DEVELOPMENT OF A NEW ANNEX TO ISO 19905-1 THAT PROVIDES AN APPROACH AND CRITERIA FOR THE USE OF FOUNDATION CAPACITIES THAT ARE CALCULATED BASED ON SOIL STRENGTH RATHER THAN APPLIED PRELOAD

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ABSTRACT

This paper describes the background and development of a new Annex that is intended to be included in the next revision of the ISO 19905-1 [1] standard for Site-Specific Assessment of Jack-Up Units. The Standard, like its predecessor SNAME T&R 5-5A, was developed on the presumption that the foundation capacity is assured by the preload applied by the jack-up. This new Annex will provide an approach and criteria, in the context of a 'Step 2b' assessment, for the use of foundation capacities that are calculated based on the soil strength rather than the applied preload.

Scenarios for using this approach typically arise where the seabed is relatively strong, such that the vertical foundation capacity is significantly greater than the available preload footing load applied during installation, and the application of a vertical load exceeding the applied preload will not therefore result in significant further penetration of the spudcan into the soil. Where the criteria are satisfied, assessments adopting such an approach can yield greater foundation fixity, and consequently an improved outcome from the site-specific assessment.

The paper describes the background to, and development of, the approach and criteria, the specific sets of conditions where the approach may be applicable and the additional checks and safeguards that are required for such an assessment.

KEY WORDS: jack-up, spudcan, ISO19905-1, calculated, foundation, capacity.

1. INTRODUCTION

In 2015, a task group was established to develop a framework for the use of foundation capacities that are greater than those that can be proven by preloading, that could be incorporated as a new Annex within the ISO 19905-1 Standard [1] on jack-up site-specific assessments. The purpose of the framework is to provide an approach and criteria, within the context of a "Step 2b" assessment, for the use of foundation capacities that are calculated based on the soil strength rather than the applied preload, without requiring the added complexity of developing a Step 3b fully non-linear continuum foundation model. <u>Where the criteria are satisfied</u>, assessments adopting such an approach can yield foundation capacities and stiffnesses that are greater than those developed by preloading, and consequently an improved outcome from the site assessment. Such an approach is already available within DNV Classification Notes 30.4 [2] and the overall fundamental approach is similar to that adopted for shallow foundations which are not preloaded, but whose foundation stability is based on foundation capacity calculations using soil strength properties, following methods such as those described within ISO 19901-4 [3].

This approach is only admissible for certain combinations of soil profiles and specific spudcan geometries. Various risks and issues need to be considered by the assessor in order to determine whether the approach is applicable to the specific case being assessed. These criteria and considerations, as well as the key features of the framework described in the Annex, are outlined in the following sections.

2. BENEFITS OF CALCULATED CAPACITIES APPROACH VS. STEP 2B STANDARD APPROACH

To illustrate the benefit of such an approach, we consider the example of a jack-up with large skirted spudcans, installed at a dense sand seabed location, as shown in Figure 1. Once the skirts have penetrated into the seabed, the underside of the spudcan will then be in contact with the seabed surface. In such a situation, the vertical load required to penetrate the spudcan further into the seabed, i.e. to cause vertical bearing capacity failure, may significantly exceed the maximum available preload footing load of the jack-up unit.

If this situation were to be assessed following the standard Step 2b approach described in ISO 19905-1:2016 [1], the size of the foundation yield surface would be related to the preload applied by the unit. The foundation's rotational stiffness would be degraded during the elevated assessment based on the proximity of the combined Vertical-Horizontal-Moment loads to that yield surface. If the storm loads resulting from such an analysis were to exceed the unfactored bearing capacity envelope, then the foundation fixities would degrade towards a pinned foundation, resulting in significant load transfer from the foundations to the leg structures and holding systems. Although the corresponding additional spudcan penetrations, calculated in accordance with a Step 3a check, would be shown to be acceptable (as negligible additional penetrations would occur to expand the factored bearing capacity envelope such that the factored storm loads lie within it), the loading within the leg structures and leg holding systems may show unacceptable over-utilisations, such that the unit would not satisfy the requirements of ISO 19905-1:2016 [1].

Such an analysis of the foundations and the capability of the jack-up unit to operate safely at the location is, however, overly conservative as the structural utilisations calculated in such an assessment would be erroneously high. The true ultimate foundation capacities of the spudcan in this situation would be significantly greater, consequently the yield surface used in the nonlinear fixity iteration and the bearing capacity envelopes used for the bearing capacity check should be larger than those developed by preloading. This difference in foundation capacity is illustrated in Figure 1, in relation to the combined Vertical-Horizontal bearing capacity envelope.

