# Finite Element Analyses in Geotechnical Engineering – *Part 1: An Indispensable Tool or a Mysterious Black Box?*

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## 1.0 INTRODUCTION

The usage of finite element analyses (FEA) in recent years has increased tremendously as a result of (i) increasing number of major geotechnical projects that need to be constructed close to existing buildings thus causing some form of soil-structure interaction which is difficult to analyse using empirical hand calculations or 1-D software and (ii) advancement of computing prowess with even home computers nowadays are capable of running simulated geotechnical analyses in a time frame not even dared to dream of some 10 years ago. With the widespread use of commercial FEA packages such as PLAXIS, SAGE-CRISP, SEEP/W, SIGMA/W, etc., prudent understanding of geomechanics principles is of fundamental importance so as to minimize computational errors as the consequences can be disastrous. This is compounded by the fact that in geotechnical engineering problems, usually it is very difficult to tell if the output from an FEA analysis is completely reasonable or not.

It is needless to say that engineering experience and judgment as well as the fundamental knowledge and understanding of theoretical soil mechanics are important ingredients in shaping a responsible and experienced FEA user. Benchmarking of FEA analyses is good practice to avoid or reduce carelessness in design. Ong (2006) highlighted that the responses from a typical simulated geotechnical analysis can be benchmarked quantitatively and qualitatively. Quantitative benchmarking involves (i) software vs. software, (ii) software vs. reliable field data and (iii) software vs. reliable laboratory experimental data, which are often used to produce closed-form and analytical solutions, while qualitative benchmarking involves software vs. experience and judgment.

This article seeks to address some of the key concerns made hereinbefore using three geotechnical engineering case studies that the writer has worked on before, namely FE analyses associated to (i) deep excavation, (ii) pile-soil interaction and (iii) embankment dam construction over three separate JURUTERA publications. By no means are these examples exhaustive as they are selected specifically to address some of the basic problems in geotechnical engineering. These illustrations serve to provoke thoughts that are otherwise often regarded as mere run-of-the-mill issues.

The first of three parts of this article describes the benchmarking of a deep excavation problem.

## 2.0 CASE STUDY: BENCHMARKING OF A DEEP EXCAVATION PROBLEM (SOFTWARE *VS* SOFTWARE)

This case study involves a 23m deep excavation carried out in a ground that consists of fill, fluvial sand and residual soil where SPT N values vary from 6 to 62. Limestone bedrock is found underlying the residual soil.

### 2.1 Modelling the problem at hand

For residual soil, use of the common elastic-perfectly plastic Mohr-Coulomb's model is considered reasonable as the soil is expected to behave closer to over-consolidated clays. Input parameters for the selected soil profile are presented in Table 1.

Commercially available 2-D finite element software, SAGE-CRISP version

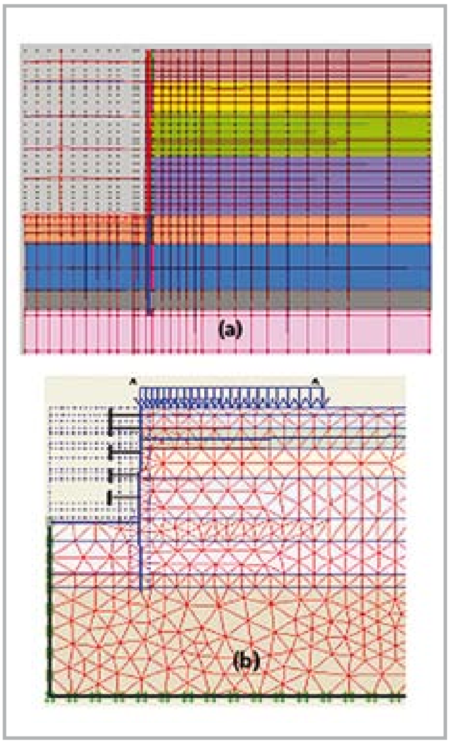


*Figure 1: Finite element meshes for a typical deep excavation using (a) SAGE-CRISP and (b) PLAXIS*

