Integration of ULS and SLS based design for interpretation of soil-structure interaction

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ABSTRACT: Finite Element Method (FEM) is usually used as a performance-based (Serviceability Limit State or SLS) design method to provide in-sights into complex behaviour of soil-structure interaction. The advantage of using FEM in challenging geotechnical projects is the ability to predict movements so as to maintain the functional use of building and infrastructure when they come into service. In this paper, an attempt is made to study the possibility and applicability of FEM in geotechnical Ultimate Limit State (ULS) design using EN1997-1 or otherwise known as Eurocode 7: Geotechnical Design – General Rules. As structural design is mainly done based on ULS, its integration with geotechnical structure, which is usually designed for SLS, remains a challenge. From the 2 idealised case studies carried out involving a shallow foundation and an anchored sheetpile wall, it is confidently shown that FEM has a place in ULS design for strength and SLS design for serviceability of geotechnical structures, but not without good understanding of geotechnical knowledge. However, its application must be prudently scrutinised especially when complex soil model is adopted as it may involve development of artificial yield surfaces, thus increasing the complexity of interpretation of the problem in hand. Therefore, sound geotechnical skills, experience and field observations are required when making the overall judgment.

INTRODUCTION

In order to provide some in-sights in analysing complex soil-structure interaction problems, two idealised case studies are illustrated, namely (i) a shallow foundation involving a square pad footing founded on soft clay and (ii) an anchored sheetpile wall embedded in clay supporting a deep excavation. The applicability of Serviceability Limit State (SLS) design via the use of finite element method (FEM) and Ultimate Limit State (ULS) design via established empirical method or Limit Equilibrium Method (LEM) in interpreting complex soil-structure behavior are examined in detailed hereinafter.

DESIGN APPROACHES OF EUROCODE 7

The summary of recommended values of partial factors for the three Design Approaches (DA) according to EN1997-1 is shown in (Table 1). Design values are factored values based on ULS design while characteristic values are unfactored original values used for analysis.

Table 1. Recommended values of partial factors in persistent and transient situations for the 3 Design Approaches according to EN1997-1 (Bauduin, n.d.)

DA	Actions or action effects γ_F		Soil parameters γ_M				Resistances	
Design Approach	Permanent unfavourable[2] γ_G	Variable[3] γ_Q	Density γ_γ	$\tan\phi'$ γ_ϕ	c' γ_c	c_u γ_{cu}		Piles
DA1-(1)[1]	1.35	1.50	1.00	1.00	1.00	1.00	1.00	1.3 – 1.6
DA2-(2)[1]	1.00	1.30	1.00	1.25	1.25	1.40	1.00	1.1
DA2	1.35	1.50	1.00	1.00	1.00	1.00	> 1.00	1.1
DA3	Geo[4]: 1.00 Struct[5]: 1.35	1.30 1.50	1.00	1.25	1.25	1.40	1.00	NA

1: Design to be based in most severe of both calculations

2: Favourable permanent action, γ_G=1.00

3: When unfavourable; For favourable action, γ_Q=0.00

4: Geotechnical action: action transmitted to the wall through the ground

5: Structural action: action from a supported structure applied directly to the wall

IDEALISED CASE STUDY 1: SQUARE FOOTING FOUNDED ON SOFT CLAY

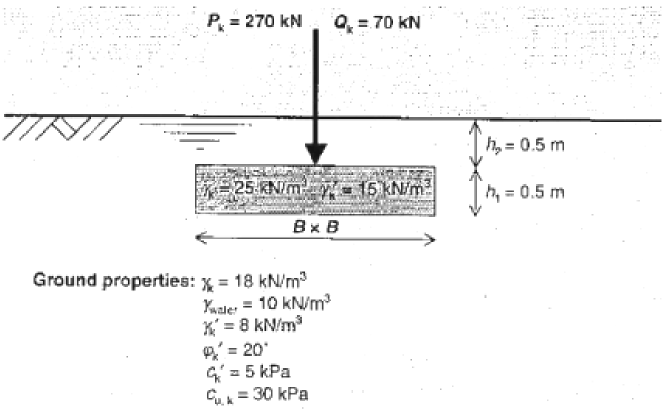
Appreciation of Problem

Case study 1 is a reproduction of Example 6.1 found in Frank et al. (2004). (Fig 1) shows a 0.5m thick square pad footing, bearing on soft clay 1m below ground level and carries a vertical, centric, permanent load of 270kN and a variable load of 70kN. The water table is at ground surface.

ULS Design Based on EN1997-1 Annex D

The ULS design of the footing is performed using the direct analytical method provided in Annex D EN1997-1. Undrained condition is to be checked using the three Design Approaches presented in (Table 1.) Annex D provides a sample analytical method for bearing resistance calculation and the approximate equations for the design of vertical bearing resistance, derived from plasticity theory.