If the true foundation capacity were to be used, instead of the preload, the corresponding degradation of rotational foundation stiffness for a given foundation load would be less and the loading within the leg structure and holding systems would be reduced. In terms of the overall assessment outcome for the unit, the use of foundation capacities that are calculated based on the soil properties would, in this situation, demonstrate an improvement in the component utilisations and the assessment outcome that may otherwise have been shown to be unacceptable if a preload-based yield surface, based on a Step 2b or Step 3a level of assessment, was adopted.

The fundamental objective of using larger foundation capacities is, therefore, to improve the estimates of structural utilisations resulting from the site assessment, as those utilisations would typically govern in such a case.

It must be stressed that the approach outlined in this paper is not intended as a substitute for preloading a jack-up at a site. A jack-up unit, whose suitability for safe operations at a site is justified on the basis of a site assessment that has relied upon calculated foundation capacities, should still be preloaded to the maximum preload footing reaction, as defined in the unit's marine operations manual. Preloading of the foundations in such situations serves both to confirm the unfactored predicted penetration response and to ensure that the spudcan-soil contact condition assumed by the site assessment is realised during installation.

3. CRITERIA FOR USE OF CALCULATED FOUNDATION CAPACITIES IN JACK-UP SITE ASSESSMENTS

The calculation approach outlined in this paper can only be used for very specific combinations of spudcan geometries, penetration conditions and soil types. The three main criteria that all need to be satisfied are related to:

- Foundation bearing capacity profile,
- Foundation geometry and spudcan-seabed contact,
- Soil type.

3.1 Foundation bearing capacity criterion

The use of calculated foundation capacities is only applicable if the ultimate vertical bearing capacity of the spudcan, as determined from the spudcan penetration curve, is significantly greater than the applied preload. To ensure adequate safety, the calculated bearing capacities must be determined using characteristic soil strength parameters in combination with a material factor on the soil strength parameters. The definition of characteristic soil strength in this context and the appropriate material factor to be used are discussed in Sections 4.1 and 4.2. Consequently, this approach is only beneficial if the gross vertical bearing capacity calculated, using <u>material factored</u> characteristic soil strength parameters, after preloading, $Q_{\mathrm{Vo,M\text{-}F}}$, is greater than the initial gross ultimate vertical foundation capacity established by preload operations, Q_{Vo} :

$$Q_{\mathrm{Vo,M\text{-}F}} > Q_{\mathrm{Vo}} \tag{1}$$

The condition described in (1) will only arise for appropriate spudcan geometries and soil conditions, and specifically where the spudcans are founded upon competent soil layers.

$Q_{\mathrm{Vo,M\text{-}F}}$ is related to the vertical load level above which significant further penetrations are predicted to occur. This corresponds to the load level at which the spudcan vertical bearing capacity profile (calculated using material factored characteristic soil strength parameters) ceases to exhibit an increase in capacity with negligible further displacement, i.e. point A as shown in Figure 2.

Where the spudcan penetration curve indicates the possibility for punch-through to occur for vertical loads that exceed the applied preload, however, specific consideration should be given to ensure that a safe estimate of the punch-through bearing capacity is evaluated to avoid any potential for a punch-through failure to occur during elevated operations.

3.2 Foundation geometry and spudcan-seabed contact criterion

Acceptable foundation geometries must not experience further penetration for vertical loads exceeding the applied preload, up to the vertical bearing capacity calculated in accordance with the methods described in the following sections, for the particular soil conditions encountered at the site being assessed.

The majority of spudcans in use by jack-ups are not appropriate for the use of these calculated capacities. Examples of unsuitable geometries would include all unskirted conical-based spudcans, such as those shown in Figure 3, as those spudcan shapes will progressively penetrate deeper into the seabed as the load on the spudcan is increased. There is therefore no plausible mechanism by which those foundation types would exhibit a step-change in foundation capacity of the type shown in Figure 2. These common spudcan foundation shapes would be assessed in accordance with the existing approaches described in Annex A of ISO 19905-1:2016 [1].

Typical examples of appropriate spudcan geometries and penetration conditions after preloading for use with calculated foundation capacities are presented in Figure 4. In each of the cases illustrated, either full spudcan-seabed contact is achieved by preloading, or by infilling any remaining voids present between the base of the spudcan and seabed surface with competent material after preloading. For flat-based spudcans, or skirted spudcans with a predominantly flat-underside, full contact of the portion of the spudcan underside and the seabed surface that is used to define the calculated foundation capacities and stiffnesses must be achieved. This is especially important where the seabed is not flat or where existing spudcan footprints or other irregularities in the seabed are present, see Section 5.2. Alternatively, any remaining voids present between the base of the spudcan and the soil should be demonstrated to not affect the foundation stiffnesses and capacities. If this not possible, the voids must be accounted for in the stiffness and capacity calculations; however, such situations may not comply with the criterion stated in Section 3.1.

