Behaviour of an underground metro station for different constitutive models - A study on the influence of Mohr-Coulomb and other advanced constitutive models using PLAXIS 2D for the proposed Chennai Metro Rail Phase 2 project

Kumar Pitchumani N1, Ganesh Deepak Kanchuboyina2, Anjana R K3, Sai Krishna Bugga4

1Regional Director, AECOM Chennai, India
2Senior Geotechnical Engineer, AECOM Chennai, India
3Geotechnical Engineer II, AECOM Chennai, India
4Senior Structural Engineer, CEG Chennai, India

Email ID: NKumar.Pitchumani@aecom.com

Abstract. The Phase 2 of the proposed Chennai Metro Rail Project contains underground stations of depths below ground varying from 20m for a 2 level station to 32m for a 4 level station. Diaphragm walls have been chosen for the purpose of earth retention during construction stage and also to be a part of the permanent structure. The centre part of the stations is constructed using top-down methodology and the ends of the station which are used for the purpose of TBM launching and retrieval activities are constructed using either bottom-up or top-down methodologies. The behaviour of the part of the station constructed using top down sequence will be different from the part constructed using bottom up sequence. The current study focusses on the comparison of the influence of the various constitutive models available in the FEM software package PLAXIS 2D such as the basic Mohr-Coulomb model and advanced models such as Hardening soil model and Hardening soil small strain stiffness (HSS) models on the bending moments, shear forces of the diaphragm walls, ground deformations, strut forces etc. This comparative study has been performed on both the top-down and bottom-up sequences and for the different ground conditions along the proposed second phase metro rail alignment in Chennai region of India.

1. Introduction
Chennai city, the fourth largest metropolis in India, is the focus of economic, social and cultural development and it is the capital of the State of Tamil Nadu. The city has a multi-dimensional growth in development of its infrastructures and population. Chennai Metro Rail Limited (CMRL) is introducing an efficient, safe and high capacity transport system of urban metro rail in Chennai. The Phase-2 of Chennai Metro is divided into three corridors i.e. corridor-3, corridor-4 & corridor-5. The Phase-2 metro system is a mix of underground and elevated stretches. The underground stretch will be constructed beneath densely populated urban areas.

Chennai is located on Eastern Coastal Plains - flat plains, on the southeast coast of India and towards the northeast of Tamil Nadu. Its average elevation is around 6.7 meters (22 ft) whereas its highest point is 60 m (200 ft). The shoreline of Marina Beach is spread over 12 km along the city’s shoreline.
river Cooum divides the city into half whereas the river Adyar divides the southern half into two parts trisecting the city of Chennai. These rivers flow towards the East. A third river, the Kortalaiyar, flows through the northern peripheries of the city [1].

1.1. Regional geology

The geology of Chennai comprises of sand deposits, clay, granite, gneiss and traces of shale and sandstone. The city is classified into three regions based on geology i.e. sandy areas, clayey areas and hard-rock areas. Sandy areas are found along the river banks and the coasts. Clayey regions cover most of the city. Hard rock areas are found in some central parts and south of the city [2]. Chennai district forms part of coastal plains of Tamil Nadu. Major part of the district is having flat topography with very gentle slope towards east. Fluvial, marine and erosional landforms are noticed in the district. Marine transgression and regressions and neo-tectonic activity during the recent past have influenced the morphology and resulted in various present landforms. Meandering streams with small sand bars are present along the course of Adyar river [3]. The geology of Chennai is shown in Figure 1.

![Figure 1. Geology of study area: Chennai city and southern part [4].](image)

2. Overview of constitutive models for soils

The process of geotechnical design of underground constructions has a unique dependence on numerical modelling of the ground conditions. The behavior of soils can be modelled with varying degrees of accuracy.

The most basic Mohr-Coulomb (MC) soil model provides a crude approximation of the soil behavior. The linearly elastic perfectly-plastic soil model requires the input of a constant soil stiffness property, or a linearly increasing soil stiffness with respect to depth. Due to the constant stiffness (independent of stress/strain in soil), model computations are relatively fast.

