EULERIAN ANALYSIS OF JACK-UP SPUD CAN FOUNDATION PERFORMANCE

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ABSTRACT
For site specific assessment of jack-up rigs, existing standards and practices require establishment of certain spud can foundation performance parameters. Among the required parameters are the maximum moment capacity and the maximum horizontal capacity. These parameters are established by use of site-specific soils data, spud can geometry and preload values. It is reasonable to expect that the proportionality of the horizontal and moment capacities to the vertical capacity should improve with increased penetration. Prior to the issue of ISO 19905-1 there was no provision for this. However, work sponsored by the International Association of Drilling Contractors' Jack-Up Committee (IADC) provided technical basis for inclusion of penetration-dependent foundation performance parameters to rectify this situation, particularly in the case of clay soils. The result was substantial increase in foundation parameter values for deep penetrations in clay. Subsequent hindcast studies using the new penetration-dependent parameters showed very good agreement with actual field performance of instrumented rigs in severe storms, and the IADC-recommended parameter formulations were subsequently adopted in ISO 19905-1 and in SNAME Bulletin 5-5.

The IADC recommendations were based, in part, on results of finite element analysis of spudcan-soil interaction. Those analyses modeled part of the spud can installation as well as the subsequent moment and horizontal loading. Both parts were done with both small strain and finite strain assumptions, but the analyses were done over a decade ago using mesh densities and analysis techniques that could now be considered outdated. More recent finite element work by Zhang et al. (2014) used better modeling techniques to substantiate more conservative recommendations, particularly in the case of clay sites with linear strength profiles. These analyses used large deformation assumptions for analysis of the installation, followed by small strain analysis of the horizontal and vertical performance. These analysis results are in good agreement with small scale (centrifuge) laboratory tests.

ISO 19905-1, Edition 3, currently in draft, would adopt formulations based on Zhang et al. (2014) for sites with strictly linear strength profiles and would restrict the present ISO 19905-1 formulations to sites with uniform strength. This raises the question as to what should be recommended for sites with other commonly encountered types of strength profiles. Templeton (2019) reported on finite element analyses of spud can installation and subsequent loading to capacity all done with full Eulerian modeling for both the spud can installation and subsequent moment and horizontal loading. Those results provided reconciliation of some of the differences in the prior work and proposed new ISO 19905-1 recommendations for commonly encountered strength profiles. The paper proposed here will provide extension and further validation of the results of Templeton (2019) and updated recommendations from that work.

KEYWORDS: Jack-up, spudcan, foundation, fixity, clay

INTRODUCTION
The objective of the subject study is to provide new recommendations for important parameters used in the calculation of spudcan foundation performance for deep penetrations in commonly encountered clay soil conditions. Without new technical work in this area the next edition of ISO 19905-1 would provide, for clay soils, separate fixity recommendations only for the extreme cases of (strictly) normally consolidated or uniform soil strength conditions. The clay strength profiles most commonly encountered offshore are intermediate to these two extreme cases. Recommendations specific to the intermediate cases should enable more accurate site assessments.

For site specific assessment of jack-up rigs, ISO 19905-1 (2012) requires establishment of certain spud can fixity parameters. Among the required parameters are the maximum moment capacity and the maximum horizontal capacity. These parameters are established by use of site-specific soils data, spud can geometry and preload values. Logically the parameter formulations should provide for improvement in horizontal and moment capacity with increased penetration. Prior to the issue of ISO 19905-1 there was no such provision. However, work sponsored by the International Association of Drilling Contractors' Jack-Up Committee (IADC) provided technical basis for inclusion of penetration-dependent foundation performance parameters to rectify this situation, particularly in the case of clay soils. The result was substantial increase in foundation parameter values for deep penetrations in clay. Subsequent hindcast studies by Templeton, Lewis and Brekke (2009) and Templeton and Lewis (2011) using the new deep penetration parameters showed very good agreement with actual field performance of instrumented rigs, and the IADC-recommended parameter formulations were subsequently adopted in ISO 19905-1 (see, Wong 2012) and in corrigenda to SNAME Bulletin 5-5 (2008).

The IADC recommendations had been based, in part, of results of finite element analysis of spudcan-soil interaction. Those analyses modeled part of the spud can installation as well as the subsequent moment and horizontal loading. Both parts were done with both small strain and finite strain assumptions. More recent finite element work by the Zhang, Bienen and Cassidy (2013, 2014) used newer modeling techniques to substantiate more conservative recommendations specific to normally consolidated clay.

The current draft of ISO 19905-1 prescribes the original IADC-based recommendations only for the case of uniform soil strength, and it prescribes the recommendations based on Zhang, Bienen and Cassidy (2013, 2014) for the case of a normally consolidated strength profile (increasing strictly linearly from negligible strength at the mudline). This situation leaves a gap in recommendations for more commonly encountered strength profiles intermediate to these two extremes.

The clay conditions assumed in the Zhang, Bienen and Cassidy (2013, 2014) work are not representative of those commonly encountered offshore. The strength profile assumed involved negligible strengths near the mudline while offshore clays have substantial near-mudline strength. This is important because, in the Zhang, Bienen and Cassidy (2013, 2014) analyses weak soil from near the mudline is stuck to the spud can and carried down to the penetration zone. This effect is apparently realistic, and the weakness of the soil carried downward limits both the horizontal and moment capacities.

The recommendations presently in ISO 19905 for deep penetrations in clay were based on work done for the IADC jack-up Committee beginning in 2002. That technical work was done with conventional finite element analysis methods of the time and is somewhat outdated by today's standards – although large strain methods were included to simulate part of the installation process as well as the subsequent in-place performance. The Zhang, Bienen and Cassidy (2013, 2014) work used newer methods which allowed analysis of the entire spud can penetration process to produce soil stress and stain field data for use in a separate analysis of the in-place performance.