Prior to the infilling of any voids, the spudcan-soil contact area should, as far as possible, be achieved by preloading, to minimise the uncertainties associated with the behaviour of infill material during storm loading. Where it is intended to infill internal voids with competent material, the spudcans would need to be equipped with the necessary pipework and valve arrangements to enable infilling of the voids after preloading, verification of the infilling and subsequent closure of the valves prior to elevated operations to prevent the escape of the infilling material during storm loading and hence assure the calculated foundation capacities and stiffnesses.

For the spudcan-seabed contact conditions illustrated in Figure 4, the tip penertration of the spudcan, or skirt, required to achieve full base contact by preloading is denoted as $h_{2,\mathrm{fullcontact}}$. In the case where full contact is achieved by infilling of any remaining voids with competent material after preloading, $h_{2,\mathrm{fullcontact}}$ corresponds to the tip penetration of the spudcan, or skirt, achieved by preloading prior to infilling of the voids.

3.3 Soil type criterion

As stated in Section 3.1, the use of calculated capacities is restricted to situations where the material-factored calculated capacity exceeds the applied preload. Furthermore, the acceptable foundation geometries described in Section 3.2 are only acceptable if the criterion described in Section 3.1 is satisfied. Consequently, locations with soft clays or relatively loose sands, or a soil profile with a punch-through risk for loads at, or slightly greater than, the applied preload would not be appropriate, even for skirted or other appropriate foundation geometries. For these situations, further penetration would typically be predicted to occur for footing loads greater than the applied preload.

This approach is also not suitable for use at locations where the seabed comprises soils that exhibit significant strength degradation under cyclic loads as these soils may develop bearing capacities during storm events that can be less than, rather than greater than, those developed by preloading. A detailed Step 3 assessment, including the cyclic effects on the soil parameters, would therefore be required. In this regard, materials of particular concern would include, amongst others, any un-cemented carbonate soils or any other soils where silt-sized particles, and in particular non-plastic silt, comprise a significant fraction of the material.

4. CALCULATION APPROACH

This section describes the overall assessment process and the background to the framework for cases where all the criteria described in Section 3 can be satisfied. Essentially the gross vertical bearing capacity calculated after preloading, using material factored characteristic soil strength parameters, $Q_{Vo,M\text{-}F}$, is used to define the vertical component of the bearing capacities. The ultimate horizontal and moment capacities are then calculated, in the usual manner described in Annex A, based on the corresponding net vertical bearing capacity. The initial stiffness calculation and stiffness degradation formulation is unchanged but the '*n*' value in the latter should be specified carefully. Each of the these aspects is described in further detail below.

4.1 Selection of soil parameters

Where foundation capacities are calculated from estimates of the soil strength parameters, such as assessments of gravity-based structures or mat-supported structures, a greater margin of safety is normally required compared to situations where the vertical bearing capacity is proven by preloading during installation, as is usually the case for jack-up units.

The margin of safety is ensured within the resistance calculation through the combination of the selection of appropriate soil strength parameters and the application of a partial factor. The partial factor can either be implemented as a resistance factor applied to the calculated capacity, or a material factor applied to the soil strength parameters used as inputs to the calculation. The selection of the partial factor proposed for the Annex to ISO 19905-1:2016 [1] is described in Section 4.2.

In order to use the calculated foundation capacities approach, the quality and quantity of the soil data should be sufficient to enable a reliable assessment of the characteristic strength of each contributing layer. These data may need to be more extensive than for foundations proved by preloading. Potential variations in soil properties below each spudcan should be evaluated using geotechnical data correlated with geophysical data and, if applicable, knowledge from previous operations at the site.

The proposed Annex refers to the appropriate soil strength parameters for use in calculated foundation capacities as "characteristic values" for consistency with the definitions used elsewhere within the ISO 19900 series of standards. A characteristic value is defined as "a value assigned to a basic variable associated with a prescribed probability of not being violated by unfavourable values during some reference period", ISO 19900:2013(E) [4]. Further discussion on the characteristic values of soil properties is found in Clause A.5.3 of ISO 19901-4:2016 [3].