The more advanced Hardening soil (HS) model provides an improved simulation of soil behavior. The soil stiffness is described with three input stiffnesses: the triaxial loading stiffness, $E_{50}$, the triaxial unloading stiffness, $E_{uu}$, and the oedometer loading stiffness, $E_{oed}$, related to a reference stress. This soil model simulates stress dependency of the stiffness moduli. In addition, parameter $m$ is used to define the stress-level dependency of stiffness.

The Hardening soil with small-strain stiffness (HSSmall) is a modification of the HS model that accounts for an increased stiffness of the soil at smaller strains. Two additional material properties are used to define this model: $G_{0}^{ref}$, the small-strain shear modulus and $\gamma_{0.7}$, the strain level at which shear modulus has reduced to 70% of the original small-strain shear modulus.

In this study, Plaxis 2D has been selected as a tool for numerical analyses. Three constitutive soil models, i.e. MC model, HS model, HSS model, were adopted to simulate the soil layers for evaluating their performances in predicting the wall forces, deflections and surface settlements induced by the construction of the station. In the models, strength parameters are the same, but stiffness parameters are different. The stiffness parameters are constant in the MC model, while they are stress dependent in the HS model and HSS models, and are strain as well as stress dependent in the HSS model.
3. Parameters for MC/HS/HSS models

The ground conditions prevailing at the underground construction sites of corridor 3 Phase 2 metro line has been standardized into two ground profiles namely GC1 (Good to medium ground conditions) and GC2 (Poor ground conditions).

The stiffness parameters for MC model is based on SPT N correlations. The stiffness moduli for HS model have been considered as follows:

\[ E_{50} = E_{oed} = E' \] (\( E' \) is the stiffness used in the MC model)
\[ E_{so} = 3E_{50} \]

**Stress-level dependency of stiffness, \( m = 0.45 + 0.003D_r \) (%) based on Teo and Wong [5]**

The stiffness moduli for HSS model have been considered to be the same as HS model. However, the small-strain shear modulus and the threshold shear strain is considered as below:

\[ G_{0r}^{ref} = 15.09 [(N_1)_{60}]^{0.74} \text{ MPa based on Anbazhagan et al. [6]} \]

**Shear strain at which \( G_s = 0.7G_{0r} \), \( \gamma_{0.7} = 10^4 \)**

The ground parameters for GC1 conditions are shown in Table 1 below.

### Table 1. Parameters for GC 1 (Good to medium ground conditions).

| Depth (m) | Strata | SPT N60 | SPT (N1)60 | \( C' \) (kPa) | \( \gamma_{90} \) (kN/m) | \( e' \) (kPa) | \( \phi' \) (˚) | \( K_o \) | \( E_{rm} \) (MPa) | MC | HS | HSS |
|-----------|--------|---------|------------|----------------|-------------------------|---------------|---------------|--------|----------------|-----|-----|-----|
| 0 -10 SM  | 15 20 18 | 0 31 0.3 0.48 - | 23 23 23 69 | 0.55 | 138.5 | 10^4 |
| -10 -14.5 SC | 15 15 18 | 0 30 0.3 50 - | 23 23 23 69 | 0.55 | 111.9 | 10^4 |
| -14.5 -17 SM | 32 30 20 | 0 34 0.3 44 - | 48 48 48 144 | 0.65 | 186.9 | 10^4 |
| -17 -21 SM | 72 70.4 20 | 0 38 0.3 38 - | 108 108 108 324 | 0.7 | 349.9 | 10^4 |
| -21 -31.5 Sandstone Grade (V) | - - | 20 59 44 0.3 50 209 | - - - - - - - - |
| -31.5 -40 Sandstone Grade (III) | - - | 24 191 60 0.2 50 1639 | - - - - - - - - |

*The engineering parameters are estimated using correlations from SPT values and the rock parameters are converted from Hoek and brown parameters to Mohr Coulomb parameters

The ground parameters for GC2 conditions are shown in Table 2 below.

