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Eurocode 7, design approach 1, for design of deep excavations in South Africa – a comparison between working stress and limit state design

Eurocode 7, Approche De Conception 1, pour la conception de fouilles profondes en Afrique du Sud – Une comparaison entre les l'analyse aux états limites et la contrainte de fonctionnement

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ABSTRACT: Eurocode 7 (EN1997-1), will be adopted in South Africa as the geotechnical design code, replacing working stress design, with limit state design. EN1997-1 design approach 1(DA1) is the preferred approach in South Africa with a special National Annex for South African practice. DA1, Combination 1 and 2 applies partial factors on characteristic shear strength properties and actions to reduce soil shear strength and increase the applied actions. Limit State Design applies a "safety" factor at the source of uncertainty as various shear strength parameters and categories of actions have various sources and levels of uncertainty. Working Stress Design (WSD) places a safety factor on the overall resistance or structural load which could result in less reliable designs. EN1997-1 DA1 will be adopted in the revised South African Lateral Support in Surface Excavations Design code. The use of EN1997 for deep excavation design has been critiqued by numerous authors stating the approach is conservative and has not been calibrated for deep excavation design. This paper compares four deep excavations, designed using working stress with a Eurocode 7 compliant solution.

RÉSUMÉ : L'Eurocode 7 (EN1997-1) sera retenu en Afrique du Sud comme norme géotechnique de référence, remplaçant l'étude aux contraintes de fonctionnement par l'analyse aux états limites. L'approche de calcul retenue est l'approche 1 avec une Annexe Nationale sud-africaine spécifique. Suivant l'Approche 1, les combinaisons 1 et 2 appliquent des coefficients partiels sur la résistance caractéristique au cisaillement et sur les actions pour réduire la résistance en cisaillement et augmenter l'effet des actions appliquées. Le calcul aux états limites applique un coefficient de sécurité sur chaque source d'incertitude car les paramètres de résistance au cisaillement et les catégories d'actions ont des niveaux d'incertitude différents. L'approche aux contraintes de fonctionnement ne génère qu'un facteur global sur la résistance ou sur les actions ce qui peut entrainer une conception moins fiable. L'approche 1 de l'EN1997-1 sera retenue sur la mise à jour du code South African Lateral Support in Surface Excavations. L'utilisation de l'EN1997 pour l'étude des excavations profonde a été critiquée par de nombreux auteurs qui affirment que l'approche est trop conservative et qu'elle ne pas été calibrée pour l'étude des excavations profondes. Cette publication compare sur quatre cas d'études d'excavations profondes le dimensionnement suivant la contrainte de fonctionnement avec une approche conforme aux Eurocodes 7.

KEYWORDS: Deep Excavations, Soldier Piles, Limit State Design, Working Stress Design, Eurocode 7

1 INTRODUCTION

The decision to adopt EN1997 was taken by the geotechnical community at a meeting held by SAICE Geotechnical Division in November 2017. At this meeting it was also decided to update the Lateral Support in Surface Excavations (LSSE) Code of Practice (1989) as a non-contradictory complementary code of practice to EN1997-1 for the design of deep excavations. The LSSE code of practice was last updated in 1989 and is currently based on WSD. The aim of this paper is to review the design of four excavations designed in accordance with WSD with that of the equivalent EN1997-1 compliant design.

2 CONVENTIONAL APPROACH - WSD

The current SAICE Lateral Support in Surface Excavation code of Practice was last updated in 1989 and allows for design using Working Stress Design methods. A survey was undertaken and forwarded to deep excavation designers in South Africa to establish which methods they typically use to design deep excavations. The results of this survey are detailed in van der Merwe & Konstantakos (2020) and van der Merwe & Parrock (2021). From this survey it was established that design methods used to design lateral support systems, in particular multi-anchored soldier pile walls, vary significantly between designers. These soldier pile walls typically consist concrete piles with stressed grouted anchors and shotcrete infill panels (lagging). Methods include using apparent earth pressures, wedge stability analyses, Beam-On-Elastic Winkler spring analysis and Finite element analysis. Some designer's methods result in a more rectangular/trapezoidal earth-pressure stress distribution, whilst others' result in a more triangular earth pressure stress distribution. Some designers do not apply pre-stress but use soil nails to retain contiguous piled walls.