Fig. 1. Case Study 1 – Square footing founded on soft clay (Frank et al., 2004)



Under undrained conditions, the design bearing resistance may be calculated from:

$$R/A' = (\pi + 2)\, c_u\, b_c\, s_c\, i_c + q \qquad (1)$$

Where:
c_u = undrained shear strength
b_c = inclination factor of foundation base
s_c = shape factor of foundation
i_c = inclination factor of load applied
q = surcharge or effective overburden pressure above footing

ULS Design Using FEM

FEM code PLAXIS version 9.0 is used to model the footing founded on soft clay. Axis-symmetry analysis is performed.

Conservation of Area Method

Conservation of area method is adopted when computing the equivalent radius or diameter for a corresponding circular footing for use in an axis-symmetry model in PLAXIS. The conservation of area method is simply:

$$s^2 = \pi r^2 = \pi d^2/4 \qquad (2)$$

Where:
s = side of a square dimension
r = radius of a circle
d = diameter of a circle

Table 2 shows the equivalent radius or diameter of a square footing for use in an axis-symmetry problem, where a square footing shall be converted to that a circular one for axis-symmetry analysis.

Table 2. Equivalent radius or diameter of a square footing (undrained case)

DA	Square footing (m)	Equiv. radius (m)	Equiv. diameter (m)
DA-1(1)	1.7	0.96	1.92
DA-1(2)	1.7	0.96	1.92
DA-2	2.0	1.13	2.26
DA-3	2.0	1.13	2.26

Soil Model Adopted

For simplicity, the common elastic-perfectly plastic Mohr-Coulomb's model is used. Total stress parameters are used for undrained analysis. Poisson's ratio of 0.495 is used for the total stress analysis. The input undrained parameters used for the clay are presented in Table 3.

Table 3.Typical soil properties for Mohr-Coulomb soil model (undrained case)

DA	γ_{bulk} (kN/m^3)	c_u (kPa)	ϕ_u (^{o})	E_u (kPa)	K_o
DA1-(1)	18	30	0	9000	1
DA1-(2)	18	21.4	0	6420	1
DA2	18	30	0	9000	1
DA3	18	21.4	0	6420	1

In the undrained analysis, the relationship $E_u \approx Kc_u$ is adopted. For soft clays, K varies between 250 to 400 (Ong et al., 2006 and Chow et al., 1996). However, in this analysis, K of 300 is used and is expected not to yield any large differences in the computed results. Depending on which Design Approach is used, the E_u value varies with the factored or unfactored c_u value.

Structural element

Beam element is used to model the footing. The equivalent beam properties used to model the footing are tabulated in Table 4.

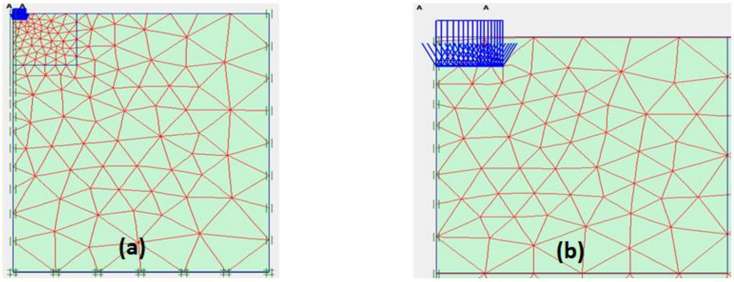
Table 4. Equivalent beam properties used to model footing

Item	Axial rigidity, EA (kN/m)	Bending rigidity, EI (kNm^2/m)	Relative weight, w (kN/m/m)
Footing	1.250×10^7	2.604×10^5	5.0

Boundary condition

The lateral and bottom boundaries of the finite element meshes should be far enough such that they do not interfere with the solution in the region of interest. Fig 2 shows the extent of the problem modelled. The hydraulic boundary condition is represented by a phreatic surface located at the ground surface.

Fig. 2. (a) Finite element mesh used in Case Study 1, (b) magnification of mesh in contact with footing



FEM Mesh

PLAXIS uses 3 integration points per 6-noded triangular element and 12 integration points per 15-noded element. In this FEM analysis, 15-noded elements are used. If 6-noded elements are used instead, the results are not expected to be very different as highlighted by Ong et al. (2006). Nevertheless, mesh refinement is done to the region that is in close contact with the footing elements for ease of computational convergence as shown in Fig 2.

In-Situ Stress Conditions

As the clay is soft, the initial co-efficient at rest, K_o of 1.0 is used. This implies that for a unit vertical stress imposed on the soil, a horizontal stress of similar magnitude is transferred to the surrounding soil. EN 1997-1 does not provide strict guidance of design values of initial stress values in ULS calculations. In the spirit of EN 1997-1, characteristic values should be advocated (Bauduin, n.d.).

Load Application

An equivalent uniform distributed load is applied instead of a centric point load to ensure even distribution and uniform contact of footing with the ground.

Interface Elements

Interface elements are not mandatory to be used here as development of shear stresses is expected to be minimal as the footing is rigid without the presence of lateral load.

