Finite Element Modelling of D-wall Supported Excavations

Modèle elément finis d'excavations soutenues par parois moulée

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ABSTRACT: Two different methods of Finite Element Modelling (FEM) of diaphragm walls are explained. Both methods are applied in state of the art geotechnical practice and comprise beam elements (Method 1) and elasto-plastic volume elements (Method 2). Selection of the appropriate method is not clear in advance and depends upon project specific requirements. In this paper the selection process is illustrated based on two cases. The first case is a large infrastructural railway project through the historical city centre of Delft, The Netherlands. The second case is an underground expansion project of the Drents Museum in Assen, The Netherlands.

RÉSUMÉ : Cet article détaille deux modèles de parois moulées à l'aide de la méthode des éléments finis (FEM). Les deux méthodes suivent les derniers développements en géotechnique utilisant des éléments de poutre (Méthode 1) et des volume élasto-plastiques (Méthode 2). La méthode appropriée s'est avérée dépendante des besoins spécifiques pour un projet donné. Le processus de sélection est décrit dans cet article à l'aide de deux exemples. Le premier est un projet d'infrastructure ferroviaire de grande envergure dans le centre historique de la ville de Delft, Pays-Bas. Le second porte sur un projet d'agrandissement souterrain du musée Drents à Assen, Pays-Bas.

KEYWORDS: Deep excavations, Diaphragm wall, Jet grout wall, Tunneling, Finite element modelling.

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/m^2), EI (kNm^2/m), EA (kN/m), n (-), R_{inter} (-), M_{pl} (kNm/m) and N_{pl} (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 ($E_{uncracked}$, MPa) should be reduced (generally to $E_{cracked}$, 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

NEN 9997-1 (2011) or equivalent. The parameter R_{inter} is the ratio tan(δ')/tan(φ'). Smear of bentonite should be considered.

The advantages of this Method 1 are: the bending moments, shear and normal forces and deformations can directly be read from the beam, structural connections to floors and struts can easily be defined and the method is suitable for strength analyses of the wall.

Disadvantages are: possible numerical problems caused by the mesh in the area around the tip of the beam and fixed-end-anchor and the unrealistic stress distribution below the tip of the beam element. There is a work around for the first disadvantage by extending the interface into underlying strata. The second is important where group effects are significant. Here application of Method 2 may be considered or a crossbeam could be introduced at the beam tip. The vertical spring stiffness of the beam/crossbeam should again be fitted to NEN 9997-1 (2011).

Application of Method 2 comprises linear elastic or Mohr-Coulomb volume elements. The elements are modelled with realistic dimensions (thickness and height). The required input consists of parameters such as γ (kN/m^3), $E_{uncracked/cracked}$ (MPa, like Method 1) and R_{inter} (-). When using Mohr Coulomb, additional strength parameters as c' and φ' are required.

The advantages of Method 2 are: better visualisation of behaviour, proper calculation of stresses and deformations in the soil, more stable numerical calculation process (especially where walls have a bearing function) and a more realistic vertical deformation behaviour at the tip (especially when interaction with the environment is considered at tip level) and of the wall itself (especially when the thickness is not constant).

Disadvantages are: load-settlement behaviour not in accordance with NEN 9997-1 (2011), bending moments and forces can not easily be extracted from the volume element and structural connections to the diaphragm walls are difficult to model. When using Mohr Coulomb for mixed or injected walls, information of soil strength and stiffness is required for the determination of strength and stiffness of the D-wall by using empirical relations (Van der Stoel, 2001). Concerning the first disadvantage, the spring stiffness can be fitted to standard load-settlement curves by introducing a thin dummy volume element below the diaphragm wall. A work around for the second disadvantage is modelling a beam inside the linear elastic volume element. This beam should not contribute to the strength and stiffness of the diaphragm wall. Where struts are required, or other structural connections, a dummy plate may be introduced to the model having EI ≈ 0 kNm^2.

It should be noted that installation effects and uncertainties at the soil-wall interface (smear) make bearing capacity and vertical stiffness hard to predict. It is common practice to apply the design approach of bored piles to situations where cast in-situ concrete walls are considered.