Typically, lumped FoS in WSD varied depending on the design life. For overall stability an FoS of 1.25 to 1.5 are typically assumed for temporary and permanent systems respectively. Moments, shears and axial loads in soldier piles are conventionally calculated for the unfactored loads, multiplied by appropriate load factors, and compared to ULS design resistance of a circular column. Typically, the WSD/SLS structural forces (bending moments and shear forces) are increased by a factor of safety of 1.5 – 2.0 (AS4678 and Konstantakos (2019)) and compared to the ULS design resistance of the structural section.

3 LIMIT STATE DESIGN USING EN1997

To understand why structural forces or embedment requirements might differ between WSD and Limit State Design using partial factors, it is important to appreciate how EN1997-1 accounts for uncertainty and how it aims to achieve a certain level of reliability.

Hansen (1965) first proposed that partial factors should be selected considering a) larger value to more uncertain quantity, and b) to maintain approximately the same dimensions using traditional WSD practice. The partial factor method is basically a semi-probabilistic approach in which partial factors must be calibrated to achieved uniform reliability levels across different design scenarios (Phoon and Retief, 2015). The partial factors in EN1997 have been chosen to give similar design solution to those using Factors of Safety ensuring that wealth of previous experience is not lost by the introduction of a radically different design method. Partial factors aren't calibrated, according to Phoon and Retief (2015), by reliability analysis and hence not a simplified RBD approach when viewed from a historical context of standardisation for structural design. The geotechnical engineer can exercise his/her discretion to adjust the characteristic value of a particular geotechnical parameter to suit a particular site and other localised aspects of geotechnical practice, and is therefore subjective.

Ideally, the designer should derive the characteristic value of a geotechnical parameter taking account of inherent variability of the ground on a specific site, the uncertainty in the determination of the soil parameters and the extent of the failure mechanism (Orr, 2016). In geotechnical design using EN1997, a characteristic value is selected subjectively by the designer on the basis of a 5% likelihood of a worse value affecting the limit state. A partial factor should then be applied to this characteristic value (X_k) to derive a design value (X_d). The partial factor should provide for the level of reliability or safety required by public, as close as possible to the target reliability of the country (Orr, 2016). In South Africa the target reliability index is 3.0 for ductile modes of failure. The partial factors aim to provide safe design against unfavourable deviations of the ground strength properties from their characteristic value and against uncertainties in the calculation model

Geotechnical design is performed under a considerable degree of inherent uncertainty when compared to other engineering designs and therefore it is often considered inappropriate to assign a single partial factor to characteristic values that are derived subjectively.

Criticism in embedded pile wall design using EN1997-1 is directed to the significant increase in length and significant increase in bending moment, especially in soft clays. A thorough experiment was therefore undertaken to establish if the design factored undrained cohesion value could be assumed reasonable. The characteristic mean can be estimated using:

$$x_k = \bar{x}(1-c_v\eta_k) \quad (1)$$

The standard deviation of log-normal distribution is given by (Bond and Harris, 2008):

$$\zeta = \sqrt{ln\left[1+\left(\frac{\sigma_X}{\bar{x}}\right)^2\right]} \quad (2)$$

The mean of log-normal distribution:

$$\lambda_X = \ln(\bar{x}) - \frac{\zeta^2}{2} \quad (3)$$

and, the probability that a worse value than the design value will govern:

$$pf = \frac{Ln(X_d) - \lambda_x}{\zeta} \quad (4)$$

Assuming an η_k of -1 (Day and De Koker, 2020) for a small volume of ground, since there is less averaging, and a cv = 0.4 for undrained cohesion. With $\overline{c_u}$ = 66kPa, the σ_x = 66x0.4 = 26.4kPa and the characteristic value $c_{u,k}$ = 66 – (1x26.4) = 39.6kPa. This would result in a design value of $c_{u,d}$ = 39.6/1.4= 28.3kPa, ζ = 0.39, λ_X = 4.11 and pf = 2.0. When using η_k of -1.64 the pf increases to 3.45 which better aligns with the target reliability of EN1997-1 in the EU. This was probably the original intent when deriving these partial factors. When doing the same for effective friction the value is not constant and the associated pf is higher than that of undrained cohesion as shown in Figure 1.