In the context of the proposed Annex for calculated jack-up foundation capacities, the characteristic soil strength should represent a site-specific cautious estimate of the soil strength that exists within the extent of influence that could affect the foundation response. The Annex states that the selected value of the characteristic soil strength should account for the variability in the soil, and the uncertainty arising from the quantity and quality of the soil strength tests performed and the results obtained. The potential effects of softening or remoulding of the soil due to preloading, cyclic loading and changes in shear strength with time should also be considered.

For most scenarios the selected characteristic soil strength should be a value that is lower than the most probable value. However, for some scenarios, a higher soil strength is more critical to the assessment outcome, and in such situations the selected value should be higher than the most probable value. In particular, this latter situation can arise where the structural utilisations at the base of the leg, the leg-to-spudcan connection or the spudcan structural integrity are found to be critical, see Section 5.3.

Where sufficient data exists for a particular soil layer, a statistical treatment of the soil strength data may be used to define the characteristic soil strength for that layer, for example as described in DNV-RP-C207:2017 [5].

4.2 Material factors for drained and undrained (cohesive/non-cohesive) soils

As noted in the previous section, a partial factor on calculated foundation resistance can either be implemented as a resistance factor on the calculated capacity or as a material factor on the soil strength parameters used as input into the foundation capacity calculation. The decision of whether to ensure safety through the application of a material or resistance factor attracted much discussion within the Task Group. The existing site assessment framework for foundation capacities that are developed by preloading within ISO 19905-1:2016 [1] uses a resistance factor for both bearing capacity and sliding capacity, whereas comparable standards, such as EUROCODE [6], DNV Classification Notes 30.4 [2] ISO 19901-4:2016 [3] and NORSOK N-001 [7], apply material factors to the soil strength parameters. It is noted that, due to the nature of the equations that define base sliding in cohesive and cohesionless soils, the resistance factor applied to the base sliding component of sliding capacity can also be considered as a material factor. As an approach was already described in DNV Classification Notes 30.4 [2] for using calculated capacities in jack-up site assessments, the corresponding material factors of 1.3 on undrained shear strength, in cohesive soils, and 1.2 on the coefficient of friction (tanϕ'), for non-cohesive soils, included in that document were initially considered. It was noted that the recommended material factor on the coefficient of friction is lower than on undrained shear strength.

These factors differed, however, from the material factor of 1.25 adopted in ISO 19901-4:2016 [3] for both the undrained shear strength and coefficient of friction, tan(ϕ'). The adoption of 1.25 in the Annex was therefore attractive for the majority of the Task Group as it provided consistency across the ISO 19900 series of standards. One member of the task group advocated the application of a resistance factor, that is independent of soil type, to bearing capacities calculated based on characteristic soil parameters, instead of applying material factors, particularly as it would apply the same factor to all soil types.

Subsequently, the Task Group became aware that DNVGL-RP-C212 [8] (published in September 2017) would supersede DNV Classification Notes 30.4 [2] and include a single material factor of 1.25 for consistency with NORSOK N-001 [7] which states "*for geotechnical analyses, the material factor shall normally not be lower than 1.25*", i.e. the same as specified in ISO 19901-4:2016 [3]. Consequently, in accordance with Clause 7.3 of ISO 19901-4:2016 [3], the Annex has adopted a single material factor, γ_m, of 1.25 to be applied to the estimates of the characteristic soil strength parameters of undrained shear strength, s_u, for cohesive soils and the coefficient of friction, tan(ϕ'), for cohesionless soils.

4.3 Assessment of calculated vertical bearing capacity

The vertical bearing capacity should be calculated in accordance with A.9.3.2 of ISO 19905-1:2016 [1] using material factored characteristic strength properties to define the undrained shear strength = s_u/γ_m for cohesive soils and equivalent drained friction angle = $\{\tan^{-1}(\tan(\phi')/\gamma_m)\}$ for cohesionless soils, as described in Sections 4.1 and 4.2 above. The vertical bearing capacity should be calculated at the spudcan (or skirt) tip penetration depth determined from the unfactored penetration curve. In practice, this penetration depth should be identical to the tip penetration depth predicted using the material factored penetration curve, provided that the criterion described in Section 3.1 above is satisfied. If the spudcan is skirted, the depth to the base of the foundation is effectively equal to the skirt tip penetration. Consequently, the weight of soil contained within the skirt should be taken into account.

