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The use of soil-structure interaction for cost effective design of vertical loaded embedded abutment walls

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Abstract

Due to construction and geometrical constraints more and more bridge abutments around the world consist of embedded retaining walls. Such structural elements serve two purposes as they act both as earth retaining and load carrying structures, thus increasing the complexity of their structural and geotechnical design. When subject to substantial vertical loads, the traditional limit equilibrium analysis for retaining walls does not allow to achieve a cost-effective design. Furthermore, the lack of general consensus on the vertical capacity of sheet piles, within practitioners and academics, results in the need for more research on the subject.

This study focuses on the design of an embedded abutment wall that supports a heavy concrete highway bridge deck spanning 24 m and with a retained height of approximately 7m to make room for a road underpass. The characteristic line load on the wall, due self-weight of the bridge deck, amounts to some 700 kN/m. The construction sequence requires a top-down methodology in order to ensure a continuous traffic flow during construction, therefore justifying the choice of using sheet pile walls.

However, due to the high vertical loads, driveability risks, and the requirement to minimise settlements of the existing embankment a traditional sheet pile wall solution was discarded and an alternative solution was sought, consisting of a combi-wall, made of alternating sheet pile and hollow steel pipe sections, offering a higher flexural stiffness and vertical capacity. The soil-structure interaction is investigated by means of limit equilibrium and finite element analyses.

article. The geotechnical design, and the structural design of the retaining walls, was carried out in FREW 19.3 (Oasys, 2017) and Plaxis 2D 2017.

The current main road shall only be minimally disturbed by the construction of the bridge and thus a top down construction method was proposed. This allows the construction of the western, new embankment and bridge first, to thus redirect traffic to while construction undergoes under the existing embankment confer Figure 2.

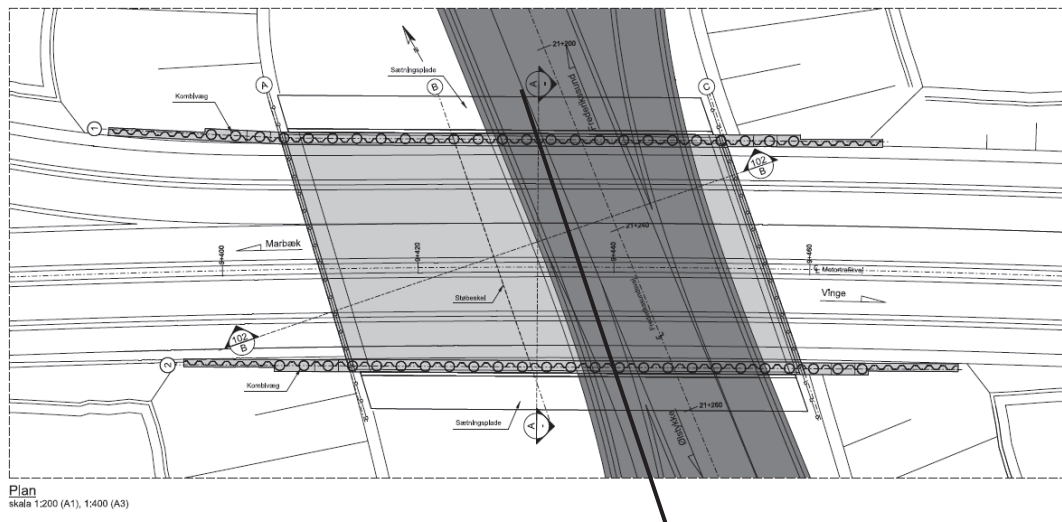


Figure 2: Underpass and embankment. Thick black line approximately separates the western and eastern side of the bridge

The final road level is in +8.9m DVR90, and the road build up amounts to 0.8 m; implementing an over-dig of 0.5 m the final excavation level is +7.6m DVR90 which results in a retained height of some 9 metres. The line loads acting on the bridge abutments in SLS and ULS are approximately 800 kN/m and 1050 kN/m respectively. The bending moments exerted by the bridge deck connection are approximately 800 kNm in SLS and 1100 kNm in ULS. This requires a minimum corroded section modulus, taking into account bending and normal force using a steel quality S355, of $W=5000 \text{ cm}^3$. Only the very largest sheet pile section from Arcelormittal, AZ50, fulfils this requirement (Arcelor, 2016). Due to the relatively high retained height combined with the relatively high dead loads, a traditional sheet pile method was abandoned in favour of a stiffer combi-wall solution. This consists of circular hollow piles, with an outer diameter of 1016 mm, steel thickness of 16 mm, connected with AZ20-700 sheet piles confer Figure 3.