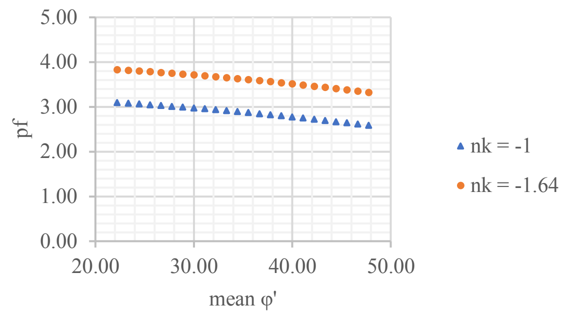

Figure 1. pf using different statistical coefficients

The reliability will therefore be influenced by the subjectivity of the design engineer and the number of tests undertaken and could result in unsafe design when undertaking too little testing whilst using a small statistical coefficient. Undrained test results (TX UU) can vary significantly even from the same horizon and in particular when derived using SPT-N results.

Schneider (1997) recommends that a characteristic value should be chosen half a standard deviation from the mean. This could result in unsafe designs if too little testing has been undertaken for a small volume of ground.

4 PROPOSED APPROACH – EN1997-1 LIMIT STATE DESIGN

Partial factors of actions are given in SANS10160-1 for South Africa. These partial factors on actions and shear parameters are summarised in Table 1.

Table 1. Partial factors on actions and soil parameters

Design Approach			
Action (SANS10160-1)	Symbol	DA1-1 (STR/ STR-P)	DA1-2 (GEO)
Permanent (Un-F)	γ_G	1.2/1.35	1.0
Permanent (F)	γ_G	0.9/-	1.0
Variable (Un-F)	γ_Q	1.6/1.0	1.3
Variable (F)	γ_Q	0/0	0
Soil Parameter (EN1997-1/SANS10160-5)			
Angle of shearing resistance	γ_ϕ	1.0	1.25
Effective Cohesion	$\gamma_{c'}$	1.0	1.25
Undrained shear strength	γ_{cu}	1.0	1.4
Unconfined strength	γ_{qu}	1.0	1.4
Weight Density	γ_γ	1.0	1.0

CIRIA C760 states that for embedded retaining walls, it is common practice to apply partial factors to the effects of action (bending moments and shear forces) rather than actions directly. Partial factors greater than unity should be placed on unfavourable variable surcharges, according to CIRIA C760, to ensure that double accounting does not occur.

CIRIA C760 states that for DA1-1 the analysis shall be undertaken with unfavourable variable surcharge using: $\gamma_Q q_{k,Q}$ = 1.11$q_{k,q}$ derived from 1.5/1.35 (UK partial factor for variable load divided by partial factor for permanent load). In South Africa both STR and STR-P needs to be checked in DA1-1 and the above equation would change to $\gamma_Q q_{k,Q}$ = 1.33$q_{k,q}$ and $\gamma_Q q_{k,Q}$ = 1.0$q_{k,q}$ for STR and STR-P respectively. The design effect of actions (wall bending moment, shear and anchor forces) should then be multiplied with a partial factor of 1.2 or 1.35 ($E_d = \gamma_E E_k$) for STR and STR-P respectively in DA1-1. Typically, STR-P will result in higher ULS design values as the variable component is typically small.

Serviceability Limit State checks are carried out using a partial factor of unity and characteristic design values in accordance with CIRIA C760. Using SANS10160-1 a partial factor of 1.1 should be placed on unfavourable surcharges.

Unfavourable (destabilising) and favourable (stabilising) permanent actions comes from a single source in deep excavation and therefore a single partial factor may be applied in accordance with EN1997-1 to the sum of these actions or their effects.

5 OVERDIG/UNPLANNED EXCAVATIONS

EN1997-1 states that the designer should allow for the possibility of unplanned excavation on the restraining side of the wall. This allows for uncertainty in the geometry.

The allowance should be the maximum value of H/10 (H is planned exposed height) or 0.5m unless tight excavation control with adequate engineering supervision is employed on site. Checks for overdig must occur at every stage of excavation as a separate analysis to confirm its effect. Keller is strict on ensuring the excavation does not proceed to depths deeper than 0.5m below the grouted anchor position by training site foreman and having regular toolbox talks before every day's work.

