F8, Deep stiff sand, as the lowest layer of soil.

Table 1. Results of the geotechnical in situ exploration

<table>
<thead>
<tr>
<th>Family</th>
<th>Description</th>
<th>Pressuremeter modulus (MPa)</th>
<th>Limit Pressure (MPa)</th>
<th>Number of tests</th>
</tr>
</thead>
<tbody>
<tr>
<td>F1</td>
<td>Surface filling</td>
<td>21.2</td>
<td>0.99</td>
<td>6</td>
</tr>
<tr>
<td>F3</td>
<td>Surface brown and grey silt</td>
<td>5.1</td>
<td>0.42</td>
<td>23</td>
</tr>
<tr>
<td>F4</td>
<td>Sand and gravels</td>
<td>37.8</td>
<td>3.6</td>
<td>60</td>
</tr>
<tr>
<td>F5</td>
<td>Grey clayey silt</td>
<td>2.46</td>
<td>0.31</td>
<td>5</td>
</tr>
<tr>
<td>F6</td>
<td>Grey sand and silt</td>
<td>27.7</td>
<td>1.76</td>
<td>118</td>
</tr>
<tr>
<td>F7</td>
<td>Deep sandy grey silt</td>
<td>14.7</td>
<td>0.85</td>
<td>49</td>
</tr>
<tr>
<td>F8</td>
<td>Deep stiff sand</td>
<td>40.2</td>
<td>2.8</td>
<td>14</td>
</tr>
</tbody>
</table>

3 SOIL DESCRIPTION

3.1 Geological description

The tunnel passes through two different geological formations, which are:
- Alluvial deposits, which are from 500m to 800m thick, and made of quaternary gravel, sand, silt, clay fillings. This is lacustrian and fluvial formation.

4 PRESSUREMETER THEORY

Pressuremeter test was though up by Ménard (1955), and is well known for foundation design (Ménard, 1957, Gambin, 1979, Amar et al., 1991). This test allows the determination of pressuremeter modulus, which is used for settlements. Limit pressure is the pressure measured for a volume of the probe, which is twice the initial one. It is used along correlation curves to find out bearing capacity of foundations.
4.1 Shearing theory of pressuremeter test

As shown by Clarke & Gambin (1998), pressuremeter probe shears the soil in the horizontal plane around the borehole. This assumption was used by Hughes et al. (1977) to find a relation between the logarithms of radial stress and strain, which is linked to the friction angle \( \varphi' \), but the expression of the pressuremeter curve is not found. Monnet (1990) and Monnet & Khelif (1994) assume that the soil, around the pressuremeter probe, is a non-standard elasto-plastic material, with a dilatancy angle \( \Psi \) linked to the interparticle angle \( \varphi_\mu \) by the relation of Monnet & Gielly (1978):

\[
\Psi = \varphi' - \varphi_\mu
\]  

(1)

The failure of the soil is ruled by Mohr-Coulomb criteria, and the plasticity, which appears between the radial stress and the circumferential stress gives the relations:

\[
N = \sigma_0' / \sigma_r' = (1 - \sin \varphi') / (1 + \sin \varphi') \quad (2)
\]

\[
n = -\frac{\delta \sigma_r}{\delta \varphi} = (1 - \sin \Psi) / (1 + \sin \Psi) \quad (3)
\]

The elasto-plastic equilibrium between the radial, circumferential and vertical stresses leads to the determination of the theoretical formulation of the pressuremeter curve, by Equation (4). The soil is assumed to be drained, horizontal and vertical equilibrium are used.

\[
\ln \left( \frac{\sigma_r}{\sigma_0} \right) + N = \ln (1-K_0) + \ln \left( \frac{1+n}{1-n} \right) \quad (4)
\]

with \( \delta = \frac{1+n}{1-N} \) and

\[
n = \frac{\delta}{\sigma_0} \left( \frac{\sigma_r}{\sigma_0} \right)^2 + (1+n) \left( \frac{\sigma_0}{\sigma_r} \right)^2 \quad (5)
\]

The C1 value is small and may be neglected, so that a linear relation is found between the logarithms of pressure \( p \) applied to the borehole and radial strain \( u_r/a \) of the borehole (Figure 3). In this relation \( G \) is the shear modulus of the soil, \( \gamma \) is the unit weight of the soil, \( z \) the depth of the test, \( K_0 \) is the coefficient of earth pressure at rest, \( a \) the radius of the borehole, and \( u_r \) the radial displacement at the borehole.