### Table 2. Parameters for GC 2 (Poor ground conditions).

| Depth (m) | Strata | SPT N60 | SPT (N1)60 | \( C' \) (kPa) | \( \gamma_{90} \) (kN/m) | \( e' \) (kPa) | \( \phi' \) (˚) | \( K_o \) | \( E_{rm} \) (MPa) | MC | HS | HSS |
|-----------|--------|---------|------------|----------------|-------------------------|---------------|---------------|--------|----------------|-----|-----|-----|
| 0 -11 SM | 15 20 18 | 0 31 0.3 0.48 - | 23 23 23 69 | 0.55 | 138.5 | 10^4 |
| -11 -21 SM | 30 25 20 | 0 33 0.3 46 - | 45 45 45 135 | 0.55 | 163.4 | 10^4 |
| -21 -24 SM | 45 54 20 | 0 38 0.3 38 - | 68 68 68 204 | 0.65 | 288.8 | 10^4 |
| -24 -40 Shale Grade (V) | - - | 20 36 30 0.3 50 114 | - - - - - - - - |

*The engineering parameters are estimated using correlations from SPT values and the rock parameters are converted from Hoek and brown parameters to Mohr Coulomb parameters
4. **Typical station geometry and construction sequence**

The typical 2 level station plan is shown in Figure 2. The station boundary is retained by reinforced concrete diaphragm walls. The centre part of the station (typically 16m wide) is constructed using top-down construction methodology, while the ends of the station (TBM launching/retrieval shafts, typically 20m wide) are constructed using bottom-up/top down construction methodology as shown in Figure 2.

![Figure 2. Typical station geometry.](image)

A typical cross section of the station section is shown in Figure 3 below. The cross section shown below is the centre part of the station constructed using top-down sequence. The paper focusses on 2 level stations (depth of excavation is typically 20m). The depth of embedment of the diaphragm wall for 2-level stations is typically 5-6m depending on the ground conditions.

![Figure 3. Typical station cross-section.](image)
4.1. Construction sequences
Top down stations are supported using temporary 2 levels of normal steel strutting system and with permanent slabs during excavation. Bottom-up launching/retrieval shafts are supported using temporary 4 levels of diagonal steel strutting system during excavation and permanent slabs during long term. Typically, the construction sequence is adopted as below:

| Table 3. Construction sequence. |
|----------------------------------|
| **Top-down construction sequence** | **Bottom-up construction sequence** |
| - Application of surcharge (construction surcharge, building surcharge) | - Application of surcharge (construction surcharge, building surcharge) |
| - Installation of diaphragm wall | - Installation of diaphragm wall |
| - Excavation to strut level 1 | - Excavation to strut level 1 |
| - Installation of strut level 1 | - Installation of strut level 1 |
| - Excavation to roof slab | - Excavation to strut level 2 |
| - Casting of roof slab | - Installation of strut level 2 |
| - Excavation to concourse slab | - Excavation to strut level 3 |
| - Casting of concourse slab | - Installation of strut level 3 |
| - Excavation to strut level 2 | - Excavation to strut level 4 |
| - Installation of strut level 2 | - Installation of strut level 4 |
| - Excavation to final excavation level | - Excavation to final excavation level |
| - Casting of base slab | - Casting of base slab |
| - Removal of struts and Backfilling | - Removal of strut level 4 |
| - Restoration of ground water | - Casting of concourse slab |
| - Long-term service condition (modelled with reduced stiffness of the structural elements) | - Removal of strut level 2 and 3 |
| | - Casting of roof slab |
| | - Removal of strut level 1 and Backfilling |
| | - Restoration of ground water |
| | - Long-term service condition |

4.2. Design considerations
Finite element analyses have been carried out using the software program PLAXIS 2D. D-wall, columns, roof slab, concourse slab and base slab are modelled as plate elements. The elastic modulus (E) for the plate elements are calculated considering concrete grade M40 for D-wall and M35 for Slabs. Concrete strength reduction of 20% is assumed for D-wall to account for the tremie concreting effect. The temporary steel struts to take the axial load from the soil are modelled as an anchor element. Other general design considerations are listed below:

| Table 4. Design considerations. |
|----------------------------------|
| **Parameter** | **Considerations** |
| **Ground water table** | - Full water table at ground level has been considered in the analyses behind the diaphragm wall |
| | - The water table is modelled 1m below the excavated level for the excavation side |
| **Strut stiffness** | **Top-down** (Spacing = 8m) |
| | - The first level of strut (Deck strut) with axial stiffness of 20.86 x 10^6 kN |
| | - The second level universal beam (UB) strut with axial stiffness 6.36 x 10^6 kN |
| **Bottom-up** (Spacing = 3m) | - All four level struts are diagonal struts with axial stiffness varying from 2.24 x 10^6 to 6.74 x 10^6 kN |
| **Surcharge construction loads** | - Uniform surcharge of 20kPa for 2m width is directly applied on the ground level |
4.3. Material properties

