Design charts for lateral deflection of embedded cantilever walls in unsaturated Perth sands

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ABSTRACT: Although single level basements are employed extensively in the dune sands of Perth's suburbs, there is little guidance available to assist estimation of lateral deflections of the embedded walls usually used. This paper uses the finite element method to extrapolate data measured in an instrumented full scale cantilever contiguous pile wall to a wide range of retained heights, sand properties and wall flexural rigidities. These results are presented in the form of design charts to assist rapid estimation of wall deflections. These charts are also shown to be suitable for assessments of deflections of buttressed piled cantilever walls, which offer an attractive alternative to propped or anchored walls.

1 INTRODUCTION

Almost all commercial property in the Perth area is now constructed with at least one basement level. Apart from the alluvial materials in Perth CBD and adjacent to the Swan River, ground conditions generally comprise dune/Aeolian sands with a water table usually more than 5m below ground level. Cantilevered contiguous bored pile walls are often the preferred retention solution for single level basements when construction is adjacent to neighbouring buildings, although sheet pile walls or kingpost walls are also employed.

There are relatively few well documented case histories providing measurements of lateral and vertical soil movements adjacent to cantilever walls (as well as their development with retained height and time). Fortunately, one case history relevant to Perth's dune sands reported by Li & Lehane (2010) provides such data for a staged excavation. This study involved a comprehensively instrumented cantilever wall at Shenton Park, West Perth and is used here as the base case to calibrate a finite element backanalysis. The backanalysis in then extended to a range of commonly encountered configurations to develop design charts to assist designers in the estimation of lateral displacements of embedded cantilever walls in these ground conditions.

2 CASE HISTORY

The Shenton Park cantilever retaining wall experiment is described by Li & Lehane (2010) and a photo taken when the retained heights was at 2.2m is shown on Figure 1a. The experiment involved the installation of 225mm diameter, 3.7m long CFA piles arranged at 300mm centres; this wall is approximately half the scale of a typical construction in Perth. Excavation on one side of the wall was conducted in stages with periods of up to 1 month allowed in between stages. Pile deflected shapes were monitored using inclinometer tubes placed centrally in the piles while wall bending moments were derived from strain gauges located on the tension and compression faces of reinforcement cages, as illustrated on Figure 1b. Vertical and lateral wall movements were measured using particle image velocimetry of the digital photographs taken of the targets shown in Figure 1b. The movements were observed to increase during the pause periods between excavation stages, which Li & Lehane (2010) showed could be explained by a gradual reduction in apparent c′ (which arises due to suction) in the sand.

The sand at Shenton Park is typical of Perth dune sands and is part of the Spearwood dune sand sequence. The sand has a mean effective particle size of 0.42mm, uniformity coefficient of 2.2 and average degree of saturation of about 15%. The end resistance (q_c) and shear wave velocity (V_s) measured in seismic cone penetration tests are shown on Figure 2, which also plots in-situ stresses assessed with the assistance of self-boring pressuremeter tests. It is evident that q_c increases approximately linearly with depth at a rate of 2.5 MPa per metre. The CPT q_c data combined with a calibration performed between q_c and densities measured in sand replacement tests (see Lehane et al., 2004) indicate that the sand below a depth of 1m has

Figure 1. (a) View of instrumented cantilever wall at Shenton Park (b) instrumented section of wall showing strain gauge details, inclinometer tubes and PIV targets

a relative density (D_r) of between 50% and 55%. Bolton's (1986) correlations suggest that the plane strain peak friction angle ϕ'_{ps} of Shenton Park sand at this relative density is approximately 40 to 42° at low stress levels.

The backanalysis of the Shenton Park contiguous CFA pile wall was conducted in plain strain using Plaxis 2D (Brinkgreve et al., 2016) employing a mesh 54 m wide and 15 m high. Plate elements were used to model the wall while the sand was modelled as a linear elastic Mohr Coulomb material and a Youngs modulus (E) that increased linearly with depth with a gradient = dE/dz. The E profile on the excavation side of the wall was, for simplicity, taken to be the same as on the active side and consequently a non-zero modulus existed at excavation level. This approach is often employed in practice and assumes that effects of overconsolidation on the stiffness of the sand compensate approximately for the reduced stress level due to the excavation.