Allowance is currently not made for overdig in the SAICE LSSE (1989) Code of Practice and has resulted in insurance claims on some sites in South Africa.

6 PARTIAL FACTORS ON STIFFNESS

Eurocode 7 does not provide any partial factors to be applied on soil stiffness. This could likely be due to the difficulty in quantifying a parameter that is normally predicted from in-situ test results such as SPT-N and are highly strain dependent. As structural forces in SSI and FE analysis are highly dependent on stiffness, the reduction in stiffness is considered prudent by the authors.

CIRIA C580 states that one should use Young's Moduli values of approximately 50 percent of those adopted in the SLS for ULS analysis due to stress-strain dependency of stiffness and lower stiffness values associated with higher strain under ULS conditions. This recommendation seems to have been removed in CIRIA C760 (revision of CIRIA C580) but comes from the notion that partial factors are applied to peak shear values and not necessarily to ensure a certain reliability level is achieved.

Nonetheless a reduction in effective friction angle and effective cohesion, irrespective of the reason, is normally associated with lower stiffness values and should therefore be considered in analysis in the authors' opinion.

7 GROUTED ANCHORS/TIEBACKS

EN1997-1:2004/A1 contains a revised Section 8 on ground anchors/grouted anchors and should be read in conjunction with EN ISO 22477-5. No distinction is made between permanent and temporary anchors as currently in the LSSE. This is shown to result in more stringent requirements in the following sections, except when calculating the required fixed length. The methods will not be discussed here in detail but in the following sections. Important to note is that anchors should be designed for their bond strength to be greater than the tendon strength as this will result in less brittle failure.

Anchor prestress loads are normally entered as their characteristic values in both SLS and ULS SSI models as they are regarded as favourable actions.

8 17M DEEP EXCAVATION, GARDENS, CAPE TOWN

A 17m deep excavation, consisting 500mm CFA Soldier piles at 1.25m c/c spacing, and shotcrete infill panels, were constructed in Gardens, Cape Town (Figure 2). Stressed Self-drilling grouted anchors (R51N), spaced at 2.5m c/c were used to temporarily tie the lateral support system back. The grouted anchors, with a free length, were locked off to a working load of 500kN. The site is underlain by colluvial materials overlying residual shales, phyllites and greywacke of the Tygerberg Formation, Malmesbury Group. The ground watertable was encountered quite shallow at 2 to 5m below ground level and was drawn down during construction.

8.1 *Soil Properties*

An exponential elasto-plastic stiffness (Winkler-Spring) model was used, and the characteristic soil parameters predicted from SPT-N. Soil parameters used in the models are summarised in Table 2. The soil stiffness is therefore allowed to vary with depth to account for the effect of increasing average effective stress. A 12kPa surcharge was applied to the surface on the retained side of the excavation. An E_{ur}/E of 3 was assumed.

Figure 2. Photo of Deep Excavation, Gardens, Cape Town

Table 2. Characteristic Soil Parameter, Deep Excavation, Gardens Cape Town

Soil Type	Depth (m)	γ_t (kN/m^3)	φ' (deg)	c' (kPa)	E* (MPa)
Dense Colluvium	0-4	18.5	36.8	2	8.9
Dense Colluvium	4-14	18.8	38.1	0	12.5
Very Dense Residual Shale	14-16	19.2	40.6	0	21.1
Very Soft Rock Shale	>16m	21	45	89	120

*p_{ref} = 14kPa, 40kPa,44kPa and 31kPa for respective layers descending with depth

8.2 *Limit Equilibrium*

Limit Equilibrium analyses were undertaken using FHWA pressure and a CALTRANS beam element allowing for 20% negative moment, with results of soldier pile bending moment, M and anchor force, F detailed in Table 3. STR should not be critical as only a small component of variable load component is modelled and therefore only STR-P and GEO limit state was considered.

The WSD anchor forces calculated was 377kN, 310kN, 345kN, 412kN, 392kN and 197kN from the top to the bottom row of anchors respectively. The sum of the anchor forces is 2033kN. A lock-off load of 3000kN (500kN/anchor) was employed in the system to ensure an FoS of 1.5.

