ABSTRACT: Tunnelling through soils results in ground loss, causing surface settlements and transverse movements. When the tunnel drive passes below an existing structure, it is important to estimate the effects upon the structure. However, the greenfield deformations should not simply be imposed upon a structure, because the structure contributes to stiffening of the ground (Potts & Addenbrooke 1997). A computational three-dimensional soil-structure interaction analysis is required, to obtain detailed stress-deformation response to the tunnel construction on existing buildings (Burd et al. 2000). As an example, the case of the Cairo metro tunnel is analysed. The numerical simulations presented here take into account the physics of the problem: face support, over-cut and the conical shape of the machine, the injection of grout in the annular void and its consolidation. After a precise description of the principles of excavation process simulation, we analyse not only the effect of final settlements on the existing building but also the stresses induced in the structure during the passage of the TBM. To analyse the influence of the foundation type, two structures are studied, the first with pad foundations and the second with a raft foundation.

1 INTRODUCTION
The movements and settlements of the soil associated with slurry shield tunnelling depend on many control parameters of the tunnel boring machine. In order to limit these movements, the pilot may control the bentonite slurry pressure on the face, the value of the over-cut of the machine itself, and the injection of the grout in the annular void around the final tunnel.

Field observations may help to analyse the contribution of the different parameters to the soil movements around a tunnel, but meanwhile their forecast is still difficult and empirical. Two dimensional simulations by considering the concept of stress release or volume loss (Bennebarek et al. 1998) for every step of construction enable the measured movement considering the contribution of every parameter to be calculated approximately step by step. Nevertheless these approaches require the application of a coefficient similar to the "deconfinement coefficient" aiming to take into consideration the strongly three-dimensional nature of the problem. Because of the complexity of the parameter’s effect and their interactions, this corrective coefficient can be only determined by back-analysis based on experimental measurements.

In order to evaluate the influence of the choice of the control parameters of the tunnel boring machine on the magnitude of the settlements, a three-dimensional numerical simulation for the excavation is proposed aiming to take into account the physics of the problem: face support, over-cut and conical shape of the machine, the injection of grout in the annular void and its consolidation. These simulations are realised with the aid of the computer program FLAC 3D using an explicit finite difference scheme and developed by ITASCA.

After a precise description of the principles of simulation of the excavation process, a simplified procedure based on the physics of the problem was adopted. An example of calculation applied to the Cairo metro is presented and analysed. This tunnel has been excavated in a fine sand layer by means of a slurry shield and is situated under existing three-storey buildings.

The numerical simulations presented here take into account: face support, over-cut and the conical shape of the machine, the injection of grout in the annular void and its consolidation. After a precise description of the principles of excavation process simulation, the analyses of two three storey structures are presented, the first with pad foundations and the second with a raft foundation. The analyses contain not only the effect of final settlements on the existing buildings but also the stress induced in the structure during the passage of the TBM.
2 PRESENTATION OF THE STUDY SECTION

2.1 Experimental section

Line 2 of Cairo subway network is 18.5 km long with 5.9 km of tunnel bored by the slurry shield technique. The 9.8 m diameter tunnel runs through soils of relatively poor geotechnical characteristics corresponding to alluvial deposits from the Nile floods. A section (with the tunnel 25 m deep) has been used in this study to observe the influence on a surface structure (Figure 1).

2.2 Geotechnical characteristics of the site

Extensive in-situ and laboratory tests were performed to provide a description of the different geological formations encountered and geotechnical parameters. Table 1 summarises the data used in the project.

<table>
<thead>
<tr>
<th>Parameters</th>
<th>Soil 1</th>
<th>Soil 2</th>
<th>Soil 3</th>
</tr>
</thead>
<tbody>
<tr>
<td>Elastic Modulus (MPa)</td>
<td>6.6</td>
<td>90.0</td>
<td>120.0</td>
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<tr>
<td>Poisson's ratio</td>
<td>0.4</td>
<td>0.3</td>
<td>0.3</td>
</tr>
<tr>
<td>Cohesion (kPa)</td>
<td>0</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>Friction angle (°)</td>
<td>20</td>
<td>20</td>
<td>40</td>
</tr>
<tr>
<td>Density (kN/m³)</td>
<td>18</td>
<td>19.4</td>
<td>20.3</td>
</tr>
</tbody>
</table>

For the 3-D numerical model, an elastic perfectly plastic behaviour has been assumed with Mohr Coulomb criterion. The simulations have been performed under drained conditions. Earth coefficient at rest is derived from Jaky's equation $K_0 = 1 - \sin \varphi$.

3 NUMERICAL SIMULATIONS

3.1 Experimental simulations

In certain parts of the layout, surface settlements of 25 mm were recorded (Figure 2). For this reason, we tried to simulate in a realistic way the digging by a Tunnelling Boring Machine with a pressurized face whose surface settlement generated would be of this value.