Comparison and Discussion of Results

Fig 3 shows the deformation under the footing when stressed. The deformation pattern shows a pressure bulb that radiates outwards stressing the surrounding soils. The comparison of stresses experienced by the soil immediately under the footing using (i) analytical calculation as presented in Frank et al. (2004) and (ii) the stress prediction from PLAXIS axis-symmetry analysis for undrained condtion is shown in Table 5 for all the 3 EN1997-1 Design Approaches. In Frank et al. (2004), the design pressures are derived using Eq. (1), whereas for PLAXIS, the stress distribution is taken by 'cutting a cross-section' through the output mesh immediately beneath the footing.

Fig. 3. Development of pressure bulb and settlement contour beneath footing

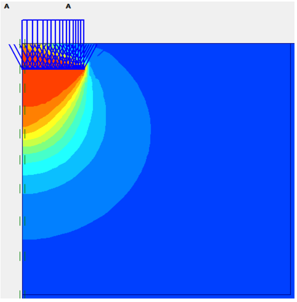


Table 5. Comparison of stresses experienced by soil immediately under the footing using (i) analytical calculation and (ii) stress prediction from PLAXIS axis-symmetry analysis, for undrained condition

DA	Equiv radius (m)	Frank et al. (2004) Analytical calculation	Predicted from FEM axis-symmetry analysis	Percentage difference of analytical calculation to FEM
		Design pressure, R_d/A' (kPa)	Stress distribution taken below footing by 'cutting a cross-section' (kPa)	
DA-1(1)	0.96	203	185	+8.9%
DA-1(2)	0.96	150	149	+0.67%
DA-2	1.13	203	170	+16.3%
DA-3	1.13	150	144	+4%

+ % difference denotes solution of analytical calculation is greater than FEM solution.

By using the conservation of area method to compute the equivalent diameter of the footing for axis-symmetry FE analysis, the results yielded are rather comparable to the analytical calculation with the largest percentage difference of about 16.3% for DA-2. The remaining Design Approaches computed by both the analyses show less than 10% difference.

Frank et al. (2004) defines the equivalent deterministic Overall Factor of Safety (OFS) as the ratio of characteristic resistance (R_k) to the actions (P_k+Q_k+G_{pad}) acting on the footing. These actions have to be factored up accordingly if checks are carried out using the various Design Approaches as shown in Table 1. In order to compare the output of the analytical calculations to that of FEM, the equivalent resisting force is obtained by 'cutting a cross-section' through the output mesh immediately beneath the footing.

The resisting force given by PLAXIS is in unit of kN/m/rad due to the axis-symmetry condition imposed on the analysis. By multiplying this force with 2πr (circumference of a complete circle for an axis-symmetry analysis), the corresponding and comparable resisting force is 'recovered' for 'apple-to-apple comparison' as presented in Table 5. The magnitude of actions acting on the footing remains unchanged. As such, the OFS can now be calculated and thus presented in Table 6. It is noted that the OFS for both analytical calculation and FEM output are rather comparable.

Table 6. Comparison of equivalent deterministic Overall Factor of Safety (OFS) for (i) analytical calculation and (ii) force prediction from PLAXIS axis-symmetry analysis, for undrained condition

DA	Equiv radius (m)	Frank et al. (2004) Analytical calculation			Predicted from FEM axis-symmetry analysis			
		R_k (kN)	$P_k+Q_k+G_{pad}$ (kN)	OFS	F kN/rad	R_k (kN)	$P_k+Q_k+G_{pad}$ (kN)	OFS
DA-1(1)	0.96	586	402	1.46	93	561	402	1.40
DA-2	1.13	812	426	1.91	109	774	426	1.82
DA-3	1.13	812	426	1.91	120	852	426	2.00

SLS design

Intuitively, since the OFS is much greater than unity (in ULS design, a value of 1.0 is deemed appropriate), is it not possible that the size of the footing be reduced to bring the OFS value closer to unity in order to save cost? The answer lies with the serviceability response of the footing and this needs to be checked. To illustrate this point, two SLS FEM runs are produced where all partial factors are neglected in a true, typical FEM analysis.

In the first case, a footing with an equivalent radius of 0.96m (same as DA-1(1)) is analysed and a corresponding settlement value of 31.7mm is observed. If the limiting settlement is 25mm, this would have been exceeded. In the second case, if a footing with an equivalent radius of 1.13m (same as DA-2 and DA-3) is analysed instead, the output settlement is 19.0mm which is deemed acceptable as it is less than the limiting value of 25mm.

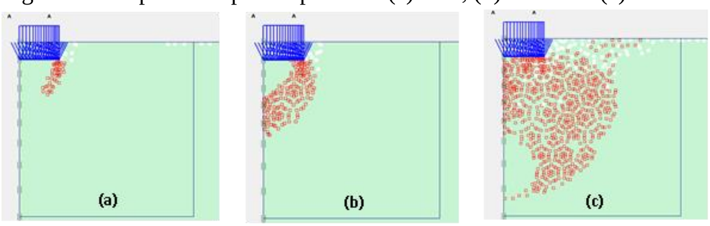
These simple examples illustrate that in a complete ULS design methodology, both ULS and SLS have to be checked for strength and serviceability, respectively, as soil-structure interaction cannot be conveniently separated. In the ULS design, the structural detailing of the footing would have been designed to resist factored (thus, greater) loads, while the size of the footing remains larger to ensure the serviceability criteria is fulfilled. This is the basis of what EN1997-1 is trying to impart and has been successfully demonstrated by the use of FEM.