Table 3. Limit Equilibrium Analysis, Deep Excavation, Gardens, Cape Town

	M (kNm/m)	F (kN)	M/M_{WSD}	F/F_{WSD}	δ (mm)
WSD	136	413	1.00	1.00	8.9
WSD × 1.35	183	557	1.35	1.35	
DA1-1	209	585	1.53	1.41	
DA1-2	220	577	1.61	1.39	

8.3 *Soil-Structure Interaction*

Non-linear (Beam-on-Elastic (BOE)) SSI analysis were undertaken in DeepEx(2020) with pre-stress forces as detailed in 9.2 with output from one model shown in Figure 3. Result of the analyses are contained in Table 4. Bending moment from overdig is given in brackets.

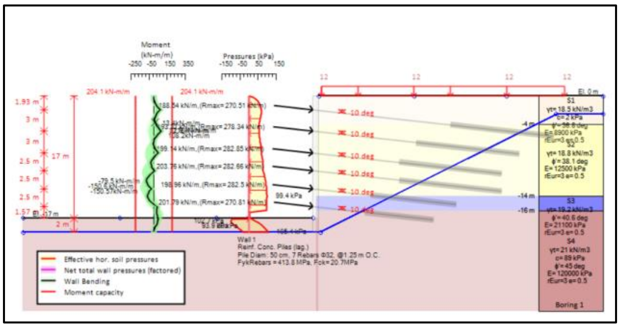

Figure 3. Elastoplastic SSI analysis, Gardens, Cape Town

Table 4. Soil -Structure Interaction Structural Forces, Deep Excavation, Gardens, Cape Town

	M (kNm/m)	F (kN)	M/M_{WSD}	F/F_{WSD}
WSD	121 (138)	524	1.00	1.00
WSD × 1.35	163 (186)	707	1.35	1.35
DA1-1	164 (188)	707	1.35	1.34
DA1-2	189 (217)	550	1.55	1.05

From the results it can be seen that the bending moments that develop in the soldier piles, when using EN1997- DA1, are 1.35 to 1.55 times larger than those predicted from the WSD analysis. This is considered reasonably comparable to conventional approach where the bending moment would have been multiplied with an FoS of 1.5 to compare with the ULS design capacity of the concrete section. When considering overdig and EN1997 the ultimate bending moment can be up to 1.8 times larger than the comparable WSD analyses (ignoring overdig). This illustrates the importance of good site supervision requirements on deep excavation projects or the allowance for overdig in design. The grouted anchor reactions are 1.05 to 1.35 times the WSD reaction load.

The same model was used, and the soil stiffness halved as per recommendations of CIRIA C580, with the results summarised in Table 5.

When comparing the WSD bending moment from Table 4 to DA1-2 from Table 5 the moment is roughly 1.67 times larger. This shows that a reduction in the soil stiffness has an effect on the derived moments.

Excessive bending moments and displacements can be reduced by reducing the pile spacing or reducing the vertical anchor spacing

An increase in vertical loads on the soldier piles due to the dead load of the supported slabs which augments the bending and shear resistance of the piles are typically only present after some time in bottom-up construction and should be modelled accordingly. Embedment depth should additionally be adjusted to account for vertical loads and the required pile capacity below BEL.

Table 5. Soil-Structure Interaction Structural forces with Stiffness halved

	M (kNm/m)	F (kN)	M/M_{WSD}	F/F_{WSD}
WSD	140	537	1.00	1.35
WSD × 1.35	189	724	1.35	1.35
DA1-1	188	727	1.34	1.34
DA1-2	203 (233)	562	1.45	1.05

8.4 *Grouted Anchor Requirements (Chapter 8, EN1997-1 Corrigendum)*

The design of the grouted anchors was checked to comply with the new section of EN1997-1:2004/A1. The ultimate force to be resisted by the grouted anchors are:

$$E_{ULS,d} < R_{ULS,d} \tag{5}$$

where $E_{ULS,d} = \max(F_{ULS,d}; F_{Serv,d})$ (6)

$= \max(707;\ 524 \times 1.35) = 707\text{kN}.$

Acceptance test shall be undertaken on all anchors to a proof load of:

$$P_P = \gamma_{a,acc;ULS} = 1.1 \times E_{uls,d} \tag{7}$$

$= 1.1 \text{ x } 707 = 777\text{kN}.$

The bar strength $R_{t,d}$, should be:

$R_{t,d}$ (R51N) = 630/1.15 = 547kN < $E_{ULS,d}$ and is therefore inadequate. The bar diameter would need to be increased to a T76 or the anchor spacing reduced.