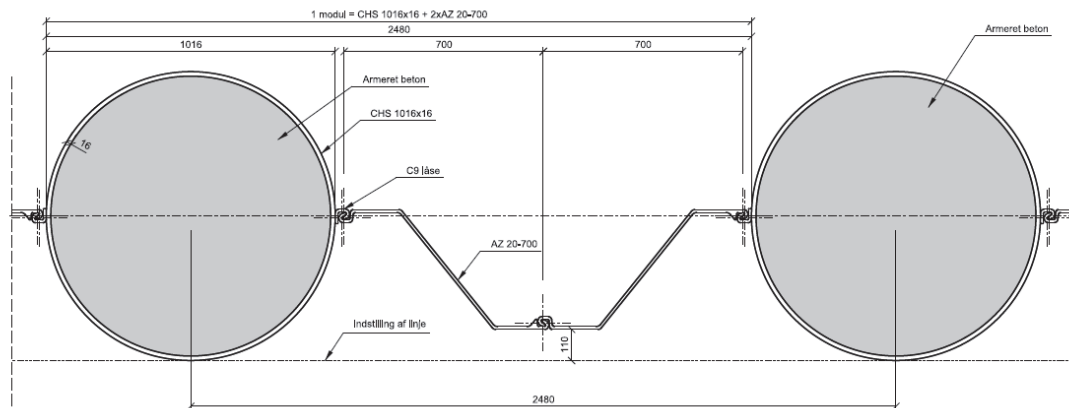


Figure 3: Cross section of the adopted combi wall.

The combi-wall solution has the advantage of offering a stiffer lateral response along with the ability to carry higher vertical loads with the hollow pile sections.

Soil conditions

Geotechnical boreholes have been carried out in the vicinity of the new bridge and show a relatively uniform stratigraphy. The existing embankment consists of remoulded clay till with good strength and stiffness properties, due to their properties at the time of the embankment construction as well as to the consolidation occurred over several decades; the new embankment will consist of coarse-grained imported fill. Both embankments are/will be founded on undisturbed clay till with undrained shear strength exceeding 250 kPa. Both undrained and drained analyses were carried out, and the latter showed to be governing and thus only the drained analysis is addressed in this article.

Project overview

A simplified soil stratigraphy is shown in the Table 1

Table 1: Simplified soil stratigraphy – existing embankment

Soil type	General input						Plaxis Hardening soil		
	Level top [m DVR90]	γ/γ' [kN/m ³]	ϕ' [°]	c' [kPa]	ν	E [kPa]	E_{s0} [kPa]	E_{ur} [kPa]	OCR [-]
Fill	+16.8	18/10	25	0	0.3	20000	24000	72000	1
Clay fill	+14.5	20/10	32	0	0.3	30000	40000	120000	2
Clay till	+6.1	21/11	30	20	0.3	60000	80000	240000	3

Groundwater is measured at approximately +7.0 m.

Embedded retaining wall modelling

Calculations were carried out in FREW and in Plaxis 2D to compare the modelling methods and to better understand the distribution of stresses.

Frew modelling

The geotechnical design was carried out in FREW, a commercial software from Oasys, using the soil parameters given in Table 1 and the wall stiffness parameters shown in Table 2.

The following was implemented in FREW

- 1) Groundwater level was set to + 11.5 m DVR90 on the retained side of the wall and + 7.0m DVR90 on the excavation side of the wall;
- 2) For lateral stability calculations, the ratio between structure-ground interface friction angle and the angle of shearing resistance in terms of effective stress was set to 0 on the active (clay fill only) and to 2/3 on the passive side.
- 3) Rotational spring of 1.150.000 kN/m/m, with a lever arm of 1m applied 0.6 m below top of wall.
- 4) Axial spring of 3.200.000 kN/m/m applied 0.6 m below the top of wall.

As the excavation progresses and the retaining wall starts to move and rotate towards active and passive failure states, the vertical load transfer, from wall to soil, depends on when the shear stresses, on the active side, changes direction. Typically, for walls that support large vertical loads at the maximum excavation stage, which can settle relative to the ground under load, the operative direction of wall friction on the retained (active) side may be reversed. Consideration must be given to how vertical loads on the wall are carried, and inconsistent or ‘double counting’ of ground/wall friction (eg upward to support a vertical load, and downward to reduce active earth pressure coefficients) must be avoided. It may be necessary to carry vertical loads primarily in end (base) bearing, and to assume zero friction over some or all of the depth of the wall on the retained (active) side (Gaba et al., 2017).

