Cap-Yield model with cohesion, back analysis of real excavations

A. Lucarelli, G. Guiducci & G. Furlani  
*Studio Sintesi, Rimini, Italy*

R. Sorge  
*Metro C S.p.A., Italy*

ABSTRACT: Cap-Yield (CY) is a powerful and flexible built-in *FLAC* soil constitutive model that has very interesting features especially for excavation problems. In this paper, two examples of back analysis of real excavations executed for the new Metro C line in Rome are presented. After a brief description of general geotechnical site conditions and monitoring system, results are presented with model parameters calibration discussion.

1 CONSTITUTIVE MODEL SHORT DESCRIPTION

Both examples have been developed adopting the CYsoil model with cohesion. In this section, a short description of the essential model features is presented. For most soils, the plot of deviatoric stress versus axial strain obtained in drained triaxial test may be approximated by hyperbolic law. When material also exhibits a cohesive component, a pseudo-linear first branch followed by the hardening frictional segment is frequently observed (see Fig. 1 for example).

![Figure 1. Material with cohesive behavior followed by frictional hardening (TXCU carried out on volcanic material – soil investigation report for T4 line – Metro C).](image)

In order to be able to capture the hyperbolic behavior, CYsoil model is supplemented by friction/strain-hardening table. Shear Yielding is defined by Mohr-Coulomb failure envelope with cohesion. By means of a cap hardening power law, the non linear volumetric behavior observed in isotropic compression test can be reproduced. Moreover, with a compaction/dilation law, the irrecoverable volumetric strain taking place in monotonic soil shearing can be modeled. For more detail concerning Cap-Yield Model description and customization, see Section 2.4.9 of the *FLAC* (Itasca 2008) Theory and Background manual.
2 EXAMPLE 1: TBM STARTING PIT

2.1 General

Figure 2 shows the typical section; TBM Pit has a rectangular planimetric shape with inside dimensions of 26×96 m. Since the inside area must be completely free, the support structure has been realized with continuous alignments of T shaped diaphragm elements of 43.60 m length.

Concrete Slab/Strut at capping beam elevation is the only structural constrain (thickness = 1.80 m). Inner excavation high is around 23.2 m with a maximum around 25.0 m. Water level is positioned at +24.00 m; bottom excavation level is located at +9.90 m.

2.2 Geotechnical conditions

The soil profile consists of typical Roman volcanic formations, from ground level (around +35.00-40.00) down to around -3.00 elevation; below, plio-pleistocene sea and continental deposits made the base formation. Inside the volcanic soil, two main portions can be clearly highlighted with separation around +15.00 m: the upper, is black-red silty-sand piroclastic material; the lower is well-cemented piroclastic silt with layer of tuff. The latter has the characteristics of a soft rock. Figures 3 & 4 show examples of upper and lower volcanic material, respectively (source: soil investigation report for line T4 – Metro C). Base soil is essentially made of intercalation of firm clay-silt, sandy-silt followed with very dense sandy-gravel. As far as the soil strength is concerned, several shear tests have been carried out. For deformability, the Shear Wave velocity profile is available. The main results are summarized in Table 1. Porosity has been assumed equal to 0.2 for all materials.