Templeton (2019) reported on finite element analyses of spud can installation and subsequent loading to capacity, all done with full Eulerian modeling, for both the spud can installation and subsequent moment and horizontal loading, employing large deformation theory throughout. Those results provided reconciliation of some of the differences in the prior work and proposed new ISO 19905-1 recommendations for commonly encountered strength profiles. The work in the present paper provides extension and further validation of the results of Templeton (2019) and updated recommendations from that work

WORK PERFORMED

Analyses with the Abaqus program were used to provide numerical solutions for the deep penetration and subsequent in-place loading of spud cans in clay soils with strength profiles involving various strengths at depth. Abaqus is a large, nonlinear, general purpose, commercial finite element analysis program with substantial soil mechanics capabilities. It has a long history of successful use in offshore geotechnical analysis. The analyses reported here were performed using the Combined Eulerian-Lagrangian capabilities available in Abaqus.

Traditionally, finite element analysis has been accomplished using what is now called a Lagrangian approach. In this approach, nodes and element boundaries are tied to material, and a fixed body of material is contained in each element. The mesh deforms with the material, and the amount of material deformation that can be achieved is ultimately limited by errors resulting from element deformation. With an Eulerian approach the nodes and element boundaries are fixed in space and material flows across element boundaries as material deformations occur without any element deformation. In this way arbitrarily large material deformations can be achieved without element performance problems. In Abaqus Combined Eulerian-Lagrangian (CEL) analysis, Lagrangian meshing for bodies with limited deformation can be combined with Eulerian meshing for bodies with large deformations, and surfaces of the two kinds of bodies can interact when in contact. In addition, large deformation theory (including large strain formulations for both stress and strain as well as proper treatment of large rotations and the effects of essential geometry change) is used throughout all steps of the CEL analyses in both Lagrangian and Eulerian bodies.

In the analyses reported here, the spud an was modeled as a Lagrangian body, and the soil was modeled as an Eulerian body. The soil and spud can were connected with a contact surface. The general arrangement for such modeling is shown in Figure 1 (after Vazquez et al., 2017). Note that this figure shows a half-space model taking advantage of the symmetry of the spud can and the fact that the motion studied is confined to a plane (the x-z plane in Figure 1). This symmetry restriction (typical of the models used in the present study) is proper for vertical (z-direction) and horizontal (x-direction) motions as well as for rocking rotations (in the x-z plane). The following are additional details of the analyses reported here:

- The spud can geometry was based on a Letourneau 116-C with the forged tip removed and the corners rounded. These simplifications improved the contact surface performance.
- A no-slip specification was used on the contact surface.
- The spud can was made rigid.
- The (clay) soil was modeled as an elastic-plastic solid with yield shear strength generally variable as a function of depth. Typically, the soil shear strength was made constant over a limited depth near the mudline and given a normally consolidated rate of increase with depth below that and continuing well below the embedment depth.
- The soil was given submerged unit weight generally consistent with interpreted submerged unit weights corresponding to the interpreted soil strength profiles reported here but preferably sufficient to cause nearly complete backfill above the spud can.

Care was taken to ensure that the modeling was done in such a manner that near-mudline soil could be adhered to the surface of the spud can and drug downward below the can during penetration. Figure 2 displays this behavior. Figure 2a shows a model plot of soil and spud can prior to penetration with the soil initially within approximately ¼ spud can diameter colored red and the initially deeper soil colored blue. Figure 2b shows a plot of the same model after penetration with the same soil colorations. These plots show clearly that a significant body of the soil originally near the mudline has been transported downward beneath the penetrated spudcan. The moment or horizontal resistance should logically be affected by the strength of the soil transported downward.

Figure 3 shows contour plots of plastic strain from CEL analysis of a penetrating spudcan. Figure 3a shows the condition after a slight penetration. Figure 3b shows the condition after substantial penetration in the same analysis. For each of the production analyses of the study, the spud can was first penetrated to a particular penetration, typically in the range of 1.0 to 1.5 spud can diameters. Next, the vertical load was reduced to one half of the amount needed for that penetration, and finally either a horizontal or rotation loading was applied while the vertical load was maintained constant at the reduced level.

The work of the present study included not only the performance of CEL analyses of penetration and subsequent in-place loading but also identification of a number of soil strength profiles commonly encountered in jack-up rig installations. The identified strength profiles were generally intermediate to normally consolidated and uniform strength conditions.

RESULTS

Figure 4. shows a number of soil strength profiles for clay. Sites 1 through 3 are from SAGE files. Sites 4 through 12 were reported by Menzies and Roper (2012). The additional cases are idealized. Figure 5 shows the same strength profiles with the strengths normalized to their values at a penetration depth of 50 ft. The leftmost blue line is a normally consolidated profile with zero strength at the mudline, similar to that used in the Zhang, Bienen and Cassidy (2013, 2014). The rightmost line is a uniform strength case. The remainder of the lines are realistic cases, most of them are for actual sites from a previous IADC study. The model developed in the present study is capable of modeling any of these profiles.

Figure 6 shows results for the moment capacity factor as a function of the relative strength of the near-mudline soil. The specific measure of relative strength used is the ratio of the soil shear strength near the mudline (specifically the minimum strength within a depth of ¼ spud can diameter down from the mudline) to the strength similarly near the spud can embedment. On this figure, the point for the uniform strength case was selected with complete backfill. Nearly complete backfill was observed in most of the analysis case results. Figure 7 shows similar results for the horizontal capacity factor. Also shown on both figures are capacity factor values from the current draft revision of ISO 19905-1 for normally consolidated and uniform strength clay. The current draft revision prescriptions for uniform strength clay are the same as the presently issued standard prescriptions for all clay soils. These are described by Wong et al. (2012), and are based on Templeton, Brekke and Lewis (2005), and Templeton (2006, 2007, 2009). The current draft revision prescriptions for normally consolidated clay are based on Zhang et al. (2014). Also shown on both figures are straight lines representing interpolations from the normally consolidated to uniform strength values. In the normally consolidated case, factors are provided in ISO 19905-1 for other values of sensitivity (although the dependence is weak). In the uniform strength case, sensitivity should be considered in the establishment of the factors per ISO19905-1 formulae. In the present work analysis cases were performed for various near-mudline relative strength ratios at sensitivity=1 and for various soil sensitivities at near-mudline relative strength ratios of 0 and 1.