Equation (4) may be used for a radial strain equal to \( \sqrt{2} - 1 \) so that the volume of the pressuremeter is twice the initial one, and the limit pressure is reached. This leads to the theoretical value of the limit pressure:

\[
P = \frac{\gamma \sqrt{2} \sqrt{1+n} \left( \frac{1+n}{1-n} \right) \ln \left( \frac{1+n}{1-n} \right) - C_1 \left[ 2G \right]^{1/2}}{\left[ (1-K_0) \right]^{1/2} \left( 1+n \right) \gamma z - 2G.C_1} \quad (6)
\]

The shearing of the soil between the radial and circumferential stresses is ruled by the vertical stress, which is perpendicular to the failure plane. Thus, Equation (6) shows that the limit pressure is proportional to the vertical stress for a frictional soil. As a consequence of theory, limit pressure is linked to the shear modulus \( G \), the friction angle \( \varphi' \) and the interparticle angle of friction \( \varphi_\mu \) through \( n \) and \( \Psi \).

4.2 Correlation between limit pressure and friction angle

A lot of pressuremeter tests were carried out by Ménard, which exhibited a lot of empirical rules. The relation between the limit pressure and the friction angle, as shown by Amar et al. (1991) was used:

\[
p_* = 250.2 \left[ (\varphi' - \varphi_\mu) \right]^{1/2} \quad (7)
\]

The empirical coefficient 250 is fitted for soil that unit weight is closed to 20kN/m\(^3\). It may be lowered to 180 for soft soil and increased to 350 for stiff soil. It is linked to the ratio between the pressuremeter modulus \( E_m \) over the limit pressure \( P_* \). The coefficient 250 was used here.

5 RESULTS OF TESTS

5.1 Experimental procedure for pressuremeter tests

The geotechnical campaign used different steps:
- Measurement of the interparticle angle of friction \( \varphi_\mu \) on remoulded or intact samples tested
on triaxial apparatus. This value is not linked to the soil density (Monnet & Gielly 1978), but to the surface between particles of soil. For particles rolled by erosion, the value of $\varphi_\mu$ is smaller than the value found for particles broken by crusher. On the other side the internal angle of friction found by triaxial tests is not validated because of its dependency to the density of the sample, which is different from the in situ one.

- Measurement of dilantancy $\Psi$, and internal angle of friction $\varphi'$ by the way of pressuremeter test. It is assumed that the soil is non cohesive and drained. The internal angle of friction is validated because the soil is tested in its in situ density. The test is analysed as a shearing test with a normal pressure equal to the vertical stress.

- Smoothing of friction characteristics with depth, so that a mean value is found. Extreme values are cancelled, which are linked to local cohesion of the soil. It is the case for large pebbles, which are closed to the probe.

5.2 Other geotechnical tests

Triaxial tests were carried out in Lirigm laboratory of Grenoble university along the French Standard NF P 94-070 (1994). Samples were 70mm diameter and 150mm height. Results are shown on Table 2, $E$ is Young’s modulus, and $R_f$ is the failure ratio. CETE, Aix en Provence carried out cyclic triaxial tests to study the liquefaction risk. The soil seems to be sensitive to seismicity, and it is found a large variation of the undrained Young modulus along cyclic loading.