Overstressing of the anchor is allowed to $0.95f_y$ in accordance with Clause 5.10.2.1 of EN1992-1-1:2005 during proof loading. The difference can be attributed to the fact that conventionally SAICE LSSE recommended a FoS of 1.6 on temporary anchors ultimate strength and 2.0 on permanent anchors. Therefore 800/1.6 = 500kN working load for a temporary grouted anchor would have traditionally been acceptable. Temporary anchors would have been proof loaded to 1.25 the working load whilst permanent anchors would have been proof loaded to 1.5 times the working load. EN1997-1:2004+A1 however does not distinguish between permanent or temporary anchors.

The effect of applying proof-load equivalent to that required by EN1997-1:2004/A1 must also be checked at the various stages of the SSI but was found to result in structural forces comparable to those predicted in Table 4 for the excavation under consideration.

Typically, in WSD, when calculating the required fixed length, one would use graphs by Ostermayer applying an FoS of 2.0 to 3.0 on the ultimate bond capacity. When reviewing the above analyses, it is clear than only an FoS = 1.35x1.1 = 1.5 govern the ultimate design. One should therefore be careful when sizing the fixed length and preferably increase the design ultimate load by a further factor of 1.35 when determining the required fix length (Konstantakos, 2020).

9 17M DEEP EXCAVATION, LUANDA, ANGOLA

A 17m deep excavation was constructed in Luanda, Angola, consisting of 600mm CFA soldier piles installed at 1.65m c/c with 420kN pre-stressed self-drilling anchors (T40/16) installed through the piles.

The site is underlain by a dense to very dense sand sequence of the Luanda Formation of Miocene age. The upper 20 metres of the Luanda Formation contains layers, up to 3 metres thick of very stiff clay and weakly cemented siltstone/mudstone.

9.1 *Soil Properties*

An exponential elasto-plastic stiffness (Winkler-Spring) model was used, and the characteristic soil parameters predicted from SPT-N, summarised in Table 6. A 7m high soil nail wall was constructed above the soldier pile wall (the effect of this wall is modelled by the near vertical layer shown in Figure 4).

Table 6. Characteristic Soil Parameter, Deep Excavation, Luanda, Angola

Soil Type	Depth (m)	γ_t (kN/m^3)	φ' (deg)	c' (kPa)	E (MPa)
Medium Dense Sand	0-3	18.5	33.0	7	20
Very Soft Rock Sandstone	3-6	22	45.0	200	50
Dense Sand	6-12	19.5	36.0	0	45
Very Dense Sand	12-18	20.0	40.0	0	100
Ext Dense Sand	>18m	20.0	42.0	0	200

*p_{ref} = 25kPa, 60kPa,175kPa, 300kPa and 400kPa for respective layers descending with depth

9.2 *Soil-Structure Interaction*

Non-linear (BOE) SSI analysis were undertaken with pre-stress forces as detailed in 9.1 with one model shown in Figure 4. Results of the analyses are contained in Table 7.

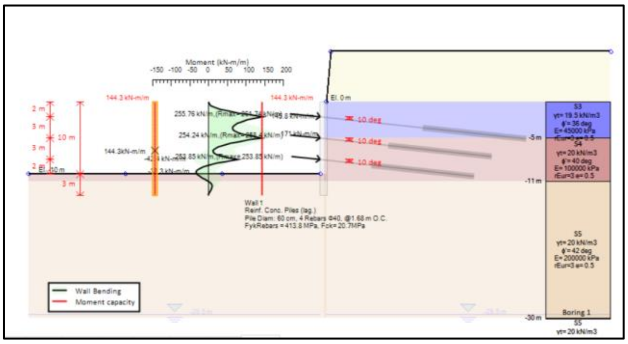

Figure 4. Elastoplastic SSI analysis, Luanda, Angola

Table 7. Soil -Structure Interaction Structural Forces

	M (kNm/m)	F (kN)	M/M_{WSD}	F/F_{WSD}	δ (mm)
Plaxis	251	463			
WSD	282 (295)	430	1.00	1.00	28
DA1-1	379 (396)	583	1.35	1.35	
DA1-2	480	469	1.70	1.09	