*Table 1: Typical soil properties for Mohr-Coulomb soil model*

| Soil Layer | Soil Description | E′ (kPa) | $K'_0$ | c′ (kPa) | $\phi'$ ($^0$) | $\gamma_{bulk}$ (kN/m$^3$) | $c_u$ (kPa) | k (m/s) |
|---|---|---|---|---|---|---|---|---|
| 1 | Fill | 8696 | 0.5 | 0 | 28 | 19 | 20 | $1 \times 10^{-7}$ |
| 2 | Fluvial sand | 8696 | 0.7 | 0 | 30 | 20 | - | $1 \times 10^{-6}$ |
| 3 | SVI N6 (clayey) | 10435 | 0.8 | 5 | 28 | 20 | 30 | $1 \times 10^{-7}$ |
| 4 | SVI N22 (clayey) | 38261 | 0.8 | 10 | 28 | 20 | 110 | $1 \times 10^{-7}$ |
| 5 | SVI N33 (clayey) | 57391 | 0.8 | 15 | 28 | 20 | 165 | $1 \times 10^{-7}$ |
| 6 | SVI N62 (clayey) | 107826 | 0.8 | 15 | 30 | 20 | 310 | $1 \times 10^{-7}$ |
| 7 | SVI N26 (clayey) | 45217 | 0.8 | 10 | 28 | 20 | 130 | $1 \times 10^{-7}$ |
| 8 | Limestone | 869565 | 0.8 | 50 | 34 | 22 | 20000 | $1 \times 10^{-7}$ |
| 9 | Backfill | 8696 | 0.5 | 0 | 28 | 19 | 20 | $1 \times 10^{-7}$ |

5.1 and PLAXIS version 8.2 were used for comparison. Ong et al. (2006) provides a more detailed description of these analyses. As the lateral and bottom of the FEA mesh extends at least 3.5 times the excavation depth, the boundary effects can be negligible. For the SAGE-CRISP analysis, similar hydraulic boundary conditions as those described in details by Tan et al. (2005) have been strictly adhered to. Drained and undrained analyses are performed using both FE software. However, only SAGE-CRISP is used to perform a coupled-consolidation analysis as this type of analysis is not well-defined in PLAXIS. In each finite element, PLAXIS uses 3 integration points per 6-noded triangular element and 12 integration points per 15-noded element, while SAGE-CRISP uses 7 integration points for a similar 6-noded triangular and 9 integration points for an 8-noded rectangular element. The FEA meshes used for analysis are shown in Figure 1.

In-situ soil stress conditions are manipulated based on local knowledge of the soils, where experience and judgment are essential at this stage. In order to reflect the actual soil-wall interaction behaviour correctly, use of interface or slip elements and their realistic values form an integral part of numerical modelling. Interface elements or slip elements are used in finite element analyses to simulate sliding between two different materials. The temporary retaining wall system uses H-section soldier piles spaced at 1.6m c/c with continuous sheetpile lagging terminated within a layer of soil with SPT N approximately 30. Strutting elements, corresponding preloads and their respective cross-sectional areas are properly accounted for in the FEA.

The excavation is carried out using bottom-up method. The idealized construction sequence is tabulated in Table 2. The detailed idealized construction sequence with its corresponding time period required for each construction stage is incorporated in the coupled-consolidation analysis using SAGE-CRISP.

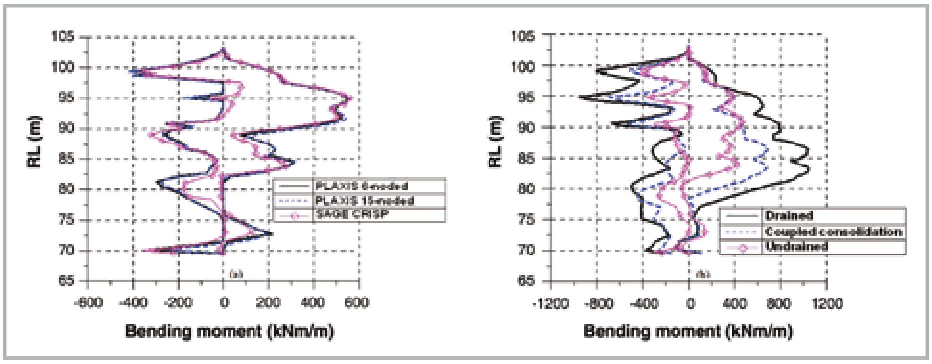


*Figure 2: (a) Comparison of results for different types of elements used and (b) for different types of analyses*