While backanalyses were performed for various excavation depths, the analysis presented here considers the case for when the retained height (H) is 2.2m, with a height to wall length ratio (H/L) of 0.6; this ratio is at the upper limit of what is usually adopted for embedded walls in dry sand. The lateral earth pressure coefficient (K_0), which has a significant effect on wall displacements, was taken as the mean measured value over the retained height of 0.7 while the interface friction angle for the wall was assumed equal to the sand's constant volume friction angle of 32°.

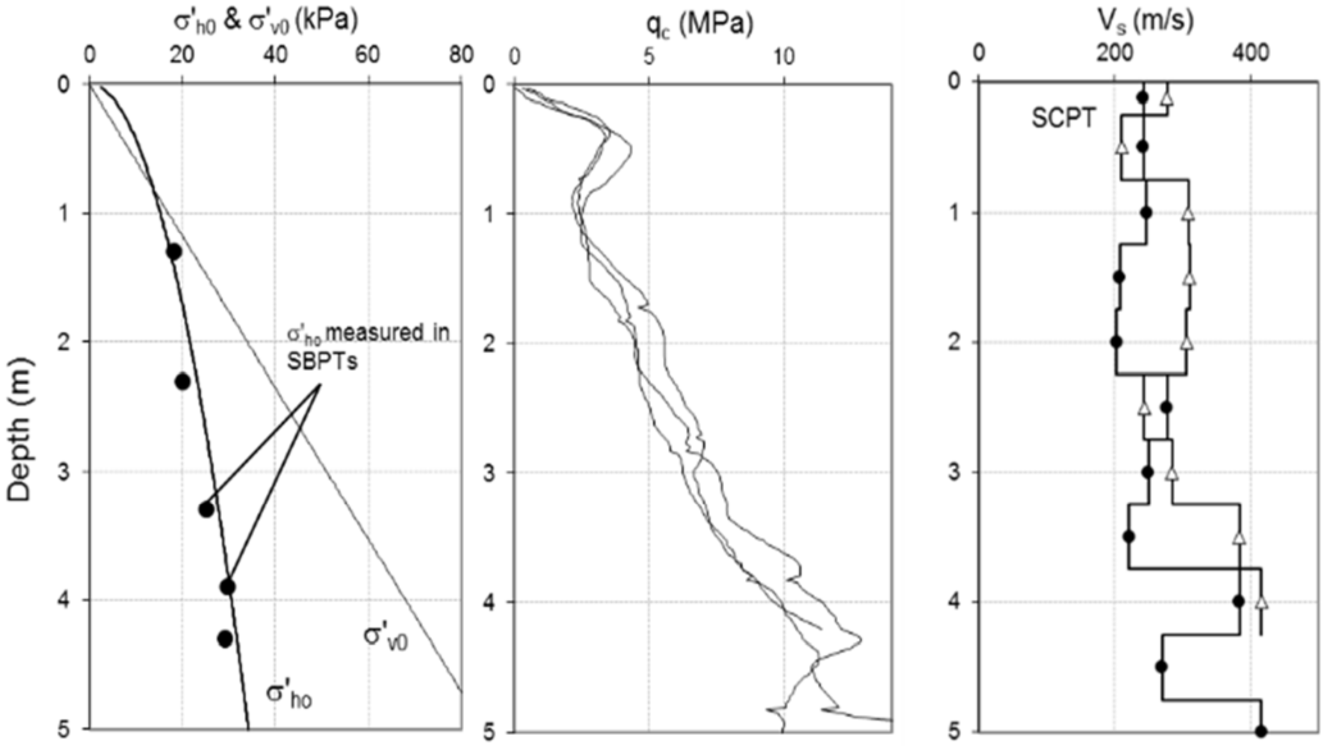


Figure 2 In-situ test data at Shenton Park

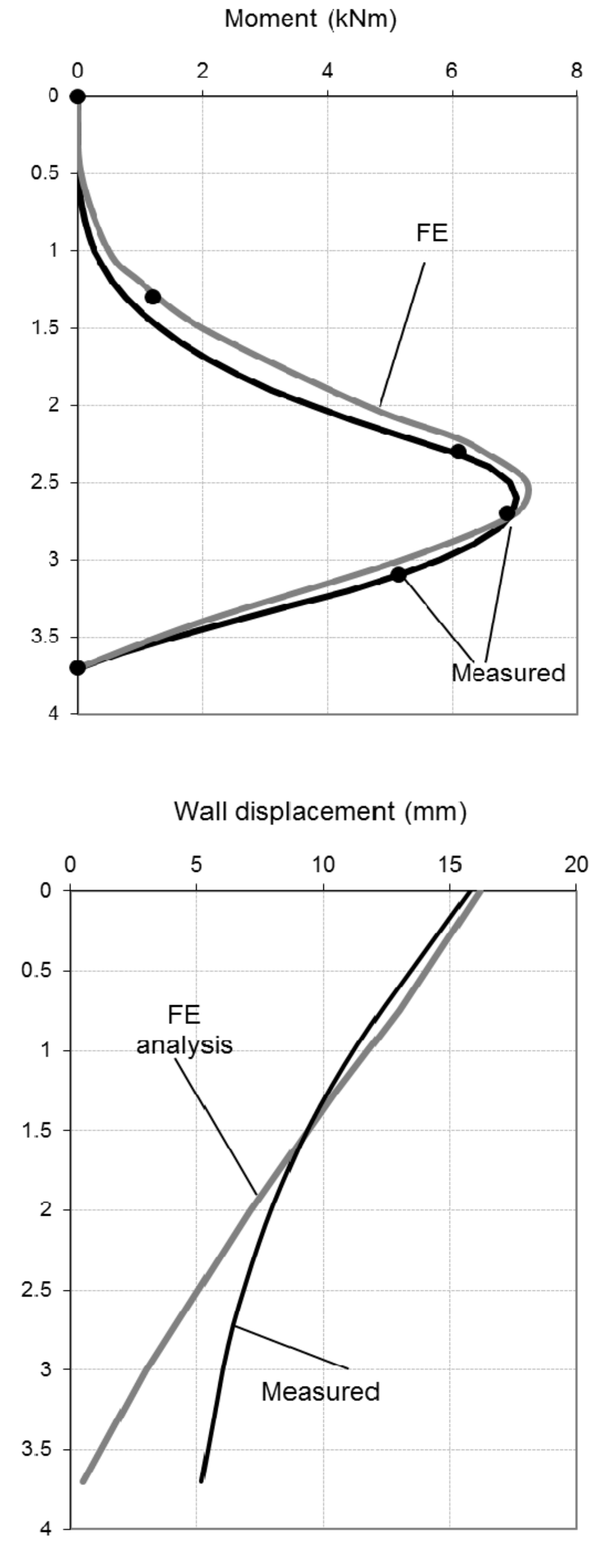


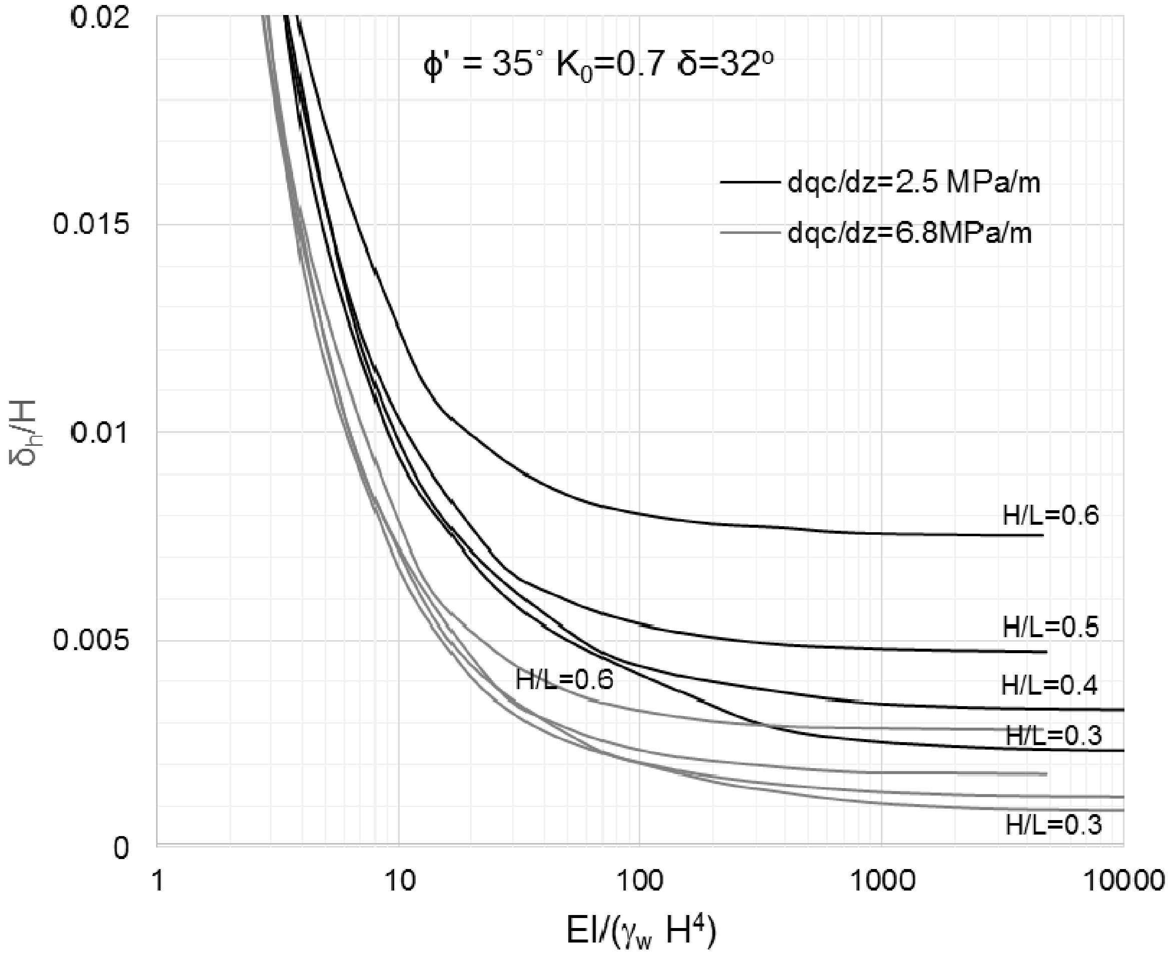
Figure 1. Comparison of measured bending moment and pile displacement profiles with predictions from the backanalysis