<table>
<thead>
<tr>
<th>Properties</th>
<th>MG</th>
<th>VS</th>
<th>PN/PR</th>
<th>TAT1T2</th>
<th>Sta</th>
<th>ST</th>
<th>SG</th>
</tr>
</thead>
<tbody>
<tr>
<td>Dry Density [Mg/m³]</td>
<td>1.70</td>
<td>1.70</td>
<td>1.70</td>
<td>1.70</td>
<td>1.80</td>
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<td>Effective Cohesion [kPa]</td>
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<td>15-10</td>
<td>5-10</td>
<td>30-40</td>
<td>20-30</td>
<td>2-10</td>
<td>5</td>
</tr>
<tr>
<td>Dilation angle [degree]</td>
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<td>5</td>
<td>5-8</td>
<td>4-6</td>
<td>0-2</td>
<td>5</td>
<td>4-6</td>
</tr>
<tr>
<td>Ave. Shear Wave vel. [m/s]</td>
<td>240</td>
<td>470</td>
<td>550</td>
<td>650</td>
<td>420</td>
<td>530</td>
<td>700</td>
</tr>
<tr>
<td>G₀ [MPa]</td>
<td>100</td>
<td>390</td>
<td>530</td>
<td>810</td>
<td>290</td>
<td>520</td>
<td>980</td>
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<tr>
<td>E₀ [MPa]</td>
<td>260</td>
<td>1015</td>
<td>1220</td>
<td>2100</td>
<td>670</td>
<td>1355</td>
<td>2550</td>
</tr>
</tbody>
</table>

MG, Made Ground; VS, VillaSenni, upper volcanic soil; PN/PR Black-red piroclastic material, upper volcanic soil; TAT1T2 lower piroclastic cemented soil with tuff layer; Sta, ST SG Base plio-pleistocene material.
2.3 Hydro-geological conditions

Permeability distribution is quite complex and in-situ Lefranc tests have shown great data dispersion. In addition, generally speaking, anisotropy might play an important role in these kinds of situations. The presence of clay layers at the top of Plio-Pleistocene soil (sub unit Sta – Fig. 2) provide a kind of hydraulic separation between volcanic formations and Pleistocene material establishing a dominant horizontal flow that involves essentially the most permeable portion of the last. Since water pressure distribution plays an important role for both soil strength and structural load, average soils permeability will be back-calculated from average discharge and the piezometers pore pressure measured during excavation.

2.4 Monitoring system description and main results

Three main instrumented sections have been set up for several physical parameters control, like displacements, water pressure, and strains. The following Figure 5 shows the position of control sections. The red arrow shows the position of one of the inclinometer inside the diaphragm element for horizontal displacement control that provides the most reliable observations: this inclinometer data will be used for back-analysis. Reading refers to a period of around 8 month during bottom slab construction; horizontal displacements have maximum values of 16-18 mm around +15.00 m el.

Pore pressure distribution has been controlled with several piezometers placed both externally and internally of excavation area. Those external have been placed at two different levels in order to be able to observe the separation effects provided by impervious clay layers. The upper Casagrande cell has been placed at +15.00 m el., inside the volcanic formations while the lower has been placed at -10.60 m el. inside the Pleistocene sandy gravel (close to wall diaphragm wall toe el., at four meters distance). As expected, upper water level did not change significantly during excavation while the lower shows a drop of 6-8 m. Moreover, daily total discharge from
inside wells has been around 570 m$^3$/day on average (Figure 6).

While upper piezometer did not show a significant level change, the lower has drop around 8 m.

Figure 5. Plan view with positions of monitoring instrumentation.
2.5 Back Analysis description

The following main modeling stages have been considered:
- Stage 1: Initialize stress state considering non horizontal ground level and water table at +24.00 m elevation;
- Stage 2: Application of CY soil parameters;
- Stage 3: Upper soldier piles wall activation;
- Stage 4: Excavation down to +33.20 m elevation for diaphragm wall execution; in order to reduce sudden change effects from big unbalance force, the effect of excavated material has been gradually applied to the surrounding model;
- Stage 5: Diaphragm wall activation, unbounded the interfaces ($c'_{int} = 0.0$ $\varphi'_{int} = 30^\circ$); top slab activation considering pin connection with lateral wall;
- Stage 6: Ground water analysis imposing boundary condition for having water level inside excavation at +7.90 elevation (bottom excavation level +9.90 m – see Figure 4);
- Stage 7: Excavation down to bottom level elevation +9.90 relaxing very gradually unbalanced force.