4.4 Yield surface

The yield surface should be calculated in accordance with the equations already provided in A.9.3.3.2 using material factored characteristic strength properties to calculate the gross vertical bearing capacity, $Q_{V,M-F}$, and the corresponding net capacity, $Q_{Vnet,M-F}$. In cohesive soils, material factored characteristic strength properties should be used in the formulations for calculate C_H. The maximum spudcan-seabed contact diameter, denoted as B in Figure 4, can be used to calculate the foundation capacities and fixities, provided that full base contact is achieved. Alternatively, it should be demonstrated that the remaining voids do not affect the foundation stiffnesses and capacities, or that they have been accounted for in the calculations.

The same material factored yield surface should be used for both structural and foundation checks as the size of the yield surface will influence the non-linear fixity iterations, and consequently the distribution of loads between the spudcans, leg structure and leg holding system and hence the resulting structural utilisations.

4.5 Initial stiffness

The question of whether material factors should be applied to the soil stiffness parameters was discussed in detail by the Task Group. Initial discussions focused on the importance of maintaining the relationship between the soil strength parameters and the soil stiffness. In the case of cohesive soils this is expressed by the rigidity index, $I_r = G/s_u$. However, in the case of sand, the only soil parameter, apart from the spudcan's stillwater bearing stress, used to define the soil shear modulus in the expression provided in Annex A of ISO 19905-1:2016 [1] is the sand's Relative Density, rather than a strength parameter. Whilst it was acknowledged that the relationship between the shear modulus and the soil strength should be respected, the Task Group concluded that the material factor is required to address the uncertainty in the foundation capacities, not the foundation stiffnesses. Furthermore, the elastic foundation stiffnesses are used in the dynamic analysis component of the elevated assessment that defines the both the jack-up unit's natural period and the Dynamic Amplification Factors (DAFs) subsequently used in the quasi-static non-linear analysis. The Task Group considered it important that the rig's natural period and DAFs should be independent of whether the ultimate foundation capacities are determined based on the applied preload or calculated using soil strength parameters. The Task Group therefore concluded that material factors should <u>not</u> be applied to soil stiffnesses in this context.

The foundation stiffnesses should be calculated based on the appropriate contact diameter. As noted above, the maximum spudcan-seabed contact diameter can be used to calculate the fixities, provided that full base contact is achieved. Alternatively, it should be demonstrated that the remaining voids do not affect the foundation stiffnesses and capacities, or that they have been accounted for in the calculations.

Stiffness depth factors can be used where appropriate; the effect of the weight of the soil contained within the skirts can also be considered in the stillwater footing reaction used to define the sand shear modulus.

4.6 Rotational foundation stiffness degradation

For spudcan foundations that have been preloaded, storm loads that lie within the spudcan foundation's yield surface will result in elastic (i.e. recoverable) foundation displacements and rotations for moderate load levels. During the installation process the loads on the spudcan are increased to the preloaded footing reaction and then unloaded back to the 'stillwater' self-weight of the unit. Subsequent storm loading therefore results in some reloading of the foundation, which would normally produce a stiffer pre-failure response compared to a foundation that had not been preloaded.

Conversely, a foundation that relies on calculated bearing capacities may exhibit a softer response compared to a foundation that was preloaded to the calculated vertical capacity value. The difference in the foundation stiffness between the two situations would depend on various factors, including the ratio of the calculated vertical capacity to the preload applied.

An illustration of this difference can be inferred from the results of linear-elastic, perfectly-plastic finite element analyses presented by Templeton [9]. Figure 5 plots the variation of the secant rotational stiffness degradation parameter, f_r, with the normalised proximity of the moment load to the moment capacity, r_f, for a spudcan that has been preloaded and unloaded to a stillwater load prior to the application of moment loading, and where the moment loading is applied without any preloading phase. The curves indicate that the secant rotational stiffness of the spudcan that has first been preloaded degrades less rapidly than the spudcan that has been not been preloaded. Furthermore, the non-preloaded spudcan is observed to initially behave elastically (i.e. no degradation), up to $r_f = 0.2$, followed by an approximately linear reduction of secant rotational stiffness with respect to r_f. It is noted that this comparison is specific to the boundary value problem and assumptions described in [9] and may not be applicable for other cases, for example a cohesionless soil.

The normalised rotational stiffness degradation response (f_r-r_f curve) included with ISO 19905-1:2016 [1] implicitly includes the benefit to fixity degradation of preloading. Consequently, the default '*n*' value of zero, as stated in the standard, may not be appropriate in the situation where the foundation response is based on calculated capacities rather than those developed by preloading.