3.2 Tunnel boring phases

According to the experimental work of Maghazi (2001), the observed subsurface movements due to the different phases of tunnel boring using a slurry shield technique exhibit cycles of deconfinement and reconfinement.

The different reloading and loading cycles are mainly due to over-cutting, shield conicity, grouting and the grout consolidation (Figure 3).

These observations led to the definition of numerical procedures accounting for the different phases in a three-dimensional approach.

Four excavation phases have been defined:
- Phase 1: The passage of the cutting face.
- Phase 2: The passage of the tunnelling machine.
- Phase 3: Annular void filling.
- Phase 4: Consolidation of the grout.

3.3 3-D analyses

The 3-D numerical procedure aims at modelling the different stages of the tunnel construction defined above. Figure 4 shows these stages.

The applied pressure at the face is 90 kPa at the tunnel axis level with a vertical gradient governed by the slurry density ($\gamma_{slurry} \approx 11$ kN/m³) as done by (Van Eekelen et al. 1997). Due to over-cutting, it is likely that the slurry propagates along the shield. A maximum distance of propagation of 1.5 m has been assumed.
Nevertheless a partial filling of the void due to over­
cutting by this slurry is not taken into account in the
analyses.

In the next phase, the soil converges towards the
shield (supposed to be infinitely rigid). A pressure of
230 kPa corresponding to the annular void grouting
is applied over a distance of 1.5 m behind the tail. Then consolidation of the grout occurs, the grout be­
ing assumed elastic (E=10 MPa, v=0.3 according to
edometer tests) and the tunnel lining rigid.

This complex loading configuration is imposed in
the three-dimensional numerical model for each exca­
vation step.

The first calculation step corresponds to the
installation of the initial stress state. Then the first
10.5 meters corresponding to the shield and, the
grouting injection at the back of the precast concrete
is imposed in one step. The following phases
4,30 correspond to advances of 0.75 meters. Thus the
loading scheme presented in Figure 4 translates to
0.75m in the Y-direction at each simulation step (see
fig. 9).

The numerical model (Figure 5) of dimensions
(X=115m, Y = 90m, Z = 35m) consists of 123 000
elements and allows an excavation length of 90 m
long, to reach a stationary state. For reasons of sym­
metry, only a half of the geometry is represented.

3.4 Building construction

During construction, due to the weak soil character­
istics under the building foundations, an improved
material (presumed elastic in the calculations) was
prepared at a distance of 5 meters all around the
building foundations and of a 3 meters thickness.

The numerical representation of an actual building
is simplified. The buildings are introduced in only
one phase and, only a column-floor system is taken
into account.

Columns are taken into account in Flac3D as beams
with elastic behaviour. Each node has six degrees of
freedom (three translations and three rotations). The
floors are represented by triangular shells, each node
having six degrees of freedom.

The structures are made up of three floors. Figure
6 presents the number of spans in the transverse
and longitudinal directions to the tunnel. According to
the longitudinal direction, the buildings are com­
posed of three spans with a column every 3,75 me­
ters. In the transverse direction, because of the same
symmetry conditions imposed for the soil mass mod­
eling, only one half of the building made up of nine
spans in total with a column every 4 meters are
represented.

The floors are placed at every 4 m height as shown
on Figures 7 and 8. Two types of foundations are
studied: pad foundations and a raft foundation. In the
two cases, in the numerical simulations the founda­
tions are modeled using shell elements:
- for the raft foundation, the dimensions are 1 meter
larger in each direction than the floors;
- for the pad foundations, each foundation has two
meters length on the transversal direction (X) and
1,75 meters length on the longitudinal direction (Y)
and is centred below the columns.
The three dimensional model for the building is composed of about 5000 structural elements.

The properties of the various structural elements of this idealized building are presented in Table 2. The connection between the ground and the structure is assumed to be fully frictional.

Table 2. Mechanical and geometrical characteristics of the square columns and floors.

<table>
<thead>
<tr>
<th>Parameters</th>
<th>Columns</th>
<th>Floors</th>
</tr>
</thead>
<tbody>
<tr>
<td>Behaviour</td>
<td>Elastic</td>
<td>Elastic</td>
</tr>
<tr>
<td>Elastic Modulus (GPa)</td>
<td>19.</td>
<td>19.</td>
</tr>
<tr>
<td>Poisson’s ratio</td>
<td>0.4</td>
<td>0.2</td>
</tr>
<tr>
<td>Thickness (m)</td>
<td>0.3</td>
<td></td>
</tr>
<tr>
<td>Side length (m)</td>
<td>0.35</td>
<td></td>
</tr>
<tr>
<td>Density (kN/m³)</td>
<td>25.</td>
<td>25.</td>
</tr>
</tbody>
</table>

3.5 Parametric study

Various calculations were carried out making it possible to highlight the influence of the tunnel excavation by a pressurized shield on the building.