The typical material properties for the reinforced concrete structural elements are listed below.

| Parameter                  | Considerations                                                                                                                                 |
|----------------------------|-----------------------------------------------------------------------------------------------------------------------------------------------|
| **Surcharge**              | - Uniform surcharge of 50kPa with minimum 20m width behind the excavation has been applied on the active side (1.5m below ground level) of the retaining system |
| **Vehicular loads**        | - Minimum 20 kPa surcharge above station box and entrances if any, is applied after backfilling to ground level                                 |

### Table 5. Material properties for diaphragm wall and slabs.

| Parameter                  | Value                  | Slab | EA (kN/m) | EI* (kNm²/m) |
|----------------------------|------------------------|------|-----------|--------------|
| Axial stiffness, \( EA \)  | 28.3 x 10⁶ kN/m        | Roof slab | 35.5 x 10⁶ | 4.26 x 10⁶   |
| Short-term flexural rigidity, \( 0.7EI \) | 1.65 x 10⁶ kNm²/m | Concours slab | 26.6 x 10⁶ | 1.8 x 10⁶   |
| Long-term flexural rigidity, \( 0.5EI \) | 1.18 x 10⁶ kNm²/m | Base slab | 38.5 x 10⁶ | 5.4 x 10⁶   |
| Actual wall Thickness, \( d \) | 1 m                    |      |            |              |

*\( EI \) is reduced to 0.5\( EI \) for long-term service condition

5. Results and discussion

5.1. Top-down construction methodology

The centre part of a typical station is constructed using top-down construction methodology. A cross section of the centre part has been modelled for both GC1 and GC2 ground conditions with MC, HS and HSS constitutive models. The bending moments and shear forces in the diaphragm wall have been presented below in Figure 4 (a) and (b) respectively.
It can be noted from Figure 4 that the change in the bending moments/shear forces is minimal with the change in the constitutive model. The deflections in the diaphragm wall and the ground settlements in the station with top-down construction have been presented below in Figure 5 (a) and (b) respectively.

![Deflections and Ground movements](image)

**Figure 5.** Top-down – Deflections and Ground movements.

The analysis suggests that there is a significant decrease in the magnitude of the wall deflections with advanced soil models, with nearly 20% decrease noted. The change in magnitude is noted in both GC1 and GC2 ground conditions. In terms of ground settlements, there is a 20% decrease noted in the GC1 settlements with advanced soil models, while GC2 settlements do not show any significant variation. The comparison of strut forces is shown in Table 6.

| Strut | Depth (mbgl) | Section | MC Force (kN) | HS Force (kN) | HSS Force (kN) |
|-------|-------------|---------|---------------|---------------|----------------|
| G1 S1 | 1           | Decking Strut (Area = 1038cm²) | 2013 | 1938 | 1775 |
| G1 S2 | 15.6        | UB 610x229x125.1 | 2942 | 2786 | 2681 |
| G2 S1 | 1           | Decking Strut (Area = 1038cm²) | 2097 | 2038 | 1958 |
| G2 S2 | 15.6        | UB 610x229x125.1 | 4219 | 4715 | 4827 |

The results in the above table indicate minimal change in the top strut forces (S1) and a variation of ± 10 to 15% variation in the mid-strut (S2) forces.

5.2. **Bottom-up construction methodology**

The launching/retrieval shafts at the end of a typical station are constructed using bottom-up/top-down construction methodology. A cross section with bottom-up methodology has been modelled for both GC1 and GC2 ground conditions with MC, HS and HSS constitutive models. The bending moments and shear forces in the diaphragm wall have been presented below in Figure 6 (a) and (b) respectively.
It can be noted from Figure 6 that the change in the bending moments/shear forces is minimal with the change in the constitutive model. The deflections in the diaphragm wall and the ground settlements in the station with top-down construction have been presented below in Figure 6 (a) and (b) respectively.