There is therefore some uncertainty, and an absence of readily applicable comprehensive research, into the variation of rotational stiffness degradation for partially preloaded foundations – i.e. where the foundation has been preloaded to a vertical load that is less than the calculated capacity that is used to define the foundation yield surface. Consequently, the stiffness degradation parameter '*n*' in ISO 19905-1:2016 [1], that defines the rate of degradation as the storm loading approaches the yield surface, needs to be carefully selected based on the best available data applicable to the jack-up and site. Fixity degradation relationships for such cases can obtained from an appropriate and detailed study of the soil-structure interaction, for example using finite element analysis. Where such site-specific data does not exist, the Task Group recommends that '*n*' be set close to 1.0 in order to achieve the most conservative rate of stiffness reduction available within the framework described in Section A.9.3.4.2.3 of ISO 19905-1:2016 [1]. The corresponding formula for 'f_r' for calculated foundation capacities therefore simplifies to (2).

$$f_r = 1 - r_f \tag{2}$$

4.7 V-H bearing capacity envelope used in the Step 2 bearing capacity check

The bearing capacity check should be performed following the approach described in Clause A.9.3.6.4 of ISO 19905-1:2016 [1] by comparing the factored spudcan reaction forces with the V-H bearing capacity envelope calculated using material factored 'characteristic' soil strength parameters. As uncertainties in the calculated bearing capacity envelope are already captured by the material factor, γ_m, no additional resistance factor is required for this type of assessment, i.e. $\gamma_{R,VH} = 1.0$. For skirted spudcans, the bearing capacity check should account for the submerged weight of the soil contained within the spudcan skirt.

4.8 Sliding check

The sliding envelope is calculated using material factored 'characteristic' soil strength parameters and no additional resistance factor, i.e. $\gamma_{R,Hfc} = 1,0$ as the material factor, γ_m, accounts for the uncertainties in the calculated sliding capacity. For skirted spudcans, the sliding capacity check should consider the submerged weight of the soil contained within the spudcan skirt and the potential for the critical sliding failure mechanism to pass through the soil contained within the skirts, as described in Bransby and Yun [10].

5. SPECIAL CONSIDERATIONS

In addition to the criteria and approach described above, the following risks and aspects should be considered prior to using calculated foundation capacities in a Step 2b assessment.

5.1 Hydraulic stability

Hydraulic instability, i.e. piping and/or scour, during installation or storm conditions may potentially reduce the contact area between the spudcan base and the seabed surface and should therefore be assessed. Where piping or scour are recognized as a potential risk to the foundation, the effects should be considered and any mitigation measures required should be implemented to ensure foundation integrity. General considerations regarding scour are already provided within Clauses 9.4.7 and A.9.4.7 of ISO19905-1:2016 [1].

5.2 Importance of spudcan-seabed contact

Guidance regarding spudcan-footprint interactions and hard sloping strata are already included in Clause 9.4 of ISO 19905-1:2016 [1]. Relatively flat-based spudcans that would be considered appropriate foundation geometries for use with calculated foundation capacities would, however, be more susceptible to adverse eccentric spudcan support caused by seabed unevenness or sloping strata. Such foundation geometries therefore require more careful assessment of spudcan-seabed interactions than for ‘standard’ conical based spudcans.

As noted in Section 3.2, any voids present remaining between the base of the footing and the seabed surface caused by seabed unevenness or footprints should either be demonstrated to not affect the foundation stiffnesses and capacities, be infilled with competent material or accounted for in the calculation of the foundation stiffnesses and capacities.

5.3 Spudcan-to-leg connection and spudcan structural integrity checks

Where calculated foundation capacities are used as the basis for the foundation yield surface, it is important to check that the storm footing reactions are within the designed structural capacity of the spudcans and the spudcan-to-leg connections. For cases where the maximum vertical component of the factored storm footing reactions exceeds the applied preload footing reaction, a detailed spudcan integrity check should be performed for the most onerous anticipated combination of Vertical-Horizontal-Moment foundation loads resulting from the use of calculated foundation capacities. The foundation loads used for such a check should be obtained from two separate assessments that use a) material factored low estimate characteristic soil strength parameters, and b) unfactored high estimate characteristic soil strength parameters, as the latter case may be more onerous.

5.4 Soil drainage

The analysis methods outlined in this paper were developed primarily for use in seabed conditions comprising uniform drained or undrained soils. For cases where fully drained conditions are assumed to apply, it should be confirmed that this assumption applies during both preloading and subsequent storm loading. If this is not the case, then partially drained or undrained conditions may apply; in such situations there is the potential for the bearing capacities during storm loading to be less than those developed by preloading, hence the approach described in this paper does not apply. This situation can be of particular concern where the soil includes silt-sized particles and / or compressible soil grains such as carbonate soils.