In the horizontal case (Figure 7), the value of Ch at near-mudline relative strength of 0 based on ISO 19905-1 draft Edition 3 includes adjustment for the influence of effect of spud can area aspect ratio (specifically the ratio of the horizontal projected area, As, to the vertically projected area, A) as provided in the draft standard. The area aspect ratio for the CEL FEA analyses and for the uniform strength point in Figure 7 was 0.42.

The results in Figures 6 and 7 indicate that both the moment and horizontal capacity factors increase with increasing strength of the near-mudline soil – a very useful and also intuitive result. On the low end (i.e. at low near-mudline strengths) the trend of the results tend to confirm the present draft ISO19905-1 revision values. On the high end, the data appear to trend toward the uniform strength case value. In the case of both the moment and the horizontal, the CEL FEA results could be reasonably well characterized by the l interpolations shown.

The interpolation for the moment factor is given by:

$$Cm = Cm\text{-}nc + (Su\text{-}nml / Su\text{-}ned)(Cm\text{-}u - Cm\text{-}nc) \qquad \text{(equation 1)}$$

where:

- Cm is the moment capacity factor
- Cm-nc is the moment capacity factor for the normally consolidated case
- Cm-u is the moment capacity factor for the uniform strength case
- Su-ned is the (undrained shear) strength near the embedment depth
- Su-nml is the minimum (undrained shear) strength within ¼ (spud can) diameter below the mudline

The interpolation for the horizontal factor is given by:

$$Ch = Ch\text{-}nc + (Su\text{-}nml / Su\text{-}ned)(Ch\text{-}u - Ch\text{-}nc)^{m} \quad \text{(equation 2)}$$

where:

Ch is the horizontal capacity factor

Ch-nc is the horizontal capacity factor for the normally consolidated case (adjusted for area ratio as necessary)

Ch-u is the horizontal capacity factor for the uniform strength case (adjusted for area ratio as necessary)

Su-ned is the (undrained shear) strength near the embedment depth

Su-nml is the minimum (undrained shear) strength within ¼ (spud can) diameter below the mudline

m is the exponent for power law interpolation.

Least squares fit to the CEL results plotted in that Figure 7 produced the exponent value, m = 0.36

CHECKS AND COMPARISONS

For the analyses of this study, a number of checks and comparisons were made in order to assure consistency with good practice and with other published work. These included:

- Checks of sensitivity of results to mesh fineness.
- Checks of sensitivity of results to location of boundaries.
- Comparisons of net vertical bearing capacity factor results to those of Skempton (1951).
- Comparisons of moment capacity results for normally consolidated clay to those of Zhang et al. (2014)
- Comparisons of moment capacity results for uniform clay to those of Templeton et al. (2005)
- Comparisons of horizontal capacity results for normally consolidated clay to those of Zhang et al. (2014)
- Comparisons of horizontal capacity results for uniform clay to those of Templeton (2009)
- Comparisons of results for various embedment depths and soil sensitivities to those of Zhang et al. (2014)

A parametric study was made of the effect of mesh fineness by way of an automatic mesh refinement technique. Starting with a base mesh similar to that shown in Figure 3, three levels of refinement were investigated. The density (number of elements per unit volume) of Eulerian soil elements near the spud can was increased, sequentially, by factors of 8 (Level 1 refinement), 64 (Level 2 refinement) and 512 (Level 3 refinement). The results at refinement Levels 2 and 3 were virtually the same. With results at these levels taken as a reference, the results at refinement Level 1 were generally off by amounts on the order of 1%, and the results for the base (unrefined) model were generally off by amounts on the order of 10%. These results were obtained for both the vertical and moment reactions. The results for the horizontal reactions showed even less effect of mesh fineness. Horizontal reactions for even the base model differed from the results at Level 2 refinement by less than 1%. Since the Level 2 and Level 3 refinement analyses required extremely long computational times (up to many days in the case of Level 3), production analyses for this study were performed at the base level and at refinement Level 1.

The adequacy of the bottom boundary location was checked by inspecting results for spud can penetrations approaching the mesh bottom. Results indicated that spudcan embedments should at least be limited to approximately 0.5 spudcan diameter shallower than the bottom of the mesh. In the main analyses of the study, they were so limited. Except for some of the embedment studies, embedments were approximately 0.9 diameter shallower. Previous studies had indicated that a model outside diameter equal to 5 spudcan diameters should be adequate. The model outside diameter used in this study was equal to 6 spudcan diameters.

Table 1 shows comparison of net vertical bearing capacity results in this study to results calculated according to the recommendations of Skempton (1951). Differences from the comparison values are all less than 3%. The comparisons here are listed for a soil sensitivity of 1.0. The Skempton (1951) work was done with remolded clay, and the analyses for this comparison were done with the elastic-perfectly-plastic material behavior model. The "Comparison Values" were calculated from a hyperbolic tangent curve fit to the published Skempton (1951) graphical curve. The "Values This Study" were derived from reaction force results in displacement-controlled analyses. This level of agreement was important to the entire study. It meant that for cases with sensitivity = 1.0, the Cm and Ch values could be determined by dividing analysis (moment or horizontal) capacity results either by the Skempton-based net vertical capacity or by net vertical capacity derived from the analysis results. In various cases, either one basis or the other was used, depending on convenience or expected accuracy.