Discussion

As described by Frank et al. (2004), geotechnical action such as earth pressures in FE analyses cannot be factored at source because they intervene with both action and resistance of the earth pressures by introducing artificial yielding. As such, the manipulation of earth pressures in FEM is minimised unlike in the LEM design where the calculation of factored earth pressures represent an important part of the analysis. The clear disadvantage of using analytical calculation in computing ULS bearing capacity is that settlement cannot be ascertained directly. One has to resort to empirical method such as Janbu's method (based on finite soil thickness) to compute the equivalent magnitude of settlement. Another drawback is that empirical method may not easily work on non-homogenous or sloping subsurface soil profile. Nonetheless, empirical checks serve the requirement of providing estimates or 'ball-park magnitudes' to cross-check with FEM output to retain the 'feel' of the problem. Where c'-ϕ' reduction method is to be used, a simple elastic-perfectly plastic soil model is recommended to be used (Bauduin, n.d.).

Fig 4 (a), (b) and (c) show the development of plastic points when analysed in various design states of SLS, ULS and OFS, where c'-ϕ' reduction method is used.

Fig. 4. Development of plastic points at (a) SLS, (b) ULS and (c) OFS states



IDEALISED CASE STUDY 2: ANCHORED SHEETPILE WALL EMBEDDED IN CLAY SUPPORTING A DEEP EXCAVATION

Appreciation of Problem

Case study 2 described herein is a reproduction of Example 9.2 found in Frank et al. (2004). Fig 5 shows that the sheetpile retaining wall has a total excavation depth of 5.4m considering an over-excavation of 0.4m. The wall is supported by one row of anchorages at elevation -1m (anchorage inclination of 10 deg). Surcharge of 10kPa is applied at ground surface on the retained side. For excavation in stiff clays, drained conditions are usually critical for stability, thus effective stress analyses are performed using effective soil parameters and steady-state hydraulic conditions. The water table in the sand layer is assumed to remain at an elevation of -1m where the ground level behind the wall is at elevation 0m.

The anchor and sheetpile wall parameters are found in Tables 7 and 8, respectively. The following hydraulic conditions are to be satisfied:
(i) Due to the excavation, the water table in the pit is lowered to an elevation of -5.4m. The gravelly sand layer (layer A) is assumed to maintain hydrostatic condition, while the total hydraulic head difference develops within the stiff clay.
(ii) The linear head loss is assumed along an idealised flow path starting from elevation -4m (behind the wall), going around the toe of the wall and exiting at elevation of -5.4m in front of the wall. This method ensures that the water pressure at the toe of the wall (both sides) is in equilibrium.

Fig. 5. Case Study 2 – Anchored sheetpile wall (Frank et al., 2004)

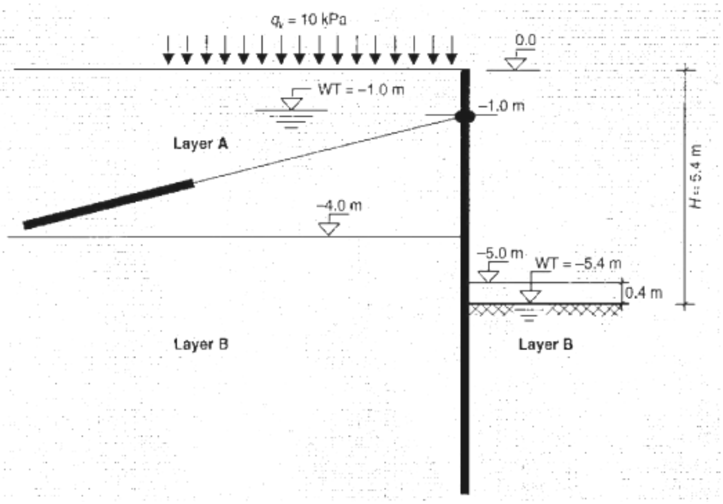


Table 7. Parameters for anchors

Item	Unit	Value
Young modulus of steel, E	kN/m^2	2.1×10^8
Cross-sectional area, A	cm^2/m	1
Pre-stress, P_o	kN/m	100

Table 8. Parameters for sheetpile wall

Item	Unit	Value
Bending stiffness, EI	kNm^2/m	5×10^4
Yield strength	MPa	355

ULS design based on Limit Equilibrium Method (LEM) and spring model

The ULS design of the sheetpile wall is performed using the Limit Equilibrium Method (LEM) and spring model as outlined in detail in Frank et al. (2004).