As FREW only model the lateral behaviour of the wall, it was conservatively assumed zero wall friction on the active side in order to determine the lowest tip level required to ensure lateral stability. Beneath this tip level the wall is assumed to behave as an axially loaded pile, implementing normal static analytical pile bearing capacity calculations. While the author appreciates that this method leads to some conservatism, it is often used in the absence of a full finite element model (German Geotechnical Society 2013).

Plaxis 2D modelling

The Plaxis model is set up using the soil parameters given in Table 1 and using the input stated in

Table 2 and

Table 3 for the combi-wall and concrete deck respectively. The hardening soil model was used to more accurately model the soil stiffness during unloading. All model stages have been run with default settings i.e.

- Tolerance: 0.01
- Iterations: 60
- Mstage 0.5

Table 2: Combi-wall input parameters for Plaxis 2D model

I [m ⁴ /m]	A [m ² /m]	EA [kN/m]	EI [kNm ² /m]
2.115E-02	3.328E-01	1.209E+07	9.443E+05

Table 3: Bridge deck input parameters for Plaxis 2D model

E [kPa]	A [m ² /m]	EA [kN]	I [m ⁴ /m]	EI [kNm ² /m]
3.200E+07	1.2	3.840E+07	1.440E-01	4.608E+06

Staged construction – Plaxis and FREW

The models are created with 9 different stages:

1. Initial phase
2. Wall installation: the comb- walls are installed
3. Concrete deck excavation: 1.2 m excavation for the concrete deck.
4. Concrete deck install: the concrete deck and the sheet pile wall are connected (install strut in FREW) using the stiffness parameters given in
5. Table 3
6. Excavation to GWL: excavation to level 11.5 m, pore pressure is set to steady state groundwater flow (Plaxis)
7. GW lowered: lower of the ground water to 0.5 m under excavation depth (Plaxis)
8. Final excavation: the final excavation level 7.5 m
9. Load: at the right side if the excavation a line load of 6 kN/m/m and two point loads of 100 kN/m
10. Safety (only Plaxis)

Comparison

Excavation steps, strut input and groundwater levels have been kept the same in both models to ensure an accurate comparison.

Horizontal displacements, bending moment and shear force of the combi-wall were compared. For the displacements, the Frew horizontal displacements are significantly larger than the Plaxis model: the maximum horizontal displacement was approx. 16 mm where Plaxis shows around 8 mm. The displacement shape,

however, is relatively similar for both programs. Bending moments and shear forces are within 10-20 %. Table 4 summarizes the comparison results in the most onerous stage, ie full excavation, and Figure 4 summarizes the results individually.

Table 4: Plaxis vs. FREW comparison, using characteristic strength and stiffness parameters.

Comparison	Frew	Plaxis
Horizontal displacement [mm]	16	8
Bending moment [kNm/m]	810	840
Shear force [kN/m]	185	225
Strut force [kN/m]	300	245