Similar comparisons for cases with sensitivities greater than 1.0 indicated that good agreement between Skempton-based net vertical capacities and analysis result-based values indicated that the best agreement could be obtained by calculating the Skempton-based capacities using the average of undegraded and remolded values of shear strength. This approach was used for interpretation of results in the present study for cases with sensitivities greater than 1.0.

Parameter	Shear Strength Profile	Sensitivity	D/B	Comparison	Comparison Value	Value This study
Nc	uniform	1	0.5	Skempton (1951)	6.99	6.80
Nc	uniform	1	0.75	Skempton (1951)	7.36	7.26
Nc	uniform	1	1	Skempton (1951)	7.69	7.60
Nc	uniform	1	1.25	Skempton (1951)	7.97	7.84
Nc	uniform	1	1.5	Skempton (1951)	8.20	8.04
Nc	uniform	1	1.75	Skempton (1951)	8.38	8.21
Nc	uniform	1	2	Skempton (1951)	8.53	8.35
Nc	uniform	1	2.25	Skempton (1951)	8.64	8.46
Nc	uniform	1	2.5	Skempton (1951)	8.73	8.55
Nc	uniform	1	2.75	Skempton (1951)	8.80	8.63
Nc	uniform	1	3	Skempton (1951)	8.85	8.70

Table 1. Comparison of results from this study for net vertical bearing capacity to values from Skempton (1951)l

Table 2 shows comparison of moment capacity results in this study to results of Zhang et al. (2014) and of Templeton et al. (2005). Differences from the comparison values are less than 5%.

Parameter	Mudline Strength Ratio	Sensitivity	D/B	Comparison	Comparison Value	Value This study
Cm	0	1	1.5	Zhang et al. (2014)	0.102	0.104
Cm	0	2.2	1.3	Zhang et al. (2014)	0.096	0.100
Cm	0	3	1.5	Zhang et al. (2014)	0.096	0.100
Cm	0	4	1.3	Zhang et al. (2014)	0.093	0.092
Cm	1	1	> 1.2	Templeton et al. (2005)	0.175	0.176

Table 2. Comparison of results from this study for moment capacity to results of Zhang et al. (2014) and of Templeton et al. (2005).

Table 3 shows comparison of horizontal capacity results in this study to results of Zhang et al. (2014) and of Templeton (2009). Differences from the comparison values are 10% or less.

Parameter	Mudline Strength Ratio	Sensitivity	D/B	Comparison	Base Comparison Value	Area Adjusted Comparison Value*	Value This Study
Ch	0	1	0.88	Zhang et al. (2014)	0.216	0.227	0.229
Ch	0	1	1.14	Zhang et al. (2014)	0.242	0.254	0.228
Ch	0	2.2	1.32	Zhang et al. (2014)	0.234	0.245	0.242
Ch	0	3	1.38	Zhang et al. (2014)	0.225	0.236	0.248
Ch	0	4	1.47	Zhang et al. (2014)	0.215	0.226	0.236
Ch	0	4	1.81	Zhang et al. (2014)	0.229	0.240	0.218
Ch	1	1	1	Templeton (2009)		0.53	0.529

* Note: Adjusted values include area ratio, As/A = 0.42

Table 3. Comparison of results from this study for horizontal capacity to results of Zhang et al. (2014) and of Templeton (2009).

For cases requiring soil sensitivities greater than 1.0, requisite sensitivities were included by use of a plastic strain softening formulation consistent with that proposed by Einav and Randolph (and as cited by Zhang et al., 2014). With this approach, soil strength reduction occurs continuously throughout the analyses. The formulation requires a parameter, ξ_{95}, described by Einav and Randolph as "the cumulative shear strain required to cause 95% reduction (from peak to remoulded)". For this study a value of ξ_{95} = 30 (i.e., 3000%) was used for consistency with the value cited by Zhang (2014). This strain softening effect of sensitivity was included during both the penetration and subsequent rotation or horizontal parts of the analyses.

CONCLUSIONS AND RECOMMENDATIONS

From the work of this study it is observed that the CEL FEA analysis technique is quite capable of modeling the penetration and subsequent in-place performance of a spud can in clay with various strength profiles. The analyses performed in this study clearly include the phenomenon of relatively weak near-mudline soil being transported downward beneath the spud can, and the results show that the subsequent moment and horizontal capacities are directly related to the strength of the soil brought down.

The results of this study are in good agreement with the present provisions of the draft ISO 19905-1 Edition 3 for both normally consolidated and uniform clay. The specific comparisons made tend to confirm results in the underlying work for those provisions, including those of Zhang et al. (2013, 2014) and those of Templeton et al. (2005) and Templeton (2009).

We propose that that, for cases of clay sites with strength profiles intermediate to normally consolidated and uniform, the moment capacity and horizontal capacity factors, Cm and Ch should be interpolated between factors for the two extreme cases, as via equations 1 and 2 of this paper

Although the analysis results presented here seem to suggest that in some cases Cm and Ch might achieve values greater than presently prescribed by ISO19905-1, proper substantiation of that possibility would require further work.