3 CASE INTRODUCTION

3.1 *Railway tunnel Delft*

The Delft railway tunnel project comprises the design and construction of a 2.4 km long, four track double railway tunnel in the historical city centre. The excavation level is approximately 10 m below ground surface. Nearby buildings are supported by shallow foundations at very close distances from excavations. Therefore, a top-down multi-propped construction sequence, using diaphragm walls was adopted.

Construction of the diaphragm walls near critical buildings require additional measures to limit deformations of the diaphragm walls in order to meet the criteria for angular distortion and horizontal strain of buildings along the tunnel alignment. The deformations of foundations of contiguities are an accumulation of deformations, as follows:

1. Earthworks for underground infrastructure (pipes and cables) in the narrow area between the buildings and the diaphragm wall. At some places the distance is less then 4.0 m and the excavation depths over 2.5 m.
2. Removing obstacles of the historic town defense walls at the proposed route of the diaphragm walls (excluded from the analyses, impact is negligible).
3. Trench deformations during excavation with the ground supported by bentonite mud or similar. Once the reinforcement cage has been lowered into place, concrete is tremmied into the slot, displacing the mud.
4. Deformations as a result of staged excavation of the strutted tunnel trench.

Finite element models (Plaxis 2D and Plaxis 3D) were used to assess the deformations of the tunnel system. The diaphragm walls have typical thicknesses of 1.0 m and have standard widths of 7.3 m. Standard excavation stages consider two strut levels; the first just below surface level and the second at 50% of the final excavation level. The model does not take account of interaction of soil and foundation slabs. It assesses green field deformations outside the tunnel trench. The deformations at foundation level can be extracted from the model.

The design approach outlined below was adopted for the prediction of deformations:

1. The ground deformations are assessed (SLS) as a result of the construction of the diaphragm walls for panel widths of 3.8 m and 7.3 m (Plaxis 3D)
2. The required dimensions of the diaphragm wall are determined with an elastic beam model using bi-linear ground springs in (ULS and SLS) in combination with structural analyses (ESA PT).
3. The ground deformations are assessed (SLS) as a result of cable and pipe trenching.
4. The ground deformations are assessed (SLS) as a result of the tunnel trench excavation taking account of detailed construction stages (Plaxis 2D). This model continues from step 3 and uses the input from step 2.
5. Finally the results of step 1 and 4 are combined. Where the deformation requirements were not met additional measures have to be taken, as described below.

Additional efforts to meet the deformation criteria of buildings focus on further limiting the deformations of the diaphragm walls by:

- Excavation in stages, were the groundwater in the building pit also is lowered in stages.
- The panel width can be reduced to 3.8 m.
- The struts could be pre-stressed to reduce elastic shortening of the steel cross section and to pre-stress the ground at the active side of the retraining walls.

3.2 *Drents Museum Assen*

The Drents Museum 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 Bailiff'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 Bailiff's House. The indoor wet deep excavation is retained by jet grout walls (VHP-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 = UCS using the empiric relations of Van der Stoel (2001):

$$\varphi' \approx \varphi'_{soil} + 0 \; \grave{a} \; 5^o \quad ; \quad c' \approx 0.2 \; \grave{a} \; 0.3 \cdot f_c$$

$$E_{50;sand} \approx 800 \cdot f_c^{0,5} \quad ; \quad E_{50;clay} \approx 500 \cdot f_c^{0,67} \quad ; \quad \nu \approx 0{,}2$$

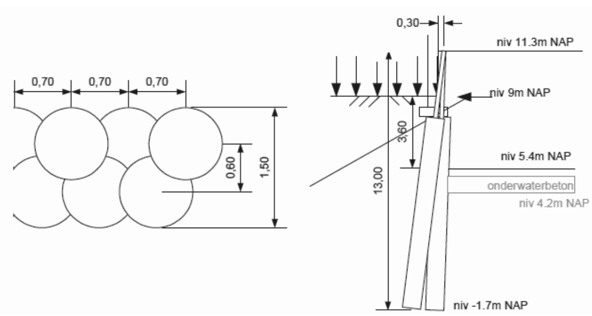

Figure 2. Jet grout wall.

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 deformations 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.