From the above it can be seen that the bending moments that develop in the soldier piles, using EN1997-1 DA1, are 1.35 to 1.70 times larger than those predicted from the WSD analysis. This is considerably larger than the FoS of 1.5 typically applied and would result in the requirement for 30% more reinforcement in the pile section. Overdig allowances did not have a significant effect on the bending moments.

Similar to the excavation in Gardens, Cape Town the temporary Grouted Anchors would not have complied to EN1997-1:2004+A1 and the cross section of the bar would have needed to be increased or the spacing of the grouted anchors reduced. Reducing the spacing further for this wall would have resulted in significant group effects and therefore an increased diameter would have been a better option. The Proof Load of 641kN would have resulted in an increased bending moment developing in the WSD calculation but still 1.34 times less than that the highest moment modelled using DA1-2.

10 4.0M DEEP EXCAVATION, NORTHERN SUBURBS, CAPE TOWN

The design of a 4.0m deep excavation, used as a temporary measure to retain approach fills to construct a new bridge abutment, was peer reviewed by one of the authors. Piles consisted 273mm cased Odex Piles with stressed SDA (R38N) at 3m c/c spacing. The fill consisted of dense sands and the materials below the fill medium dense to dense sand followed by clayey greywacke residual Malmesbury Group. A shallow water table was encountered 1m below the excavation level. The wall was located only in sand layers and therefore the following parameters were assumed:
γ_t = 18kN/m3, φ' = 30° and c' = 1kPa.

The analyses were undertaken using FHWA earth pressures and CALTRANS 20% negative moments. The output of one of the analyses is shown in Figure 5.

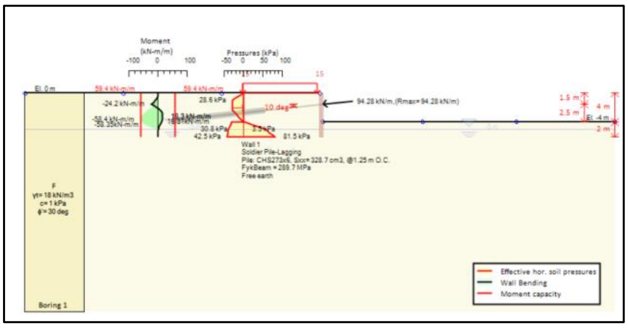

Figure 5. Limit Equilibrium analysis, Northern Suburbs, Cape Town

10.1 *Limit Equilibrium*

A summary of the results of the Limit Equilibrium analyses are provided in Table 8.

Table 8. Limit Equilibrium Analysis, Northern Suburbs, Cape Town

	M (kNm/m)	F (kN)	M/M_{WSD}	F/F_{WSD}
WSD	43	209	1.00	1.00
WSD × 1.35	58	282	1.35	1.35
DA1-1	72	282	1.67	1.35
DA1-2	67	258	1.55	1.23

Bending moments, using EN1997-1 DA1, results in an increase of 1.5 to 1.67 times higher than that calculated using WSD. This is considerably higher than the 1.5 typically used in design using WSD.

The model was re-analysed making allowance for 0.5m overdig (Table 9). In this model the bending moments remained similar when not accordingly increasing the embedment depth, but the grouted anchor reactions increased with a factor of 1.35. The embedment (D_m) was also not sufficient to prevent rotation and required an increased depth of wall of between 0.5-1.0m to ensure safety.