2.5.1 Material Properties

Table 2 summarizes the CY soil properties that have given the best numerical results compared to experimental data. For others model parameter a common value for all soil has been considered (Table 3). The linear elastic properties for diaphragm wall and top strut are given in Table 4.

Table 2. Hardening CY soil properties.

<table>
<thead>
<tr>
<th>Properties</th>
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<td>1.80</td>
<td>1.85</td>
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<td>Ultimate Friction angle [degree]</td>
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<td>30</td>
<td>35</td>
<td>35</td>
<td>26</td>
<td>35</td>
<td>35</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Ultimate Dilation angle [degree]</td>
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<td>5</td>
<td>5</td>
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<tr>
<td>Cohesion [kPa]</td>
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<td>5</td>
<td>35</td>
<td>20</td>
<td>5</td>
<td>5</td>
<td></td>
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</tr>
</tbody>
</table>
The main results relative to Stage 6 (dewatering) and 7 (final excavation level) are presented. Dewatering has been simulated with a stationary seepage process (setting off mechanical calculation—uncoupled analysis). Internal water level has been imposed 2.0 m below the bottom excavation level. Seepage analysis has been conducted searching the permeability value that provides: a) inflow equal to measured average discharge; b) pore pressure distribution similar to that observed in the piezometer outside excavation. The following shows the results: inflow match average discharge; pore pressure, at the lower Casagrande cell, is very similar to measured value. The permeability that has provided the best result is $k_{xx} = k_{yy} = 9.56 \times 10^{-5}$ m/s. Finally, Figure 8, shows the horizontal displacements comparison with the mobilized friction angle: ultimate friction has probably been reached at bottom excavation.
3 EXAMPLE 2: ARTIFICIAL TUNNEL

The second example is related to an artificial tunnel realized with discontinues piles walls and top slab. Lateral piles have a diameter of 0.60 m posed at center-to-center distance of 0.80 m. Top concrete slab has a thickness of 1.0 m. The tunnel falls almost exclusively in Villasenni tuff. In this case, water is not present. Some of the piles have been instrumented with inclinome-
For this case has been necessary to develop some considerations regarding the structural piles response. In particular, non-linear concrete behavior with change in flexural stiffness after crack opening has been taken into account in a simplified manner inside FLAC model reducing concrete Young modulus for pile element in connection with top slab. In addition, a plastic hinge has been applied with a plastic moment of 150 kNm that correspond to cracking moment with an axial load of 200 kN for the pile section (dead load due to slab self weight and filling material). CEB/FIB Model Code For Concrete Structures (1990) has been applied for structural equivalent bending stiffness analysis. After cracking, the tension stiffening effects have been taking into account via a smeared crack approach. Plastic hinge allow for relative rotation between the pile and slab node, after the plastic moment has been reached: observing the inclinometer rotation at pile head is evident that the pile-slab connection is semi-rigid. Figure 10 summarizes the evaluation of the secant bending stiffness applied the pile-slab connection element.

As far as the soil parameters are concerned, the same values adopted for Example 1 and described in Table 2 have been applied to Villasenni tuff. FLAC grid has been developed taking advantages of the problem symmetry and the sequence construction phases modeled are:

- geostatic stress initialization;
- application of CYsoil parameter;
- local excavation for piles and slab execution;
- pile, slab activation;
- refilling above top slab;
- inside excavation

Figure 11 shows the horizontal displacements comparison between inclinometer and numerical back-analysis.

Figure 9. Example 2: structural layout, FLAC grid, monitored displacements.
4 REMARKS

The best results have been obtained considering $E_{\text{ref}}/1/3-1/5$ of small-strain $E_0$ modulus and $K_0$ around $1/4$ of $E_{\text{ref}}/1/3$. As far as the relation between shape parameter $\alpha$ and $K_0$ is concerned, for the assigned $K_0$ values (see table 2) the best results have been obtained considering $\alpha$ equal to 1.
REFERENCES


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