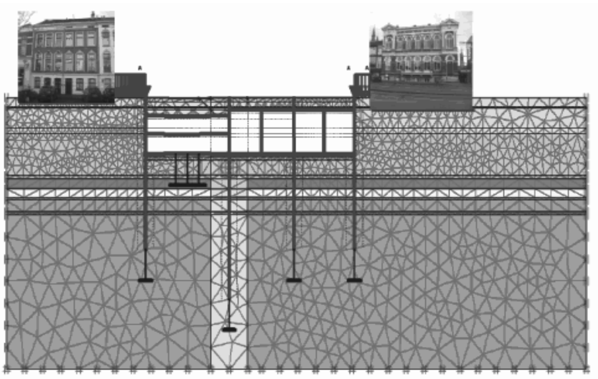

Figure 3. Cross section FEM Method 1

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).

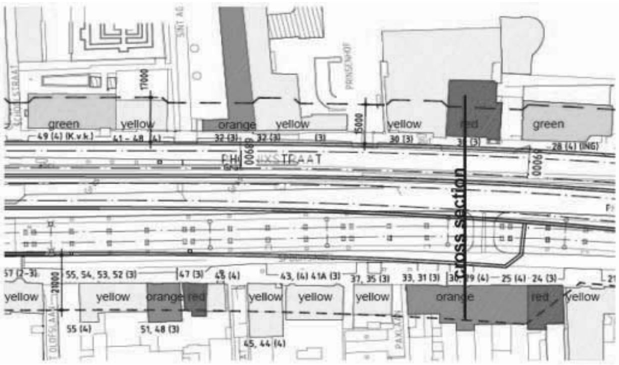

Figure 4. Cross section location map.

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).

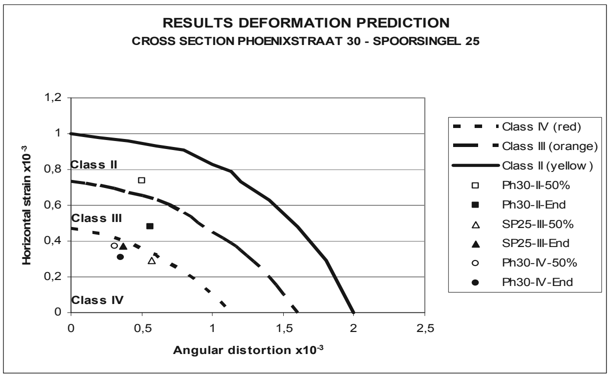

Figure 5: Verification of allowable building deformations

Comments Figure 5:

- Phoenixstraat 30, new part (class II): II-50% (construction stage), II-End (final stage)
- Phoenixstraat 30, old part (class IV) IV-50% (construction stage), IV-End (final stage)
- Spoorsingel 25 (class III) III-50% (construction stage), III-End (final stage)

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.

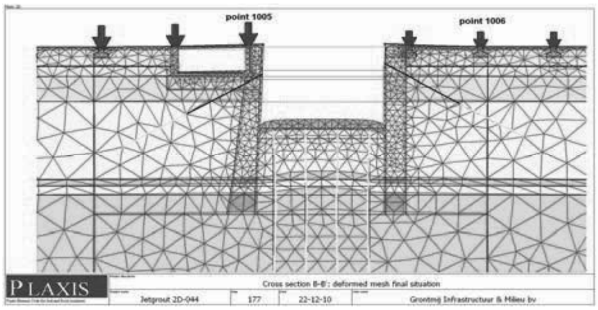

Figure 6. Cross section FEM Method 2.

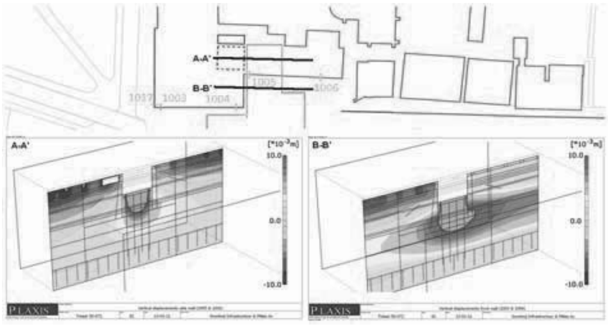

Figure 7. 3D-FEM postdiction of vertical displacements.
Note: Figure 7 should be read in combination with Figure 6.

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.

6 ACKNOWLEDGEMENTS

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