Table 9. Limit Equilibrium Analysis with allowance for overdig, Northern Suburbs, Cape Town

	M (kNm/m)	F (kN)	M/M_{WSD}	F/F_{WSD}	ΔD_m
WSD	44	240	1.00	1.00	N/A
DA1-1	72	369	1.63	1.53	0.5m
DA1-2	71	348	1.61	1.45	1.0m

11 BOND AND HARRIS EXAMPLE

Ou (2006) recommends that one does not use cantilever walls in soft clays due to reliability issues. It is rather suggested that one should confine cantilever walls to sands, gravels and stiff clays. The free-earth support method can be used to calculate the embedment depth required to balance moments and the required embedment depth then factored up by 1.5 to ensure shear forces are balanced (Konstantakos, 2019) as the wall rely purely on passive soil resistance. Various other approaches exist such as applying factors of safety to the bending moment or passive resistance.

Example 12.1 in Bond and Harris (2008) was compared to a conventional WSD solution (Figure 6).

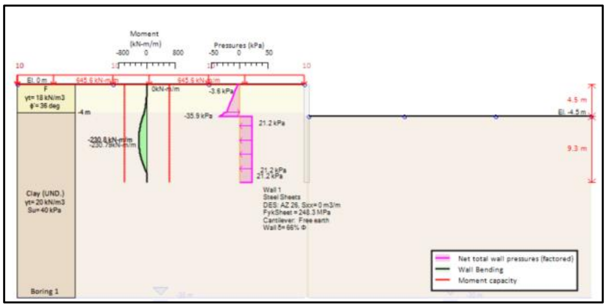

Figure 6. Limit Equilibrium analysis, Example 12.1 (Bond and Harris,2008)

Table 10. Limit Equilibrium Analysis, Bond and Harris Example 12.1 (2008)

	D_m (m)	M (kN.m/m)	F (kN)	M/M_{WSD}	F/F_{WSD}
WSD	4.0	92	100	1.00	1.00
DA1-1	9.3	136	147	1.45	1.47
DA1-2	9.3	230	250	1.45	2.50

From Table 10 it can be seen that the required embedment length for the cantilever wall in soft clay is more than double that required when using WSD methods, additionally the bending moment in the sheet pile is 2.5 times larger than that of the equivalent WSD.

It is considered that the partial load factor method would be more appropriate as the strength of the soil is the variable parameter and the uncertainty and required reliability is accounted for at the source as demonstrated in Section 3 for a probability of a worse condition governing less than 0.1%.

12 CONCLUSIONS

The design of soldier pile walls is not easy and requires significant knowledge from the designer as various structural elements and soil behaviour are at play. Various methods are used in South Africa to assess these walls (van der Merwe, Konstantakos (2019)). An attempt was made to compare the design of four walls using WSD and Limit State Design. The following themes were identified:

- Characteristic values are derived subjectively. Assuming small statistical coefficients could result in less reliable designs. Care should be exercised in choosing characteristic values combined with sufficient testing.
- Cantilever walls designed using EN1997, especially in soft clay requires significantly deeper embedment depths to ensure safety. The reduced soil strength used in DA1-2 results in moment up to 2.5times that calculated using conventional methods for the example analysed in this paper. In sand this effect is less severe.
- Grouted anchor reaction forces are typically not that different when compared to WSD for multi-anchored wall (typically 1.35 times higher) but could be up to 1.7 times higher for walls with a single row of grouted anchors.
- The requirements for the grouted anchors differ from those stipulated in LSSE code of practice and do not distinguish between temporary and permanent applications. This results in the requirement for larger tendon cross-sectional area or more strand cables.
- When sizing anchors using EN1997 DA1, the design force might result in shorter fixed length than that using WSD methods. It is therefore advised that one should compare the ultimate capacity with a design force, if governed by DA1-1, increased by a further factor of 1.35.
- For multi-anchored walls, the ultimate structural forces are typically up to 30% larger than that computed using WSD forces factored by 1.5,.
- Reducing the stiffness of the soils (halved) to correlate with lower shear parameters resulted in increased bending moments when considering the excavation in Gardens, Cape Town. The effect of reducing the stiffness needs to be further tested.
- Overdig, a formal requirement of EN1997-1, results in an increase in bending moment in the piles for most of the analyses,
- The combined effect of overdig, reduced stiffness and using EN1997-1 DA1-2 reduced shear strength parameters can lead to bending moments of up to 1.9times that using the conventional WSD approach in accordance with SAICE LSSE code of practice (1989).

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