Table 2. Characteristic values for geotechnical parameters

<table>
<thead>
<tr>
<th>Family</th>
<th>$E$ (MPa)</th>
<th>$\nu$</th>
<th>$\varphi_\mu$</th>
<th>$C$ (kPa)</th>
<th>$\varphi'$</th>
<th>$R_f$</th>
</tr>
</thead>
<tbody>
<tr>
<td>F6</td>
<td>36</td>
<td>0.265</td>
<td>29.8°</td>
<td>0</td>
<td>36.4°</td>
<td>0.847</td>
</tr>
<tr>
<td>F6</td>
<td>0 - 13.5</td>
<td>37.4°</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>F7</td>
<td>23.8</td>
<td>0.392</td>
<td>32.9°</td>
<td>0</td>
<td>38.2°</td>
<td>0.835</td>
</tr>
<tr>
<td>F7</td>
<td>17</td>
<td>0.388</td>
<td>30.2°</td>
<td>0</td>
<td>36.4°</td>
<td>0.696</td>
</tr>
<tr>
<td>F7</td>
<td>0</td>
<td>35.2°</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

5.3 Pressuremeter analysis

We used a push in pressuremeter probe with a slotted tube 60mm external diameter, 49mm internal diameter, and slots 110mm length. The calibrations of French Standard NF P 94-110-1 (2000) were used, but additional corrections were made as shown in Gaiatech patent (1989), to take into account, the influence of the external radius where soil reacts and the internal radius where pressure is applied, the shape of the probe under solicitation as shown by Fawaz et al.(2000), and the non-uniformity of the stress along the probe as shown by Basudhar & Kumar (1995). The values issued from these corrections are noticed $E^+$, $E'$, $p_f$ in Table 3.

Table 3. Results of the pressuremeter analysis – Borehole SPl2

<table>
<thead>
<tr>
<th>Family</th>
<th>Depth (m)</th>
<th>$E^+$ (MPa)</th>
<th>$E'$ (MPa)</th>
<th>$p_f$ (kPa)</th>
<th>$p_{theo}^+$ (kPa)</th>
<th>$\varphi'$</th>
</tr>
</thead>
<tbody>
<tr>
<td>F3</td>
<td>4</td>
<td>7.9</td>
<td>22.7</td>
<td>590</td>
<td>700</td>
<td>36°</td>
</tr>
<tr>
<td>F4</td>
<td>6</td>
<td>7.7</td>
<td>18.3</td>
<td>1640</td>
<td>1550</td>
<td>52°</td>
</tr>
<tr>
<td>F4</td>
<td>8</td>
<td>15.9</td>
<td>50</td>
<td>3780</td>
<td>3240</td>
<td>53°</td>
</tr>
<tr>
<td>F6</td>
<td>13</td>
<td>15.2</td>
<td>31.3</td>
<td>810</td>
<td>975</td>
<td>31°</td>
</tr>
<tr>
<td>F6</td>
<td>16</td>
<td>16.7</td>
<td>30.2</td>
<td>685</td>
<td>1010</td>
<td>30°</td>
</tr>
<tr>
<td>F6</td>
<td>18</td>
<td>27.5</td>
<td>42.7</td>
<td>760</td>
<td>1130</td>
<td>28°</td>
</tr>
</tbody>
</table>

Equation (4) shows that pressuremeter curves are linked to shear modulus $G$, interparticle angle of friction $\varphi_\mu$, internal angle of friction $\varphi'$, and depth. As the depth of the test, and interparticle angle are known, there are only two parameters, which remain unknown. A cyclic unloading-reloading is carried out to separate the influence of shear modulus and friction angle. Shear modulus is measured on the slope of the cycle (Figure 4) and elastic modulus $E^+$ is found, while friction angle $\varphi'$ is measured (Table 3) by the slope $\delta$ of the linear relation between the logarithms of pressure versus borehole deformation (Figure 3). This linear relation is used upper than the swelling pressure so that the influence of disturbance linked to the borehole drilling can be neglected. The final control of the measured values is made by fitting the theoretical curve over the experimental one (Figure 4), and by comparison between the experimental limit pressure $p_f^+$, and the theoretical one $p_{theo}^+$ (Table 3). For the soils that interparticle angles are unknown a value of 30° is assumed.