**Figure 7.** Top-down – Deflections and Ground movements.
The analysis suggests that there is a decrease in the magnitude of the deflections with advanced soil models, with nearly 10% decrease noted. The change in magnitude is noted in both GC1 and GC2 ground conditions. In terms of ground settlements, there is a 30% decrease noted in the GC1 settlements with advanced soil models, while a 20% decrease in GC2 settlements is evident. The comparison of strut forces is shown in Table 7.

### Table 7. Table of strut forces.

| Strut | Depth (mbgl) | Section | MC  | HS  | HSS |
|-------|--------------|---------|-----|-----|-----|
|       |              | Force (kN) | Force (kN) | Force (kN) |
| GC1   | S1 0.60      | UB 610x229x125.1 | 1001 | 988 | 923 |
|       | S2 5.10      | UB 610x229x125.1 | 1712 | 1818 | 1844 |
|       | S3 8.50      | UB 610x229x125.1 | 3769 | 4033 | 4103 |
|       | S4 14.10     | UB 610x229x125.1 | 2463 | 2466 | 2390 |
| GC2   | S1 0.60      | UB 610x229x125.1 | 779  | 795 | 733 |
|       | S2 5.10      | UB 610x229x125.1 | 1747 | 1893 | 1855 |
|       | S3 8.50      | UB 610x229x125.1 | 5212 | 5299 | 5274 |
|       | S4 14.10     | UB 610x229x125.1 | 3525 | 3839 | 3861 |

The results in the above table indicate minimal change in the top strut (S1) and second strut (S2) forces and a variation of ± 5 to 10% variation in the bottom strut (S3 and S4) forces.

6. **Concluding Remarks**

The comparative study has been performed on both the top-down and bottom-up sequences and for the different ground conditions along the proposed second phase metro rail alignment in Chennai region of India.

It can be noted that the forces in the diaphragm wall as well as the strut forces for both top-down and bottom-up sequences vary marginally with the use of advanced soil constitutive models. The structural design of the diaphragm wall and struts will not yield any significant changes with advance soil models.

There is a decrease in the deflection of the diaphragm wall with the use of advanced soil models, the reduction being more prominent in top-down sequence. The ground settlements near the diaphragm wall show a decrease in the case of advanced soil models. The reduction in the top-down sequence is dependent on the ground conditions, while the reduction in the bottom-up sequence is similar in both types of ground conditions considered.

The conclusion drawn from the face of the study is that there is a significant variation in the wall deflections and ground settlements in the models with advanced soil-constitutive models, which is attributed to the stress-dependent and strain-dependent stiffnesses adopted in the HS and HSS models. However, the use of advanced soil models need to be validated with the recorded forces and deflections at site before drawing concrete inferences from the study presented in this paper.

**References**

[1] Ministry of Water Resources, River Development & Ganga Rejuvenation, Central Ground Water Board, Government of India 2017 *Report On Aquifer Mapping And Aquifer Management Plan For The Chennai Aquifer System, Tamilnadu*.

[2] Geological Survey of India, Government of India 1998 *Explanatory Brochure on Geological and Mineral Map of Tamilnadu and Pondicherry*.

[3] Balakrishnan T. Ministry of Water Resources, Central Ground Water Board, Government of India 2008 *District Groundwater Brochure Chennai District - Tamil Nadu*. 

9
[4] Geological Survey of India, Government of India 2005 District Resource Map - Geology, Chennai District, Tamil Nadu.

[5] Teo P L and Wong K S 2012 Application of the Hardening Soil model in deep excavation analysis *The IES Journal Part A: Civil & Structural Engineering* 5 (3) 152-165.

[6] Anbazhagan P, Aditya Parihar, Rashmi H N 2012 Review of correlations between SPT N and shear modulus: A new correlation applicable to any region *Soil Dynamics and Earthquake Engineering* 36 52-69.