The LEM calculation procedure as adopted by Frank et al. (2004) is summarised as follows:
(i) Permanent actions (limiting active earth pressures and net water pressure), unfavourable variable actions (surcharge) and soil parameters enter the calculations with partial factors applied according to Table 1.
(ii) The overturning moment is calculated, which is the sum of the moments of the active earth pressure and the net water pressure with respect to the anchorage point.
(iii) The horizontal component of the limiting earth pressure and the stabilizing moment are calculated with respect to the anchorage point.
(iv) The wall embedment is determined from the ULS requirement.
(v) The characteristic value of the anchor force is determined by checking the horizontal equilibrium of the actions and resistances.
(vi) Finally, the characteristic values of the bending moment along the wall are calculated from the known actions, anchor force and earth pressure. The design values of the anchor force and bending moment are obtained by multiplying the corresponding partial factors are shown in Table 1.

The methodology for spring model as adopted by Frank et al. (2004) is summarised as follows:
(i) The wall embedment is determined with the standard calculation similar to the LEM method.
(ii) The spring model is employed to determine the internal forces and the anchor force.
(iii) Finally, the calculated values of the bending moment and anchor force are multiplied by corresponding partial factors for unfavourable actions to determine their design values.

ULS Design Using FEM

FEM code PLAXIS version 9.0 is used to model the deep excavation supported by sheetpile wall. Plane-strain analysis is performed.

Soil Model Adopted

It is acknowledged that more sophisticated soil model for soft soil may be more appropriate in typical SLS analysis, but under current circumstance where ULS design is involved and for simplicity, the common elastic-perfectly plastic Mohr-Coulomb's model may be a good starting point as recommended by Bauduin (n.d.) and Simpson (2007).

Soil Parameters Adopted

The unfactored and factored soil strength parameters for Layers A and B are shown in Table 9. The partial factor of safety used in the calculation of the factored soil parameters are shown in Table 1.

Table 9. Unfactored and factored soil parameters

Item	Unit	Unfactored - Layer A	Factored - Layer A	Unfactored - Layer B	Factored -Layer B
Dry density, γ_k	kN/m^3	18	18	-	-
Saturated density, γ_{sk}	kN/m^3	20	20	20	20
Friction angle, ϕ'_k	deg	35	29.3	24	19.6
Cohesion, c'_k	kPa	0	0	5	4
Wall-ground interface co-efficient, δ_k	deg	23	19.6	Active side: 16 Passive side: 12	13.1
Subgrade modulus, k_h	kN/m^3	10000	-	6000	-
Young's modulus, E (used in FEM only)	kN/m^2	-	8000		4800
Co-efficient of earth pressure at rest, K_o	-	0.5	0.5	0.95	0.95

Length of Wall

In order to provide an 'apple-to-apple' comparison, the lengths of wall embedment used in the FE analysis is maintained similar to the lengths found in Table 10, which are determined through the LEM method in Frank et al., (2004).

Table 10. Length of wall embedment according to various EC7 Design Approaches (Frank et al., 2004)

EC 7 Design Approach	Length of wall embedment (m)	Total length of wall (m)
DA-1(1)	6.62	12.02
DA-1(2)	6.62	12.02
DA-2	7.89	13.29
DA-3	6.62	12.02

Boundary Conditions

A concern in FE modelling of geotechnical problems is the extent of the meshes. The lateral and bottom boundaries of the finite element meshes should be far enough such that they do not interfere with the solution in the region of interest. It has been found that if the lateral and bottom extents are at least 3.5 times the excavation depth, the boundary effects can be negligible (Ong et al., 2006). Fig 6 shows the FE mesh used in the analysis.

Hydraulic Boundary Conditions

In modelling this excavation process, the hydraulic boundary conditions are perhaps the most challenging requirement to apply to the FE model. As it is required (i) to maintain hydrostatic condition in the sand layer, (ii) to generate total hydraulic head difference within the stiff clay and (iii) to ensure that the water pressure at the toe of the wall at both sides is in equilibrium, normal application of groundwater by means of user-defined phreatic line in PLAXIS cannot be used as this would have generated hydrostatic conditions in both the sand and clay layers instead, without having the water pressure balanced at the toe of the sheetpile.

Therefore, the 'ground water flow' function in PLAXIS needs to be used to simulate similar ground water condition as per the problem in hand so that 'apple-to-apple' comparison can be made. In order to develop the ground water flow in the FE model, the hydraulic boundary conditions have to be manually defined in both (i) the initial condition stage and (ii) relevant construction stages, by specifying the levels of water heads driving the ground water flow from the retained side to the excavated side of the soil. The net water pressure approximates zero at the toe of the wall as shown in Fig. 7, as the water pressure distributions on both sides of the sheetpile is in equilibrium.

Fig. 6. FE mesh used in the analysis of Case Study 2

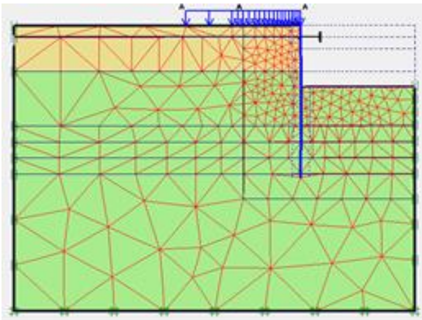
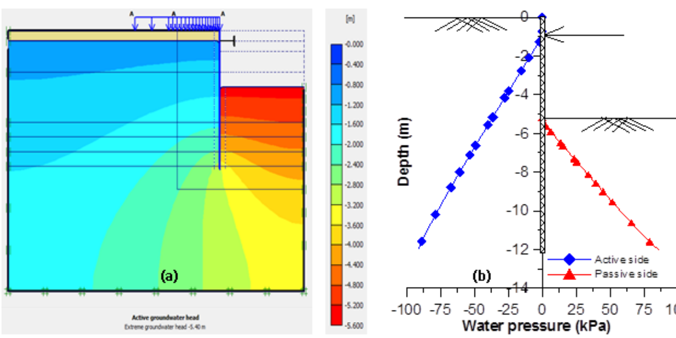