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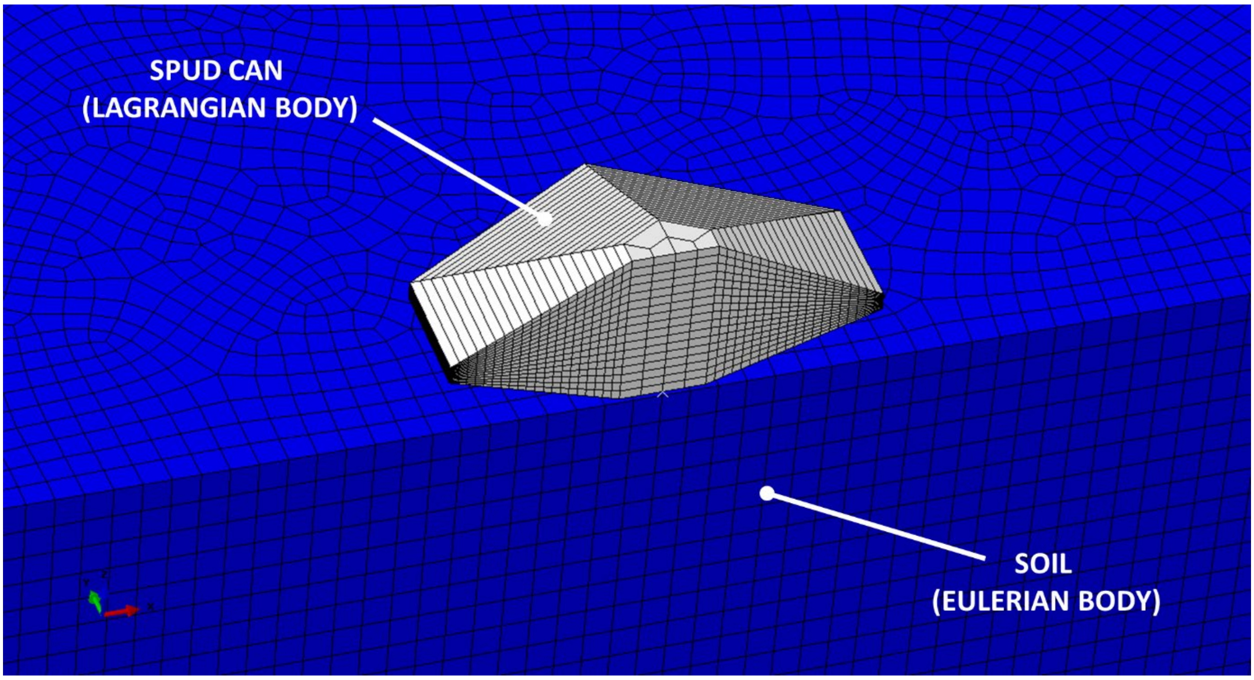