Where undrained conditions are assumed to apply, the in-situ undrained soil strength may increase over time under the static foundation load imposed by the jack-up. In such situations, however, a detailed Step 3 assessment using advanced analytical methods will be required to determine the appropriate increase of undrained shear strength that can be relied upon for the determination of the foundation bearing capacities.

6. POSSIBLE FUTURE DEVELOPMENTS

As alluded to in Section 4.6, it would be desirable to have a greater understanding of the rate of degradation of rotational foundation stiffness for spudcans that rely on foundation capacities that are greater than proven by the available preload capacity of the unit. Due to the apparent lack of directly relevant data in the technical literature, there is the potential opportunity, with further work, to develop less conservative expressions and parameters to enhance the benefits achieved from the use of calculated foundation capacities.

Example ‘go-by’ steps and calculations for a jack-up site-specific assessment reliant upon calculated foundation capacities are proposed to be developed for inclusion within the next revision of ISO 19905-2 [11].

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8. SYMBOLS

B	effective spudcan diameter at uppermost part of bearing area in contact with the soil
B_S	soil buoyancy of spudcan below bearing area i.e. the submerged weight of soil displaced by the spudcan below D, the greatest depth of maximum cross-sectional spudcan bearing area below the sea floor
C_H	horizontal capacity coefficient
f_r	foundation rotational stiffness reduction factor
G	shear modulus of the foundation soil
h_2	spudcan or skirt tip penetration depth
$h_{2,fullcontact}$	tip penetration of the spudcan, or skirt, required to achieve full base contact by preloading. In cases where full contact is achieved by infilling of any remaining voids with competent material after preloading, $h_{2,fullcontact}$ corresponds to the tip penetration of the spudcan, or skirt, achieved by preloading prior to infilling of the voids.
H	horizontal bearing capacity
I_r	rigidity index = G/s_u
n	secant rotational stiffness degradation parameter
Q_{Vo}	initial gross ultimate vertical foundation capacity established by preload operations
$Q_{Vo,M-F}$	gross vertical bearing capacity calculated, using material factored characteristic soil strength parameters, after preloading
r_f	failure ratio
s_u	undrained shear strength of cohesive soils
V	gross vertical bearing capacity
V_L	vertical reaction under the spudcan
V_{Lo}	maximum vertical reaction under the spudcan required to support the weight of the jack-up during the entire preloading operation (including all water buoyancy effects)
$W_{BF,o}$	submerged weight of the backfill during preloading
ϕ'	effective angle of internal friction for cohesionless soils, expressed in degrees
γ_m	material factor applied to characteristic soil strength parameters (s_u for cohesive soils, tan(ϕ') for cohesionless soils)
$\gamma_{R,Hfc}$	partial resistance factor for horizontal foundation capacity
$\gamma_{R,VH}$	partial resistance factor for foundation capacity

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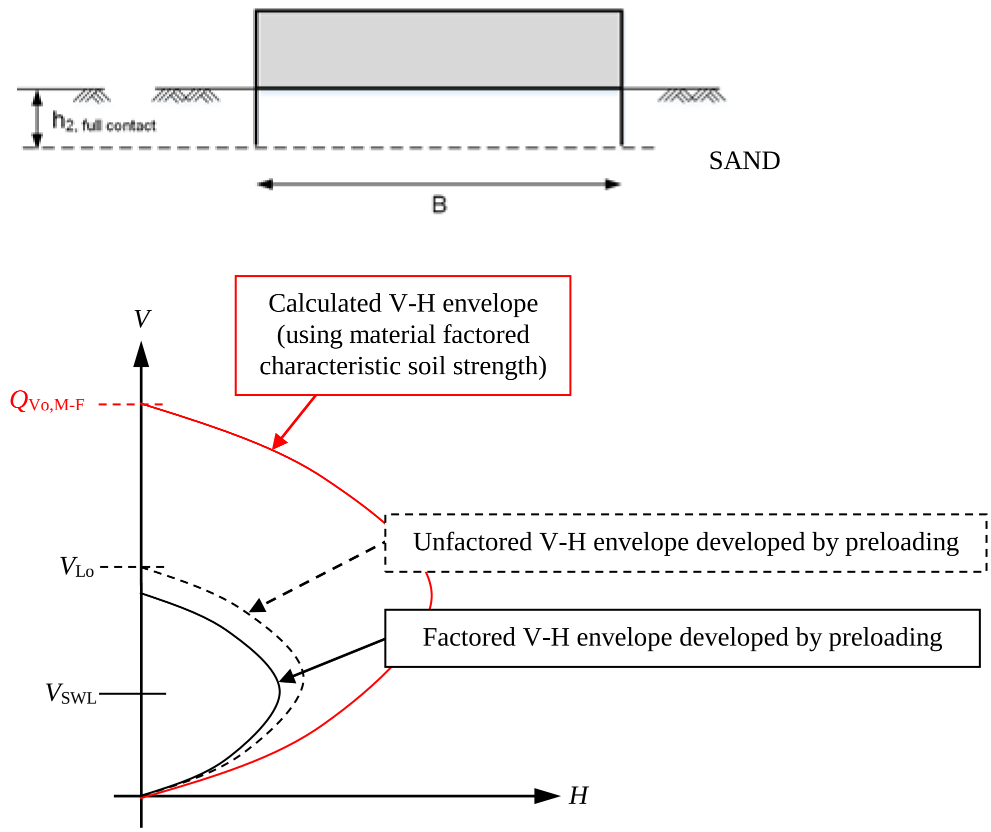

