Finite Element Modelling of D-wall Supported Excavations

Modèle élément finis d’excavations soutenues par parois moulée

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1 INTRODUCTION

Practical Finite Element Modelling (FEM) is important in geotechnical design of excavations. It is a powerful tool were excavations are located in urban areas. In those areas the impact on the environment is high. Application of FEM plays a role in risk and damage control. Where space is scarce, underground structures, such as tunnels and basements, often support buildings. Other assignments may involve construction close to existing historical buildings. Staged construction of such structures and the impact to their environment can be analysed in all-embracing calculation models.

This paper discusses two cases of D-wall supported excavations. Attention is paid to practical modelling approaches. In FEM D-walls may be modelled as elasto-plastic beam elements, or as linear elastic, non-porous volume elements. Both methods of D-wall modelling are appropriate. However a distinct selection can not be made in advance. The selection depends on project specific functional conditions. What information shall be delivered by the model? Is the D-wall vertically loaded, or does it only retain? What are the environmental conditions? Should soil deformations between the excavation and adjacent buildings be minimised? Or, are structural connections required, between for example D-wall and floors, in order to model the behaviour of the total underground construction?

For two cases the selection of the modelling approach is discussed. The first case is the design of a railway tunnel through the historical city centre of Delft, The Netherlands. Here the elasto-plastic beam elements are applied. The other case concerns the underground expansion of the Drents Museum in Assen, The Netherlands. For the design of the expansion of the Drents Museum the linear elastic, non-porous volume elements were applied to model a jet grout wall. Both projects cannot be compared by means of soil conditions or nature of the proposed developments. The cases are used to provide background for discussion of benefits and disadvantages of both methods.

Selection and application of modelling methodologies and the application of calculation results in the design may provide the reader information to support the selection of the elastic beam elements, or the linear elastic volume elements for other projects.

2 FEM MODELLING OF DIAPRAGHM WALLS – 2 METODS

For design purposes two methods are commonly applied for finite element modelling of diaphragm wall supported excavations (CUR 231, 2010). This section explains the two methods in detail. Advantages and disadvantages are provided that may contribute to pre-selection of the model that fits best to the specific project features. The two models can be described as follows:

- Method 1: elastic (or elasto-plastic) beam element;
- Method 2: linear elastic or Mohr Coulomb, non-porous volume element.

Modelling diaphragm wall as beam element (Method 1) requires input parameters such as w (kN/m²), EI (kNm²/m), EA (kN/m), n (-), Rinter (-), Mpl (kNm/m) and Npl (kN/m). The latter two parameters apply to the elastoplastic model. Current generation of user friendly FEM software (Plaxis) do not comprise material models simulating concrete behaviour. The properties of the diaphragm walls should be varied manually. Where the bending moment exceeds the cracking limit the Young’s modulus ($E_{\text{cracked}}$, MPa) should be reduced (generally to $E_{\text{cracked}}$, 10,0 MPa to 12,5 MPa). Diaphragm walls have high weights and often a bearing function. In order to model such features in FEM a “fixed-end-anchor” (spring element) should be defined at the bottom of the diaphragm wall beam. The vertical spring stiffness of this fixed-end-anchor can be fitted to...
1. Earthworks for underground infrastructure (pipes and an accumulation of deformations, as follows: alignment. The deformations of foundations of contiguities are distortion and horizontal strain of buildings along the tunnel diaphragm walls in order to meet the criteria for angular requirements. Additional measures to limit deformations of the construction sequence, using diaphragm walls was adopted. Approximately 10 m below ground surface. Nearby buildings

3.1 CASE INTRODUCTION

3.2 Drents Museum Assen

The Drents Museum Assen is located on a historical rich site in the city centre of Assen, the provincial capital of Drenthe. As a result of further development and growth of the museum, a new large underground exhibition hall is realised. The expansion provides an underground connection of the exhibition hall with the monumental main building. To realise this connection, an underground excavation right underneath the monumental Bailift’s House is executed.

The excavation, to a level of about 8 m below ground surface, is realised in two separate building pits: the main excavation for the exhibition hall and the indoor excavation (Figure 1) below the monumental Bailift’s House. The indoor wet deep excavation is retained by jet grout walls (VH-Grouting). These walls also support and reinforce the existing shallow foundations (Figure 2). To achieve the required wall thickness of about 1.0 m up to 1.5 m two rows of columns are installed in a triangular mesh of 0.6 m to 0.7 m. Each column
has a grout diameter of about 0.9 m with an overcut of about 0.2 m up to 0.3 m. The column dimensions are verified by continuous monitoring of jet pressure and injected grout volumes.

The jet grout walls are installed from foundation level (NAP +9.0 m) to tip level at NAP -2.5 m (Figure 2). To reduce the risk of failure of the foundations the installation sequence of the grout columns is adjusted. At critical locations larger intervals between fresh casted columns is applied. The columns are reinforced to obtain the required strength and stiffness.

FEM analysis with PLAXIS 2D and 3D is used to assess the wall thickness and excavation sequence with underwater concrete floor and anchor piles. And to predict and postdict the deformations of the existing foundations.

Figure 1. Indoor wet excavation.