The wall displacements and bending moments recorded one month after completion of the excavation are plotted on Figure 3 (noting that wall displacements were three times lower immediately following completion of the excavation). Various attempts were made to match these measurements with FE predictions and it was found that a best match, as shown on Figure 3, was found for dE/dz =2750 kPa/m with c'=0 and ϕ'=43°. These parameters are closely comparable to those deduced by Li & Lehane (2010) using the SAFE FE program. The same Young's modulus gradient led to reasonable predictions for different retained heights for H <2.5m. This gradient corresponds to a ratio to the CPT q_c value (E/q_c) of 1.1, which interestingly is about 5 times less than the ratio adopted by practitioners in Perth. Relatively low E/q_c ratios were also deduced by Lehane et al. (2008) in backanalyses of footing tests on Perth's dune sands.

3 PARAMETRIC STUDY

Given the ability of the FE analyses to replicate the observations in the Shenton Park experiments, a series of analyses were performed to identify the relative influence of the wall's flexural rigidity (EI), sand friction angle and CPT q_c gradient. Preliminary analyses for typical retained heights (H) and wall lengths (L) showed that the normalised lateral deflection at the top of the wall (δ_h/H) could be described as a function of (H/L). The parametric study examined a H/L range of 0.3 to 0.6 using wall lengths (L) =3.7m and 10m, in total, involved approximately 300 FE analyses. The dE/dz gradient was assumed to be 1.1 times the q_c gradient ($dq/dz = q_c'$) in line with the Shenton Park backanalyses. CPT q_c gradients (q_c') of 2.5 MPa/m (as measured at Shenton Park) and of 6.8 MPa/m (representative of a dense sand) were examined while friction angles ϕ' of 35° and 43° were investigated for both q_c' values. The wall flexural rigidities were varied by three orders of magnitudes and attention was focussed on cases for which lateral deflections were less than 2% of the retained height (0.02H), which is a typical upper limit in design.