Fig. 7. (a) Hydraulic boundary condition and ground water head (equipotential lines) for FE analysis and (b) equilibrium water pressure distribution on active and passive sides of excavation



In-Situ Stress Conditions

In-situ stress conditions of K_o equals 0.5 for the sand layer and 0.95 for the clay layer are used to generate the initial stress conditions of the soils.

Structural Element

Beam element is used to model the wall. Table 11 shows the beam properties used to model the sheetpile. The anchor axial rigidity used is also found in Table 11.

Table 11. Equivalent beam and anchor properties

Item	Axial rigidity, EA (kN/m)	Bending rigidity, EI (kNm^2/m)	Relative weight, w (kN/m/m)
Sheetpile wall	3.340×10^6	5.0×10^4	0.970
Anchor	2.1×10^4	-	-

Interface Elements

In order to reflect the actual soil and sheet pile wall interaction behaviour, slip interface has been modelled with realistic value of 0.67. Interface elements or slip elements are used in finite element analyses to simulate sliding between two different materials. These elements have thin width and shear stiffness comparable or less than the surrounding materials.

Comparison and Discussion of ULS Results

As ULS design involves usage of partial factors on soil parameters and/or actions, the subsequent forces generated in the structural elements are used for sizing and for strength design only. The resulting deflections (serviceability criteria) of these structural elements would then not reflect the actual behaviour and thus, shall be neglected.

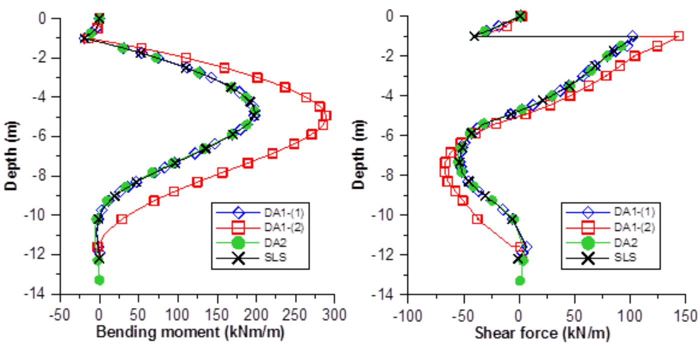
Limit Equilibrium Method (LEM) involves the calculations of equilibrium of moments, vertical force components and horizontal force components. This method ensures the compatibility of moments and forces, thus the stability of the overall system against failure. In such analysis, factor of safety of unity is sufficient.

In the FE analyses, DA-1(1), DA-1(2) and DA-2 whose corresponding partial factors as defined in Table 1, are carried out. The reason why DA-3 is not carried out is because it is expected to yield similar output response to DA-1(2) since both these methods are based on the Materials Factoring Approach (MFA) (Bauduin, n.d.) with identical wall embedment length of 12.02m as shown in Table 10. DA-1(1) and DA-2 are based on the Load Resistance Factoring Approach (Bauduin, n.d.) with different wall embedment length and as such, separate analyses have to be performed.

It is obvious that for this particular excavation problem, the MFA approach of DA1-(2) yields greater bending moment and shear force as shown in Figs 8(a) and 8(b), respectively, when the soil parameters are factored down as compared to the LRFA approach of DA1-(1) and DA2 where the surcharge is the only parameter factored up as all soil parameters remain as characteristic values. This indicates that the wall responses are more sensitive to the MFA than the LRFA method.

In the FE analysis, wall length for DA2 is slightly longer than that of DA1-(1), but this has no effect on the bending moment and shear force distribution along the wall as shown in Figs 8(a) and 8(b), respectively. This phenomenon is explained in more detail under the SLS loading condition.

Fig. 8. (a) Bending moment and (b) shear force profiles, as computed by FE analyses (before multiplying partial factor of 1.35 for LRFA methods – DA-1(1) and DA-2)



As the spring model utilises net water pressures on the wall, the actual distribution of water pressure is not shown in Table 12. However, when the hydraulic boundary conditions in FEM are carefully accounted for as described earlier, the water pressure distribution obtained seems to agree reasonably well with the LEM method of computation as can be seen in Table 12.