### 2.2 Results, interpretation and discussion

The results shown in Figure 2(a) prove that there are only very slight differences in the wall bending moment envelopes, regardless if 6-noded or 15-noded triangular elements are used in an identical PLAXIS analysis. The wall bending moment envelope is a useful plot that entails and encompasses all the possible development of bending moment profiles for all the defined construction stages. Together with the bending moment envelopes produced from the corresponding SAGE-CRISP analysis, the results generally entail similar consistency. Figure 2(b) shows the wall bending moment envelopes resulted from the effective stress

*Table 2: Construction sequences and respective durations for coupled consolidation analysis*

| Stage | Activity | Duration (days) | Cum. Duration (days) | Cum. Increment/Steps |
|---|---|---|---|---|
| 1 | Excavate to 0.5m below S1. Install S1 and preload | 60 | 60 | 30 |
| 2 | Excavate to 0.5m below S2. Install S2 and preload | 60 | 120 | 50 |
| 3 | Excavate to 0.5m below S3. Install S3 and preload | 60 | 180 | 70 |
| 4 | Excavate to 0.5m below S4. Install S4 and preload | 60 | 240 | 90 |
| 5 | Excavate to 0.5m below S5. Install S5 and preload | 60 | 300 | 110 |
| 6 | Excavate to formation | 60 | 360 | 130 |
| 7 | Place lean concrete. Cast base slab | 50 | 410 | 140 |
| 8 | Backfill. Place lean concrete packing. Remove S5 | 7 | 417 | 145 |
| 9 | Construct walls up to 1.0m below S4 | 30 | 447 | 155 |
| 10 | Backfill and remove S4 | 7 | 454 | 160 |
| 11 | Construct walls and cast concourse slab | 45 | 499 | 170 |
| 12 | Backfill. Place lean concrete packing. Remove S3 | 7 | 506 | 175 |
| 13 | Construct walls and cast roof slab | 95 | 601 | 185 |
| 14 | Backfill. Place lean concrete packing. Remove S2 | 17 | 618 | 190 |
| 15 | Backfill to 1.0m below S1. Remove S1 | 22 | 640 | 195 |
| 16 | Backfill to ground level | 10 | 650 | 200 |
| 17 | Allow one year consolidation | 365 | 1015 | 210 |
| | *Total time considered in analysis* | *1015 days or 2.8 years* | | |

analysis, the results generally entail similar consistency. Figure 2(b) shows the wall bending moment envelopes resulted from the effective stress undrained, coupled-consolidation and drained analyses. It is clear that the largest bending moment envelope is derived from the drained analysis, followed by the coupled-consolidation and then the undrained analysis, which is intuitively correct because undrained and drained analyses are idealized analysis of the two extreme ends of a consolidation process analysis.

### 2.3 Concluding remarks for Case Study 1

This proves that for a same problem in hand, as long as we understand the use and limitations of each software, we can still obtain similar or logical responses which subsequently increase the confidence level that the complex FEA is done correctly. One has to realize that Mohr-Coulomb soil model may be reasonable in this case as residual soils can be loosely said to "behave more like over-consolidated clays". However, the same cannot be said if this excavation is performed in soft clays where the use of Mohr-Coulomb soil model will gravely over-predict the soil strength. A typical example where such unfortunate mistake was made was in the case of Nicoll Highway collapse in Singapore (Yong *et al.*, 2007). Understanding fundamental soil mechanics principles and FE modelling technique are important factors in ensuring successful and reliable simulation of complex geotechnical engineering problems, especially those involving soil-structure interaction. ■

## REFERENCES


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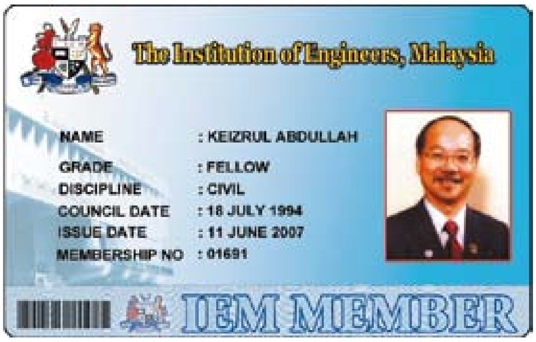


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