Figure 1. Lagrangian Spud Can and Eulerian Soil

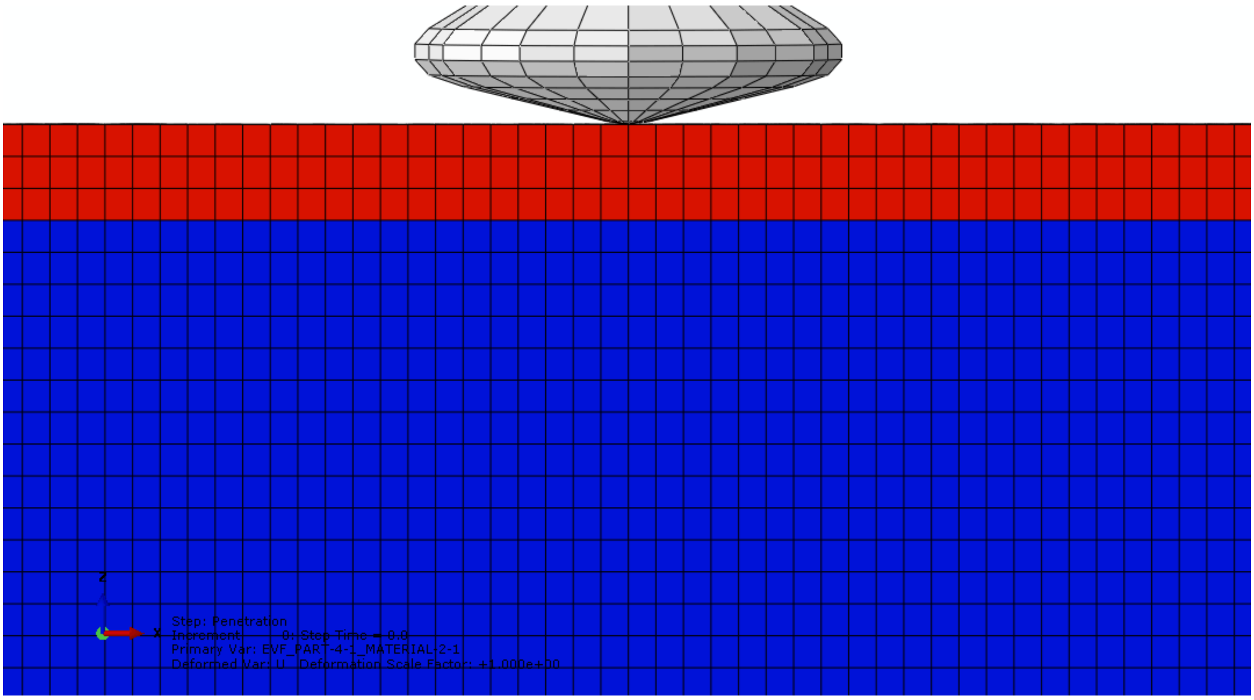

Figure 2a. Soil configuration prior to penetration

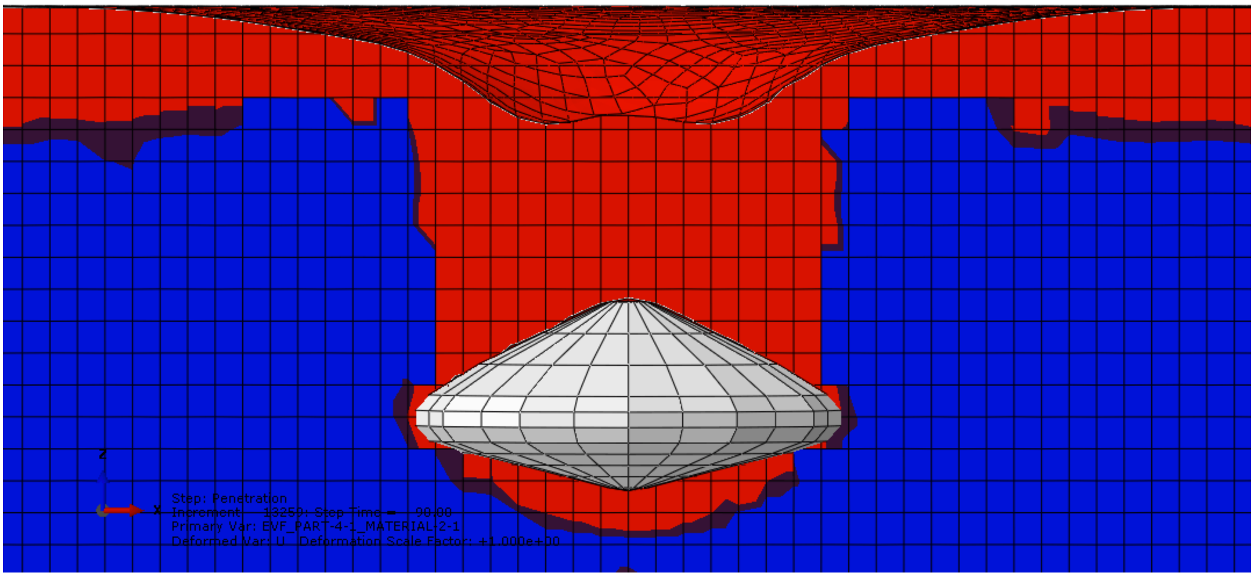

Figure 2b. Soil configuration with penetration

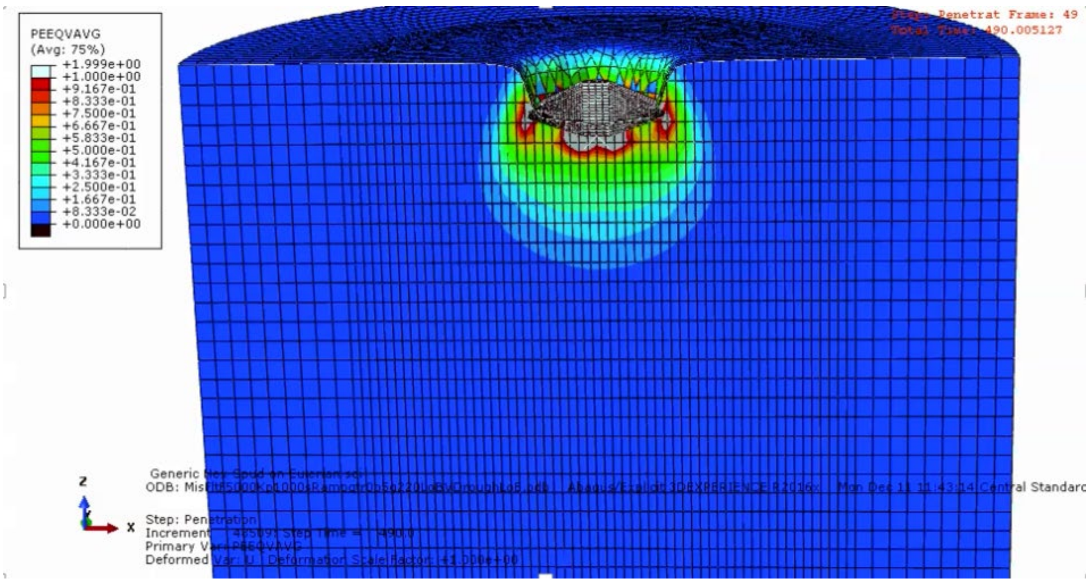