The comparisons of numerical values of various maximum design force and pressure components computed based on LEM, spring model and FEM are shown in Table 12. The factored bending moment, shear force and anchor force are calculated by multiplying the maximum calculated values with partial factor of 1.35 for LRFA methods only, namely, DA-1(1) and DA-2. In general, the outcome shows that the magnitudes of FEM bending moments are the smallest amongst the corresponding Design Approaches of the three types of analytical methods. However, even though the corresponding FEM horizontal anchor forces are, in general - except for DA2, greater than the values based on LEM method, the difference in magnitudes are not as great as the bending moment magnitudes. It may be intuitively correct to assume that due to the re-distributed active pressures along the wall when it deflects as compared to one that does not in the case of

LEM, the horizontal anchor picks up a greater portion of the ‘balancing’ force as compared to the wall.

It is worthy to note that the elevations of maximum bending moment and shear force between the LEM and FEM methods are reasonably comparable, providing an in-sight to the anticipated reasonable compatibility of the bending moment and shear force profiles. The design values of bending moments and anchor reaction obtained using the spring model or FEM are much lower than the values obtained using the LEM due to the fact that the spring model uses characteristic values of the earth resistance and the partial factors on the earth resistance are only introduced at the end to check the mobilisation level (Frank et al, 2004).

Table 12. Comparison of various maximum design force and pressure components computed based on LEM, spring model (Frank et al., 2004) and FEM

Item	Unit	Frank et al. (2004)					Analysed			
		LEM			Spring model		FEM			
		DA1-(2)	DA2	DA3	DA1-(1)	DA2	DA1-(1)	DA1-(2)	DA2	DA3
Wall embedment below max. exc.	m	6.62	7.89	6.62	12.02	13.29	6.62	6.62	7.89	6.62
Length of sheet pile	m	12.02	13.29	12.02			12.02	12.02	13.29	12.02
Characteristic PWP										
Elevation -1.0m (active side)	kPa	0	0	0			0	0	0	0
Elevation -5.4m (passive side)	kPa	0	0	0			0	0	0	0
Elevation -4.0m (active side)	kPa	30	30	30			26.3	26.3	26.3	26.3
Elevation -5.4m (active side)	kPa	39.8	40.4	39.8			38.2	38.2	38.2	38.2
Toe of wall (both sides)	kPa	86.1	99.1	86.1			91	91	91	91
Design value of the horiz. component of anchor force	kN/m	172	228.5	172			199.53	179.4	202.77	179.4
Max. BM in wall	kNm/m	446.7	649.5	446.7	347	283	267.1	288.5	266.0	288.5
Elevation of max. BM	m	-5.4	-5.4	-5.4			-4.9	-4.9	-4.9	-4.9
Max. shear force in wall	kN/m	159.1	215.7	159.1	170	157	138.0	143.7	145.2	143.7
Elevation of max. shear force	m	-1	-1	-1			-1	-1	-1	-1

The benefit of performing analyses using spring model and FEM is that such methods enable the re-distribution of active pressures to account for wall deflections, which otherwise would be taken fully by a 'rigid, undeformed wall' in a LEM analysis. This may provide an explanation as to why LEM analysis would generate upper-bound forces compared to both the spring model and FE analyses.

The use of slip elements or interface elements in FE analyses enables a more rigorous pressure distribution method in the soil-wall interface, which the spring model cannot perform. In order to reflect soil-wall interaction behaviour, correct use of slip elements and evaluating their realistic values form an integral part of numerical modelling. Slip elements are used in FE analyses to simulate sliding between two different materials. These elements have thin width and shear stiffness comparable or less than the surrounding materials (Ong et al, 2006). Details of using slip elements can be found in Desai et al. (1984) and Griffiths (1985). In this particular example, wall-ground interface co-efficient of 0.67 is used. It is expected that when wall-ground interface co-efficient is used in the FE analysis, realistic slippage between wall and soil can readily occur, thus reducing the active pressures that can actually act on the wall. This may be another why FE output forces are smaller than those of spring model.

Comparison and Discussion of SLS Results

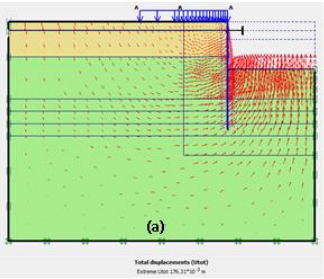
In a typical Serviceability Limit State (SLS) analysis, the design values of all actions, resistances, soil parameters, wall and anchor stiffnesses are equal to their characteristic (unfactored) values.

Serviceability criteria such as wall displacements and ground deformations can be effectively analysed using FE analyses, where soil-structure interaction can be studied in detail. By virtue of connecting the structural elements to the soil elements through the FE meshes, the deformation of one would lead to the response of another, thus the soil-structure behaviour can be effectively analysed. For example, a typical FE model, being two-dimensional, also allows for ground deformations behind the wall to be analysed directly in order to verify the serviceability conditions of any supported structures and utilities.

Such complex soil-structure interaction (structural element-wall displacement-retained ground deformation), unfortunately, cannot be analysed using one-dimensional analytical model such as the spring model. The spring model can only be used to estimate the wall displacements, and not the retained ground deformation, as the soil is modelled as a series of elasto-plastic (elastic-perfectly plastic or p-y curve) springs yielding at the active and passive earth pressures. In such circumstance, some form of calibration needs to be carried out prior. This means that the soil spring stiffnesses are obtained by considering the wall movement required to mobilise the active and passive earth pressures.