The analyses showed that, in keeping with the Rowe's flexibility number and also the well-known equation for the deflection at the end of a cantilever, the computed normalised lateral displacements at the top of the wall (δ_h/H) varied with the retained height (H) raised to the power of 3 and were inversely proportional to the wall's flexural rigidity (EI). These trends are represented on Figure 4 by plotting the normalised lateral displacement (δ_h/H) with the system stiffness coefficient proposed by Clough et al. (1989), $EI/(\gamma_w H^4)$, where EI is the flexural rigidity per metre length of wall; the unit weight of water (γ_w = 10 kN/m^3) to non-dimensionalise the expression. The following trends are evident on inspection of Figure 4:



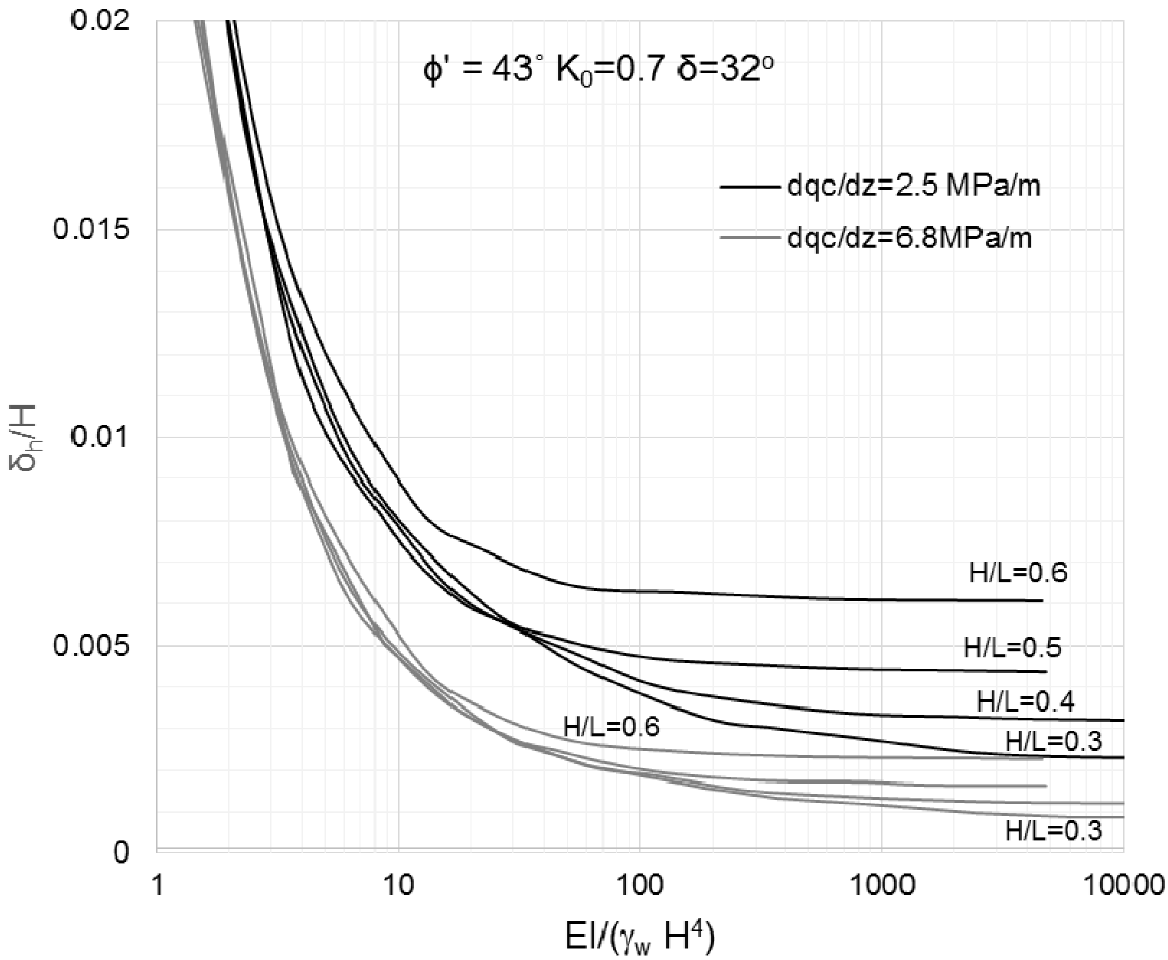


Figure 4 Design charts for estimation of maximum lateral deflection δ_h

- As might be anticipated, the normalised lateral displacement (δ_h/H) is strongly dependent on H/L, with deflections up to 3 times higher at H/L=0.6 compared to those at H/L=0.3.
- The normalised maximum wall deflection varies directly with the system stiffness coefficient for $EI/(\gamma_w H^4) < 20$.
- The q_c gradient (q'_c) has a significant effect δ_h/H for both rigid and flexible walls. An increase in q'_c from 2.5 to 6.8 MPa reduces wall deflections by between 1.5 and 3 times.
- Increasing the wall's EI value has little effect in reducing the lateral deflection for stiff walls with $EI/\gamma_w H^4 > 100$; this characteristic has important design implications, given that increasing wall stiffness is usually assumed to lead to a reduction in wall movements.
- Increasing the friction angle from 35° to 43° only reduces wall deflections by about 25% for H/L=0.6 and has little effect on deflections for lower H/L values.

It is important to note that a reduction in the K_0 value from the value of 0.7 employed in Figure 4 to a more commonly adopted value of $K_{0,nc}=1-\sin\phi'$ leads to a considerable reduction in the calculated δ_h/H ratio. For example, use of $K_{0,nc}$ in Shenton Park experiment would have under-predicted the measured δ_h value by 40%.

4 EXAMPLE APPLICATION