Figure 3a. Plastic strain contours with moderate penetration

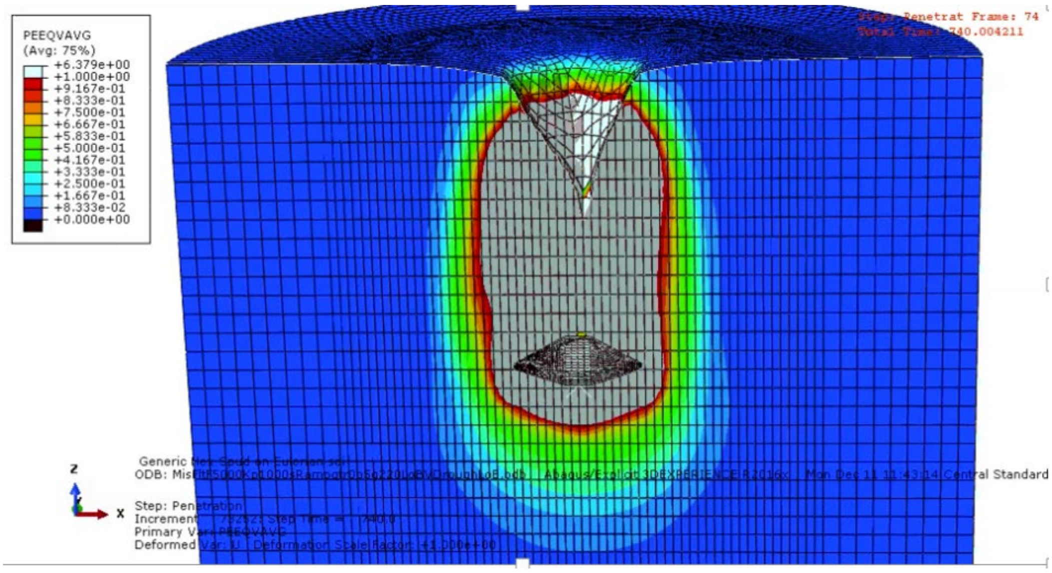

Figure 3b. Plastic strain contours with deep penetration

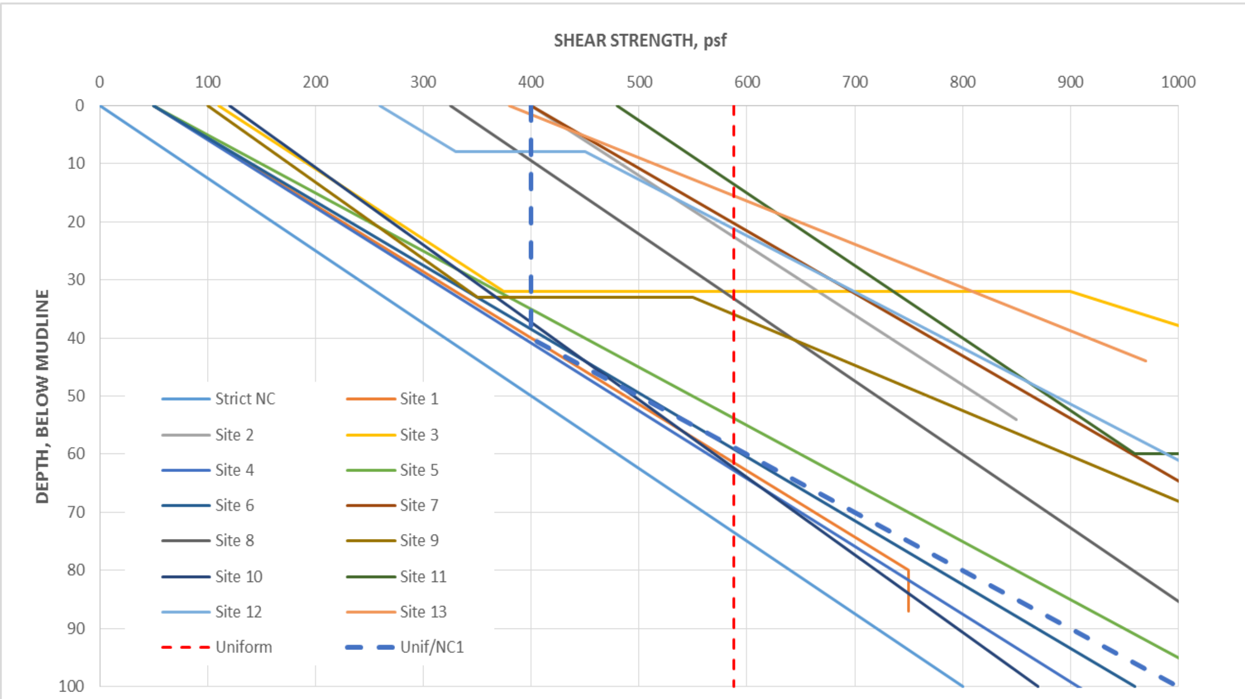

Figure 4. Soil strength profiles

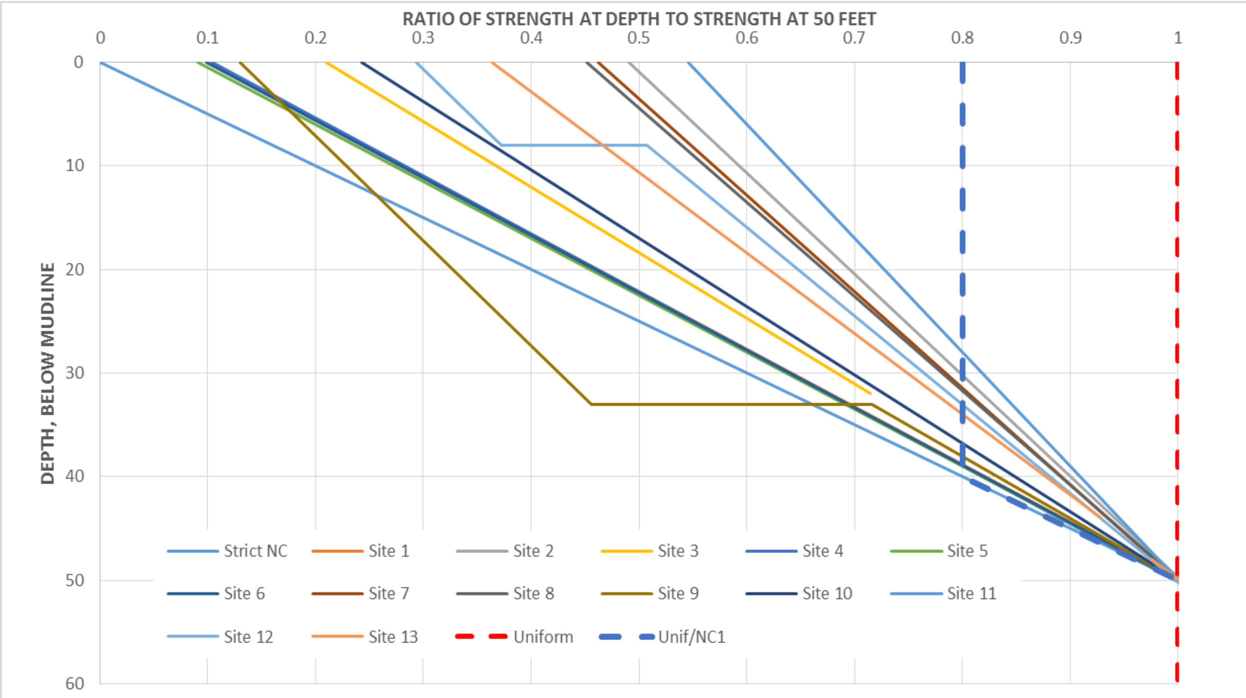