Initially an analysis without a building was carried out to visualize the influence of the tunnel excavation on the soil mass. Then two other simulations were made by first installing the building, and after, digging the tunnel, the structure being founded on pad foundations or on a raft foundation.

4 RESULTS

4.1 Numerical calculations without building

Figure 9 presents surface settlements along the tunnel axis in the longitudinal direction. Maximum settlement is about 2.75 cm and is observed 15 m behind the face. The influence of the excavation begins about 15 m in front of the tunnel face.

Figure 9. Longitudinal surface settlement.

The transverse troughs presented on Figure 10 have a width of about 3.5 diameters. The three curves taken at various tunnel face excavation positions are identical, showing that the model length is sufficient.

Figure 10. Settlement trough.

Figure 11 presents horizontal strains at the surfaces induced by excavation. The classical compression zone near the tunnel axis, followed by an extension zone are clearly evident.

Figure 11. Horizontal surface strain (length bored = 45 meters).
These reference analyses will allow the influence of the two structures on the induced movements to be highlighted.

4.2 Influence of the building

4.2.1 Displacements

Figure 12 presents the settlement increases under the building foundations. For an excavation length of about 45 meters (middle of the structure) we can note very little influence. On the other hand as soon as the gallery passes beneath the structure, we can observe a settlement increase of 0.3 cm. This influence is visible for ten meters on each side of the building position.

Figure 12. Longitudinal surface settlement

Figure 13 shows the vertical displacement field under the building for the case of the pad foundations: the settlements under the foundations and on the dotted line situated on the central axis of the building are almost the same. Thus for further comparisons, the value of the settlements calculated on the central axis will be adopted.

Figure 13. Surface settlements under the building with pad foundations.

Concerning the transverse settlement troughs (Figure 14), it appears that the building slightly stiffens the soil-structure system, resulting in less hogging curvature. Furthermore, there is no noticeable difference between the case of the building with a foundation raft or with pad foundations.

With respect to the horizontal displacements (Figure 15), it is evident that the presence of the structure reduces these displacements and so also induces lower horizontal strains than without the structure. For the raft foundations, the horizontal displacements are almost eliminated but the pad foundations reduce the horizontal displacements by an half compared to the case without a structure.

4.2.2 Forces in the beams

Figure 16 presents the maximum loads obtained in the columns for each of the X, Y, Z directions (longitudinal, transversal and axial). The loads obtained in the columns at the end of the structure’s installation are taken as reference, thus it is the ratio between the loads of the reference phase (known as initial) and those noted for various excavation lengths that are shown.

The axial load is very few affected by the excavation (∼3%) for either of the two types of foundations (Figures 16 and 17).

On the other hand there is a very clear influence of the tunnel excavation on the transverse and on the longitudinal shear force. For the transverse shear force, there is an increase of about 750 % for the case of a raft foundation and of about 250% in the
case of the pad foundations. These values correspond to the excavation phase under the building and the passage of the shield which are the most critical. When the Tunnel Boring Machine moves away, the shear force remains high, decreasing to only 600% of the initial transversal shear force for the raft foundation.

The behaviour in terms of longitudinal shear force is very different for the two foundations types. In the case of the raft foundation, it decreases to a value representing 20% of its initial value at the time of the TBM passage then after the value goes to 180% of its initial value. For the case of the pad foundations, the longitudinal shear force increases to 170% of the initial value and decreases to 70%.

It can be noted that the relative increase on shear forces are lower on the structure with pad foundations than on that with raft foundation. This can be explained by the fact that the initial forces in the structure with pad foundations are much higher. Indeed the initial shear forces for the structure with pad foundations are twenty times greater in absolute values than the case with raft foundation (in terms of longitudinal shear force: 14kN against 0.7kN).

Thus these two cases cannot be directly compared, The marked difference is probably due to the fact that the simulations do not take into account the construction process of the building.

5 CONCLUSIONS
With a relatively rigid building, a very clear reduction in the strain of the foundation is observed when compared with those obtained in greenfield. This fact shows that traditional approaches considerably overpredict building strains and consequently damage. Nevertheless, the increase in transverse shear force appears to be extremely significant in certain structural elements.

The case with a pad foundation seems to be the case which generate greater loads in the structure. Therefore it is important to respect in a numerical modelling the real interaction between the soil mass and the structure.

A more detailed analysis of these results is now needed to permit a comparison with a calculation based on usual assumptions. We also plan to simplify the excavation procedure by using the concept of ground loss, which would make reducing the calculation times possible.

REFERENCES