The design chart on Figure 4 was recently used by Belpile Pty Ltd to assess the influence of buttressing a contiguous piled wall at a development in Perth suburbs. There was a requirement at this development to support a 5m high cantilevered section over part of the site perimeter. The retention works around the remainder of the development were constructed using 10m long, 600mm diameter CFA piles and therefore Belpile opted to use the same pile type but with the plan configuration shown on Figure 5 for the 5m cantilever. The system stiffness was calculated using the EI value (per metre length) of the complete section indicated on Figure 5, given that a 800mm high heavily reinforced capping beam ensured that the wall would behave as one unit. The system stiffness of this buttressed arrangement is 600 compared to value more than 10 times less of 47 for the wall with no buttress piles. The q_c' value at the site was about 1.5 MPa/m (i.e. only 60% of the value at Shenton Park of 2.5 MPa/m). Figure 4a was used to estimate the lateral deflections assuming ϕ'=35° with H/L=0.5. This gives δ_h/H values of 4.8×10^{-3} and 6×10^{-3} for the buttressed and un-buttressed wall respectively for q_c'=2.5 MPa/m. Based on extrapolation from the q_c' values considered in Figure 4 to the actual q_c' value of 1.5 MPa/m, best estimate long term δ_h values of 35mm and 45mm can be determined for the buttressed and un-buttressed cases, respectively.

A full 3D FE analysis using the mesh shown on Figure 6 was also performed to examine the effects of buttressing. This analysis, which used the same set of parameters employed for derivation of Figure 4a but with q_c'=1.5 MPa/m, gave lateral displacements of the capping beam of 26mm and 38mm with and without buttressing respectively. These displacements are in good agreement with those determined from Figure 4a and show that the primary influence of buttressing is to increase the overall wall rigidity. Based on the trends shown on Figure 4, this rigidity has a significant influence on lateral movements for flexible walls, but less so for more rigid walls. Buttressing combined with the capping beam also leads to a considerable reduction in maximum pile bending moments (and hence wall steel requirements) due to the capacity for the large reverse moments to develop at the pile head. Further 3D analyses showed almost an identical response when the buttress piles are located on the excavation side of the wall.