Figure 5. Normalized soil strength profiles

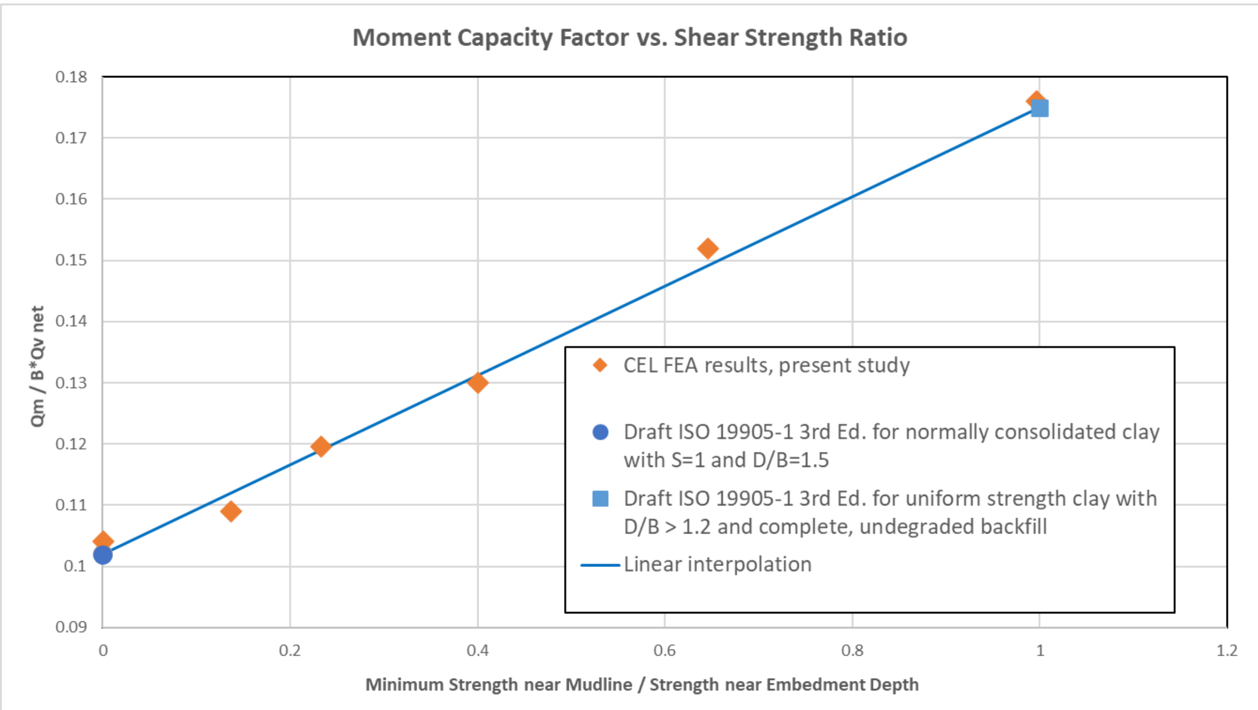

Figure 6. Normalized moment capacity vs. soil shear strength ratio

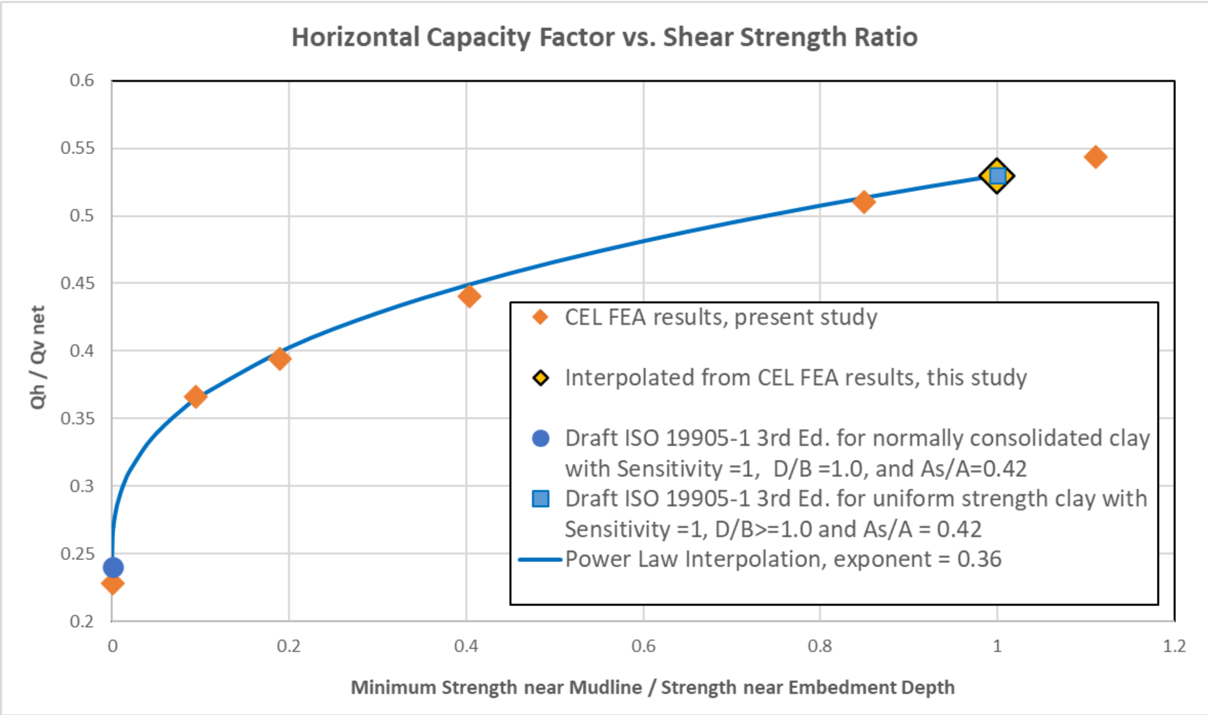

Figure 7. Normalized horizontal capacity vs. soil shear strength ratio