Figure 4. Control of the mechanical characteristics, comparison between theoretical and experimental pressuremeter curves, Borehole SP12 at 6m depth
These results were compared with the solution of Equation (7) of Ménard correlation (Table 4). A maximum difference of 2° is observed for family F4 (sand and gravels) and F7 (Deep sandy grey silt). The difference grows to 5° for F6 (Grey sand and silt), and the correlated value (35.3°) is close to the triaxial value (37.3°) but samples tested on laboratory were consolidated under confinement pressure, so that large volume variations were observed, and friction angle is overestimated. The difference is 8° for F3 (Surface brown and grey silt), which are located between 3 to 7m depth. In this range, the Ménard Equation (7) underestimates the angle of friction by the constant coefficient 250, which is fitted for deeper tests.

However, the correlation values were used for design parameters as the standard analysis shown by Amar et al. (1991)

Table 4. Soil family synthesis for the geotechnical campaign

<table>
<thead>
<tr>
<th>Family</th>
<th>$P_l$ (MPa)</th>
<th>$E_m$ (MPa)</th>
<th>$E/E_m$ (MPa)</th>
<th>$\phi'$ (°)</th>
</tr>
</thead>
<tbody>
<tr>
<td>$F_1$</td>
<td>0.99</td>
<td>21.2</td>
<td>31.6</td>
<td>31.9°</td>
</tr>
<tr>
<td>$F_3$</td>
<td>0.42</td>
<td>5.1</td>
<td>7.6</td>
<td>27°</td>
</tr>
<tr>
<td>$F_4$</td>
<td>3.6</td>
<td>37.8</td>
<td>114.5</td>
<td>39.4°</td>
</tr>
<tr>
<td>$F_5$</td>
<td>0.31</td>
<td>2.46</td>
<td>12.1</td>
<td>25.2°</td>
</tr>
<tr>
<td>$F_6$</td>
<td>1.76</td>
<td>27.7</td>
<td>55.4</td>
<td>35.3°</td>
</tr>
<tr>
<td>$F_7$</td>
<td>0.85</td>
<td>14.7</td>
<td>21.9</td>
<td>31.1°</td>
</tr>
<tr>
<td>$F_8$</td>
<td>2.8</td>
<td>40.2</td>
<td>121.8</td>
<td>37.9°</td>
</tr>
</tbody>
</table>

6 DESIGN OF THE NORTH TRANSVERSAL TUNNEL

6.1 Choosing geotechnical parameter for the design

All the pressuremeter results, and mainly the elastic modulus $E$, and the internal angles of friction $\phi'$, were used in a statistical program to fit the mean value $X$, the standard deviation $\sigma_n$, the characteristic value $X_k$ over a total number N of tests. Characteristic value is calculated so that there is $\beta$ probability (equal to 5%) that experimental value would be smaller than $X_k$. Equation (8) is used with parameter $k_D$ equal to one, parameter $k_V$ equal to $\sqrt{5}$ (for large diameter tunnel), and parameter $k_N$ which is read in probability tables.

$$X_k = k_D \left( \bar{X} - k_N / k_V \cdot \sigma_n \right)$$  

The characteristics values are shown on Table (5).

6.2 Consequences on the tunnel design

The tunnel design is made with data from geotechnical campaign, and status of the town planning, which is sensitive to settlements. The longitudinal profile is designed, taking into account soil data, urban site and tunnelling construction (Chapeau & Schenwarzfeier, 1987).

Table 5. Characteristic values for geotechnical parameters

<table>
<thead>
<tr>
<th>Family</th>
<th>$P_l$ (MPa)</th>
<th>$E_m$ (MPa)</th>
<th>$E/E_m$ (MPa)</th>
<th>$\phi'$ (°)</th>
</tr>
</thead>
<tbody>
<tr>
<td>$F_1$</td>
<td>0.65</td>
<td>16</td>
<td>24.6</td>
<td>6.7</td>
</tr>
<tr>
<td>$F_3$</td>
<td>0.3</td>
<td>3</td>
<td>10</td>
<td>0.67</td>
</tr>
<tr>
<td>$F_4$</td>
<td>2.4</td>
<td>18</td>
<td>7.5</td>
<td>0.33</td>
</tr>
<tr>
<td>$F_5$</td>
<td>0.3</td>
<td>1.9</td>
<td>6.3</td>
<td>0.5</td>
</tr>
<tr>
<td>$F_6$</td>
<td>1.3</td>
<td>16</td>
<td>12.3</td>
<td>0.5</td>
</tr>
<tr>
<td>$F_7$</td>
<td>0.6</td>
<td>7.2</td>
<td>12</td>
<td>0.67</td>
</tr>
<tr>
<td>$F_8$</td>
<td>2.4</td>
<td>26.4</td>
<td>11</td>
<td>0.33</td>
</tr>
</tbody>
</table>