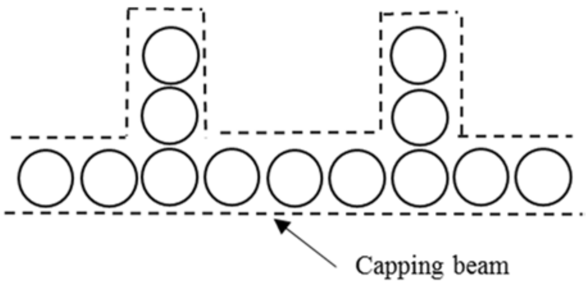


Figure 5 Geometry of buttress wall (600mm CFA piles at 650mm centres)

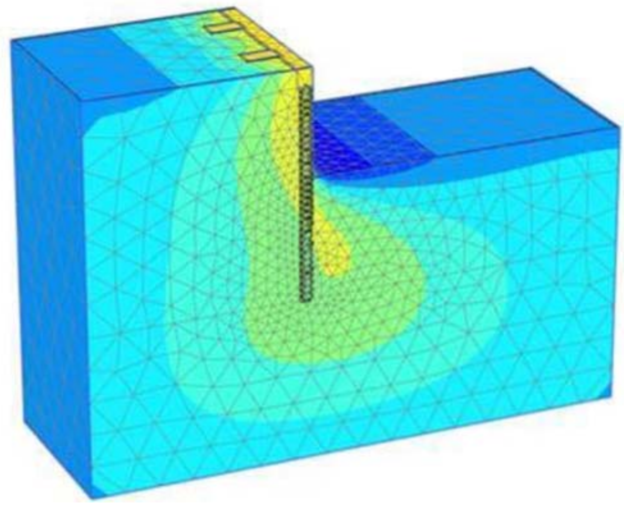


Figure 6 FE mesh showing contours of lateral displacement)

The buttressed cantilever wall option can therefore provide a suitable alternative to a propped or anchored wall, noting that the buttress piles, when located on the excavation side, can be removed following casting of the permanent slabs.

5 CONCLUSIONS

A series of a parametric analyses was performed based on backanalysis of measurements obtained from an instrumented cantilevered contiguous pile wall in unsaturated Perth dune sand. The analyses show that maximum wall movements vary directly with the system stiffness for very flexible walls with $EI/\gamma_w H^4 < 20$, while movements vary with H/L and are independent of wall rigidity for $EI/\gamma_w H^4 > 100$. Lateral deflections generally depend on the system stiffness ($EI/\gamma_w H^4$), the CPT q_c gradient (q_c') and the assumed K_0 coefficient.

Design charts are presented that can be used to obtain a rapid estimate of the likely lateral deflections for various options being considered. The charts also provide designers with a means to assess the impact of using a wall of different flexural rigidity or H/L value.

The analyses show that the charts are applicable for buttressed cantilever walls, which provide an interesting alternative to propped or anchored walls by reducing the lateral deflection and maximum bending moments.

6 REFERENCES

Bolton, M. D. (1986). The strength and dilatancy of sands. *Geotechnique*, 36, No. 1, 65–78.

Brinkgreve, R. B. J., Kumarswamy, S., & Swolfs, W. M. (Eds.). (2016). *PLAXIS 2D*. Netherlands.

Clough, G. W., Smith, E. M., and Sweeney, B. P. (1989). Movement control of excavation support systems by iterative design. In *Foundation engineering, current principles and practices*, K. H. Kulhawy, ed., Vol.2, 869–884

Lehane, B. M., Ismail, M. & Fahey, M. (2004). Seasonal dependence of in-situ test parameters in sand above the water table. *Geotechnique*, 54(3), 215–218.

Lehane B.M., Doherty J.P. and Schneider J.A. (2008). Settlement prediction for footings on sand. Keynote Lecture, *Proc. 4th International Symposium on deformation characteristics of Geomaterials*, Atlanta, 1, 133-152, IOS press, The Netherlands.

Li, A. and Lehane B.M. (2010). Embedded cantilever retaining walls in sand. *Geotechnique*, 60 (11), 813-823.