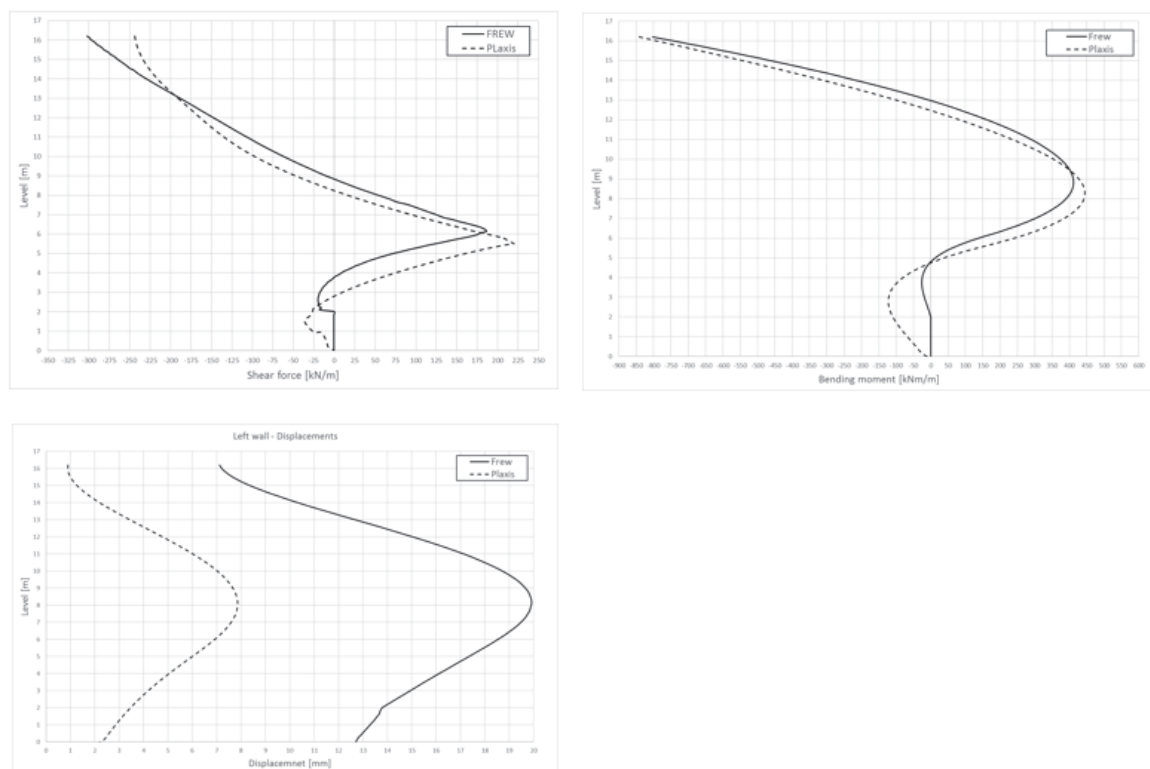


Figure 4: Bending moments, shear forces and displacement from FREW and Plaxis

Factor of safety, FoS, and horizontal stability:

The minimum toe level, for lateral stability, has been investigated in both FREW and Plaxis. FREW can calculate the minimum embedment based on horizontal equilibrium, however this is not a functionality of Plaxis, and thus the embedment length was shortened until collapse to estimate the minimum toe level. It is noted that safety analysis in Plaxis are subject to a number of limitations which are beyond the scope of this paper.

As mentioned previously, δ the structure-ground interface friction angle, was set to 0 on the active side, in the clay fill, and $2/3 \phi$ on the passive side and in the clay till, in FREW to conservatively determine the minimum tip level. Table 5 summarises the results and as it is noted the tip levels are the same.

Table 5: Comparison of minimum tip level, to ensure lateral stability

Program	Minimum tip level [m]
FREW	4.5
Plaxis	4.5

FoS cannot be calculated by FREW simply, however Plaxis can estimate this by the “phi-c reduction” procedure until failure is achieved. The overall factor of safety is 2.5: it is noted that failure is observed underneath the pile tip as expected and depicted in Figure 5.

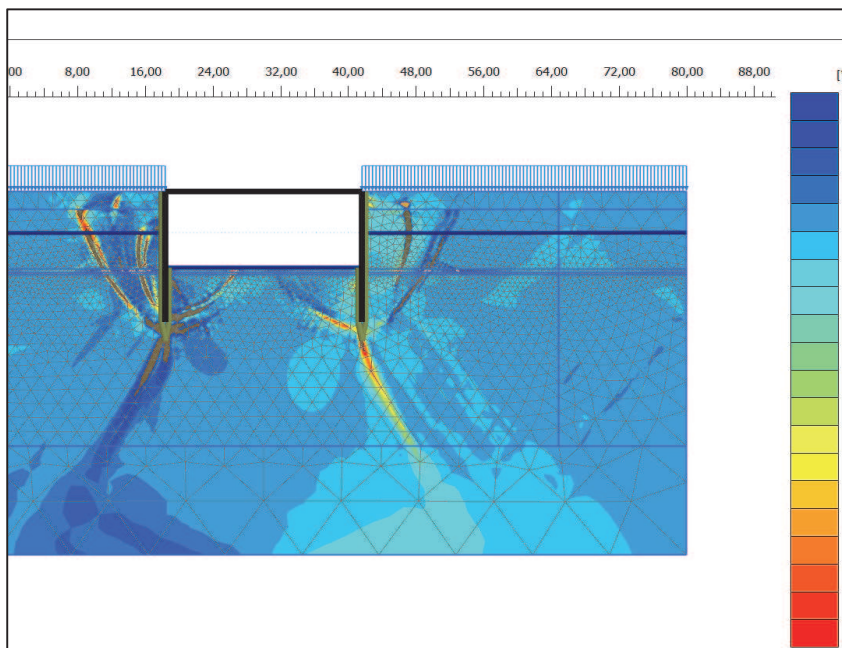


Figure 5: Incremental strain plot from Plaxis.