Figs. 9(a) and 9(b) show the vectors and contours of the total displacements of the sheetpile and the surrounding soils as a result of the excavation process, respectively. The settlement as a result of surcharge at the retained side and heave at the excavation formation level as a result of stress relief are some of the quantities that can be analysed using FEM, but not with LEM nor the spring model.

Fig. 9. (a) Vectors and (b) contours of total displacements as a result of excavation process (SLS)



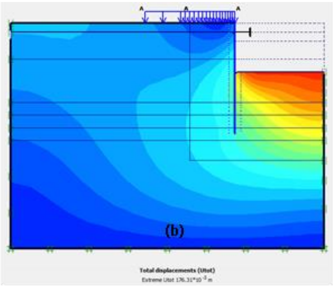
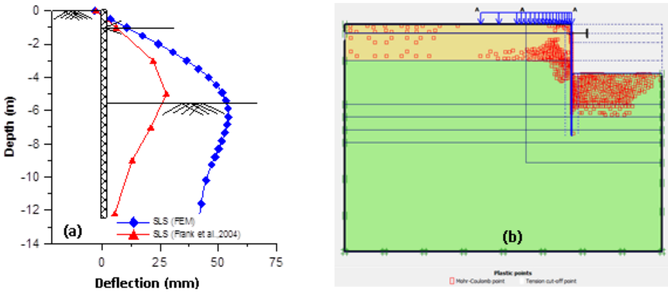


Fig. 10(a) shows the comparison of SLS displacement profiles between the analyses done using FEM and spring model by Frank et al. (2004). It is observed that FEM analysis gives a much larger deflection than the spring model. Besides, relatively large magnitude of heave (170mm) is also observed at formation level resulting in large wall toe displacements for the FEM analysis as observed in Fig 10(a). Fig 10(b) shows the development of plastic points after the excavation process. It is noted that most parts of the soils under the formation level has turned plastic. Such observations, however, are not observed when analysis was done using the spring model simply because the spring model could not be used to analyse the behaviour of the surrounding soil as a result of the excavation process.

One possible explanation on the reason behind the relatively large wall toe deflection (seems to be independent of length of sheetpile wall) in FEM could be due to the under-estimation of the unloading stiffness of the clay at the floor of the excavation. It is known that the unloading stiffness of clay can be as high as 2 to 3 times its average stiffness. Long (1997) reported a value of 4.5 times the average stiffness E_{50} was obtained through back-analysis of a case history where field observation using heave stakes was used to monitor the heave at formation level of the basement excavation. However, such unloading stiffness could not be input in a simple elastic-perfectly plastic soil model unless a more sophisticated non-linear soil model that has the capability to analyse strain hardening effect due to unloading is used in lieu, which is not the scope of this paper.

Fig. 10. (a) Comparison of SLS displacement profiles between analyses done using FEM and spring model and (b) development of plastic points after the excavation process modelled in FEM



It is therefore highlighted that capability in execution of FE analysis alone is not sufficient to make overall judgment as experience and field observation methods are also crucial to ensure the success of a safe deep excavation design.

CONCLUSIONS

Two cases of finite element (FE) analyses, which simulate the cases of (i) a square footing on soft clay and (ii) an anchored sheetpile wall embedded in clay supporting a deep excavation, have been presented to study the applicability of such analysis in Ultimate Limit State (ULS) design based on Eurocode 7.

In the first case study involving a footing on soft clay, the stress distribution beneath the footing and the Overall Factor of Safety (OFS) computed using FEM are rather comparable to the calculations produced by analytical method using Limit Equilibrium Method (LEM), where an established bearing capacity equation and associated parameters are used. For completeness of the design of a footing, Serviceability Limit State (SLS) has also been carried out to assess the settlement potential of the footing of the chosen size.

For the second case study involving an anchored sheetpile wall supporting an excavation, the wall responses seem to be more sensitive to the Materials Factoring Approach (MFA) than the Load Resistance Factoring Approach (LRFA). In general, the outcome shows that the magnitudes of FEM bending moments are the smallest amongst the corresponding Design Approaches of the three types of analysis methods used namely, Limit Equilibroum Method (LEM), spring model and Finite Element Method (FEM). Soil-structure interaction and stress distribution along the wall is imperative and reflective of actual condition and thus, analysis using FEM may help shed light on the overall complex soil-structure interaction behaviour of the system. Serviceability criteria such as wall displacements and ground deformations can be effectively analysed using FE analyses.

From the studies carried out hrein, it can be concluded that FEM has a place in ULS design for strength and SLS design for serviceability of any geotechnical structure. However, its application must be prudently scrutinised especially when complex soil model is adopted as it may involve development of artificial yield surfaces, thus increasing the complexity of interpretation of the problem in hand. The current FE analyses performed herein are limited to the simple elastic-perfectly plastic soil model. It is to be borne in mind that capability in execution of FE analysis alone is not sufficient to develop overall judgement, as experience and field observation methods are also crucial to ensure the success of any analysis and execution of geotechnical works.

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