It can be seen on Table 4 that the elastic modulus $E$ of the soils $F_1, F_3, F_5, F_6, F_7$ are very low, and surface settlements could appear. To prevent this phenomenon, a mud pressure machine will construct the tunnel. The mud will be adapted to sands and gravels, and the risk of pressure breakdown will be reduced by the low value of permeability ($k < 10^{-3}$ m/s), the amount of fine sand, and the continuity of the soil, which is without voids.

The main difficulty of the machine design is its ability to drill sands and silt without surface displacements, and rock of Bastille district on this western part of the profile.

The tunnel lining will be made of concrete pre-cast segments. It can be seen on Table 5 that the characteristic angle of friction $\phi'$ is close to 30° and the machine should have safe support. The soil is pulverulent and under water table, so that the void between the tunnel machine and the soil must be filled by injection to limit surface settlements.

The seismic risk is pending but settlements will be controlled if pore pressure cannot dissipate. The large size of the tunnel, the proximity of the surface, and the high level of water table which bounds effective vertical stress will reduce this phenomenon.

Longitudinal profile is made of five different parts for the design (Figure 1), which are from East to West:

- **Tunnelling under Isère river (620+200m):** this part is located in the Sablon district. The access shaft used for tunnel machine positioning can be constructed into this non-urbanized area. Crossing through Isère river can be made by pre-built concrete boxes, which will be sunk under water in a long underwater excavation. This technique was not chosen, by the risk of Isère river floods, which digs the waterbed over several meters. The construction will be made by a tunnel machine with mud pressure, which has a precise connection to the next part of the profile.

- **Tunnelling under Ile Verte district (920m):** This part is without particular problem towards
the east, but surface constructions become higher towards the west, and a church with a slim structure founded on piles will be very sensitive to settlements.

- **Tunnelling through interface between soil and rock (1240m):** for soil, tunnel-drilling machine with mud pressure is used, but for rock classical drilling machine is preferred. The equipment must be adapted, and it is necessary to build temporary technical shaft for the access to the front of the machine.

- **Crossing Isère river on Porte de France district (860m):** this part is without special problem, with a bridge over the river. Tunnelling under Isère river is not adapted.

- **Crossing through water table under Presqu’île district (520m):** two rivers surround this area, Isère towards the north and Drac towards the south, with a groundwater that flows towards the northwest along the two rivers. As there is an impervious layer of soil 20m depth, the presence of walls inside the first 10m of the soil, will stop the water along the profile of the road. The coast of this solution is very high by hydraulic studies that are necessary, and additional constructions needed. Thus, the design is made with superficial constructions, which are above water table.

## 7 CONCLUSION

Geotechnical campaign for the north transversal tunnel in Grenoble was carried out by in situ tests and mainly by cyclic pressuremeter tests, which sheared intact soil, while triaxial tests were used to measure physical characteristics like interparticle angle of friction on remoulded samples.

Pressuremeter test allows fitting the internal angle of friction with specific management of the test.

The geotechnical analysis shows a variability of soil characteristics, which is taken into account by statistical analysis where 95% of measured values are higher than values used for the design.

## 8 REFERENCES


Clarke B.G., Gambin M., 1998, Pressuremeter testing in on-shore ground investigations, ISSMGE Committee TC16, 1st Int. Cong. on site Characterisation, Atlanta, Balkema.


French Standard NF P 94-110-1, 2000, Essai pressiométrique

Ménard, Partie 1 : Essai sans cycle, AFNOR

French Standard NF P 94-070, 1994, Essai à l’appareil triaxial de révolution, AFNOR