Vertical load transfer

The vertical load from the deck and live load shall be carried by the wall. However, due to the wall movement, confer Eurocode 7, 2013, it is not safe to calculate the whole length of the wall as a pile and thus the following were investigated. It is common practice, to use the soil-wall friction angle δ of $0.5 - 2/3 \phi$. Plaxis calculates the relative soil-wall movement and the direction of the principal stresses, which can be observed on Figure 6. The principal stress, σ_3 , commences to act upwards around level +6.5m. This is in line with the

assumption made in the FREW calculation and also corroborates the similarities in minimum tip level.

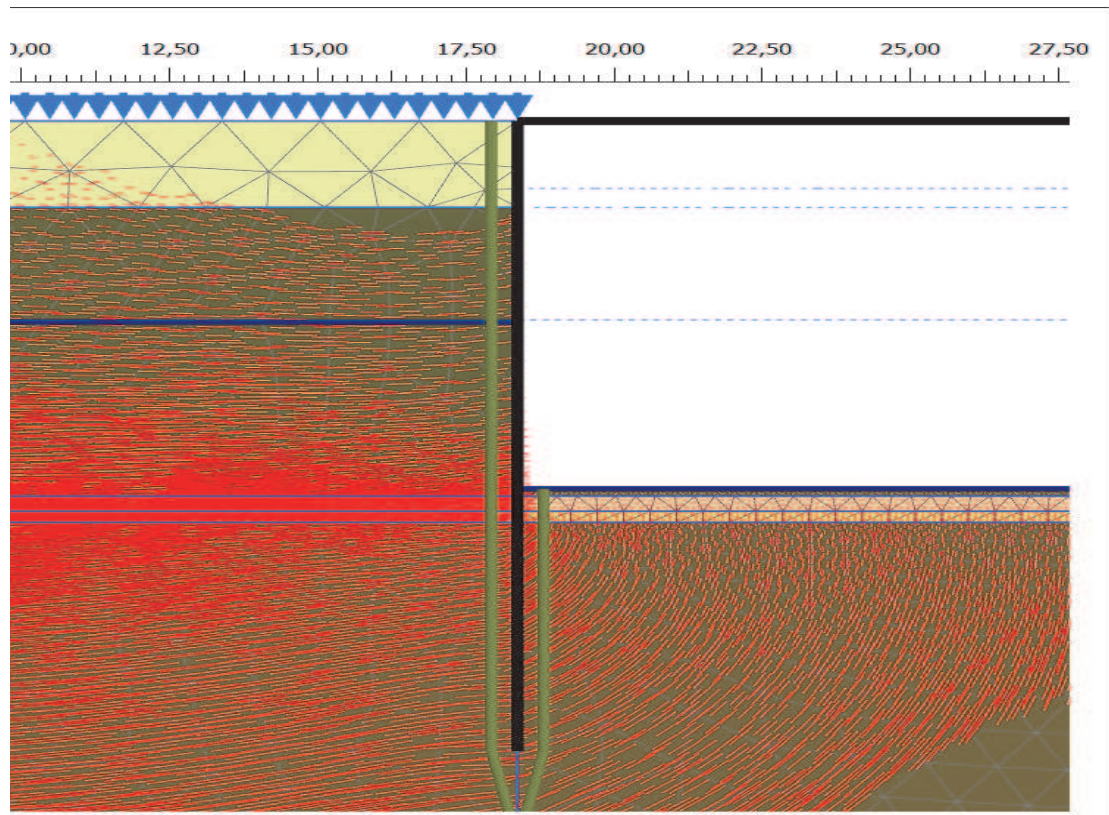


Figure 6: Direction of principal stress – Plaxis 2D - 2017

Conclusion

A combi-wall abutment supporting a heavy highway bridge deck has been modelled using both FREW and the finite element software Plaxis 2D. The soil model in Plaxis has been set to the hardening soil model to estimate consequences of unloading, and the FREW model has been set up using linear elastic stiffness parameters. The results from both programs are very similar and corroborates the assumptions and simplifications made by FREW. The largest difference observed is with regards to horizontal wall movement, where FREW yields two times the movement to mobilise the same earth pressure. The reason behind is most likely due to the mobilisation curves of active and passive earth pressure and the input parameter K_r , which partially controls the stiffness response. In terms of stresses induced in the wall, however, the two software provide very similar results as the wall deflection shape is comparable. Further investigation into the soil-wall friction angle is needed, when using this type of walls as bridge abutments, to further optimise the wall design.

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