To model the jet grout wall with Mohr Coulomb, the strength and stiffness were calculated by means of the ultimate compression strength $f_c = \text{UCS}$ using the empiric relations of Van der Stoel (2001):

$$\phi^\prime = \phi^\prime_{\text{soil}} + 0.5 \cdot \phi$$

$$c^\prime = 0.2 \cdot 0.3 \cdot f_c$$

$$E_{50,\text{sand}} \approx 800 \cdot f_c^{0.5}$$

$$E_{50,\text{clay}} \approx 500 \cdot f_c^{0.67}$$

$$\nu \approx 0.2$$

Figure 2. Jet grout wall.

4 SELECTION OF METHOD

4.1 General

Modelling method selection is part of the design process. The engineer should have an overview of environmental features such as foundation types of contiguities, dimensions and soil profile and properties. To make the selection several questions need to be answered. What information should the model produce? What loads are applied on the diaphragm wall (vertical, lateral, both)? Should deformations be quantified of buildings supported by shallow foundations or deep foundations? Are the retaining walls connected to concrete slabs and temporary struts? Are properties of such structural elements critical to the performance of the construction in relation to deformations.

Where modelling ground surface response at the active side should be emphasised for (temporary and multiple) supported walls, Method 1 is recommended. In cases of modelling vertically loaded walls, interaction with neighbouring pile foundations or other walls (group effects) Method 2 is recommended.

4.2 Railway tunnel Delft – Method 1

Primary focus for this project was assessment of the deformations of buildings and monuments. The allowable deformations of the contiguities are very small and were according to an amplified Boscardin and Cording (1989) approach. They are combinations of angular distortion and horizontal strain. Most buildings in Delft are supported by shallow foundations with foundation levels at about 0.8 m below ground surface.

In co-operation with structural engineers the tunnel outline was designed. Detailed geotechnical analyses comprised FEM in order to assess the interaction of the tunnel construction with the environment for each distinguished construction stage. A flexible design model was required to allow for rapid modifications in the model where the building deformation criteria were not met.

The emphasis was put on surface settlement assessment and verification of preliminary structural design. Method 1 was the appropriate model.

Along the tunnel alignment the buildings were classified based on the allowable additional deformation, from slight to negligible. The condition of each building was accurately recorded. This way imperative behavioural design could be fit to each individual building case.

Finite element models (Plaxis 2D and Plaxis 3D) were used to assess the deformation of the tunnel system. The model does not take account of interaction between soil and foundation slabs. It assesses green field deformations outside the tunnel trench. The deformations at foundation level can be extracted from the model.

Using a cross section over Phoenixstraat 30 and Spoorsingel 25 (Figure 3) the deformation analyses is explained. Figure 4 shows a location map with the location of the example cross section. The building Phoenixstraat 30 has an old part which is in poor conditions (class IV) and a new part which is in fair conditions (class II). There is a basement below the building at about 2.0 m below ground surface. The building Spoorsingel 25 (class III) opposite of Spoorsingel 30 does not have a basement. This building has a foundation level at 0.8 m below ground surface.

Figure 3. Cross section FEM Method 1

Calculations proved that additional measures are required to limit the horizontal deformation of the diaphragm wall during the first excavation stages. Measures selected for this cross section are the introduction of additional struts at surface level and the use of 3.8 m wide diaphragm wall panels (standard width 7.5 m).
Figure 5 presents an up-scaled graph based on Boscardin and Cording (1989). It shows that the most critical construction stage for Phoenixstraat 30 is at the end of the construction of the eastern tunnel tube (about 50% the total construction period).

The critical construction stage for Spoorsingel 25 is the final stage. Further, the verification of deformation criteria proves that the combination of horizontal strain and angular distortion is met during all intermediate design construction stages.

4.3 Drents Museum Assen – Method 2

One of the critical requirements was the maximum tolerated settlement and heave of the foundation during the excavation below the monumental building. The maximum allowable vertical displacement for the foundations is 5 mm to 10 mm which corresponds to relative rotations of 1:500 to 1:1,000. The existing foundations are modeled as separate shallow foundations (including basement) as shown in Figure 6.

To evaluate the applied geotechnical calculation models and the predicted soil and structural behaviour, post diction analyses with 3D-FEM have been performed (Figure 7) based on the latest monitoring results during execution. Due to the wet excavation, the foundation settlement was 4 mm to 9 mm. After dewatering the excavation, the postdicted foundation rebound was about 4 mm to 5 mm due to developing tension resistance in the anchors below the elastic underwater concrete floor during the instantaneous swell of the underlying soil layers and the primary swell of the deeper slightly over-consolidated clay.

The main reason for selection of Method 2 was to assess foundation deformations as well as swell deformations of the bottom of the excavation based on realistic stress distribution.

5 CONCLUSIONS

In this paper two methods are described for finite element modelling of diaphragm wall supported excavations. Advantages and disadvantages are given that may contribute to pre-selection of the model that fits best to the specific project features.

Method 1 was applied for modelling the railway tunnel in Delft because of the requirement of flexible design models in combination with shallow foundations sensitive to deformations.

For the case in Assen Method 2 was selected. The requirements for this case better agree with the advantages of better visualisation of wall and soil behaviour and calculation of stresses and deformations in soil, wall and foundation.

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7 REFERENCES