Figure 1 – Illustrative comparison of the combined Vertical-Horizontal bearing capacity envelopes developed by preloading and those based on calculated foundation capacities.

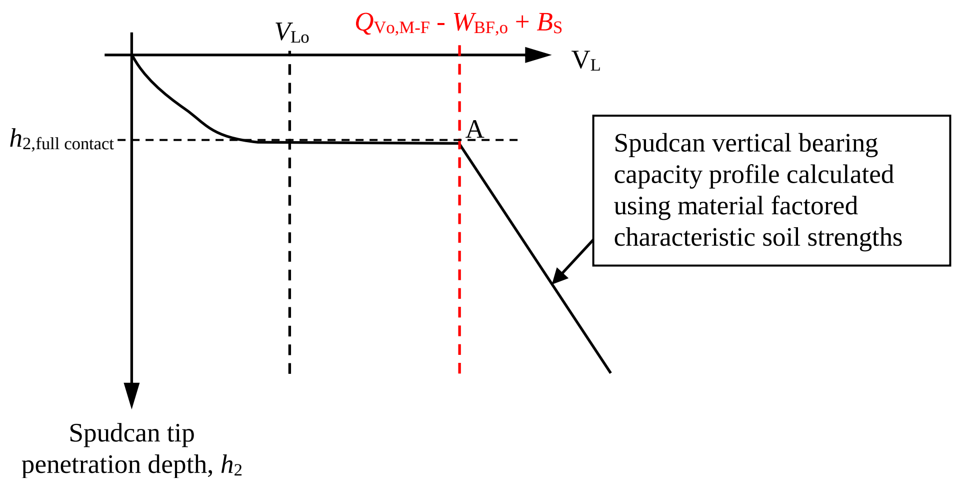

Figure 2 – Visualisation of $Q_{Vo,M-F}$ in relation to the spudcan vertical bearing capacity profile calculated using material factored characteristic soil strength parameters.

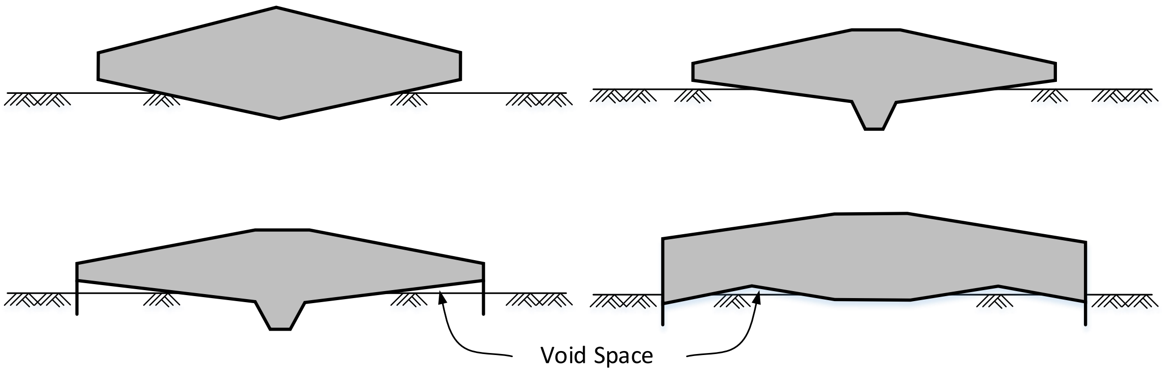

Figure 3 - Illustration of spudcan geometries and penetration conditions after preloading that are <u>not</u> appropriate for use with calculated foundation capacities.

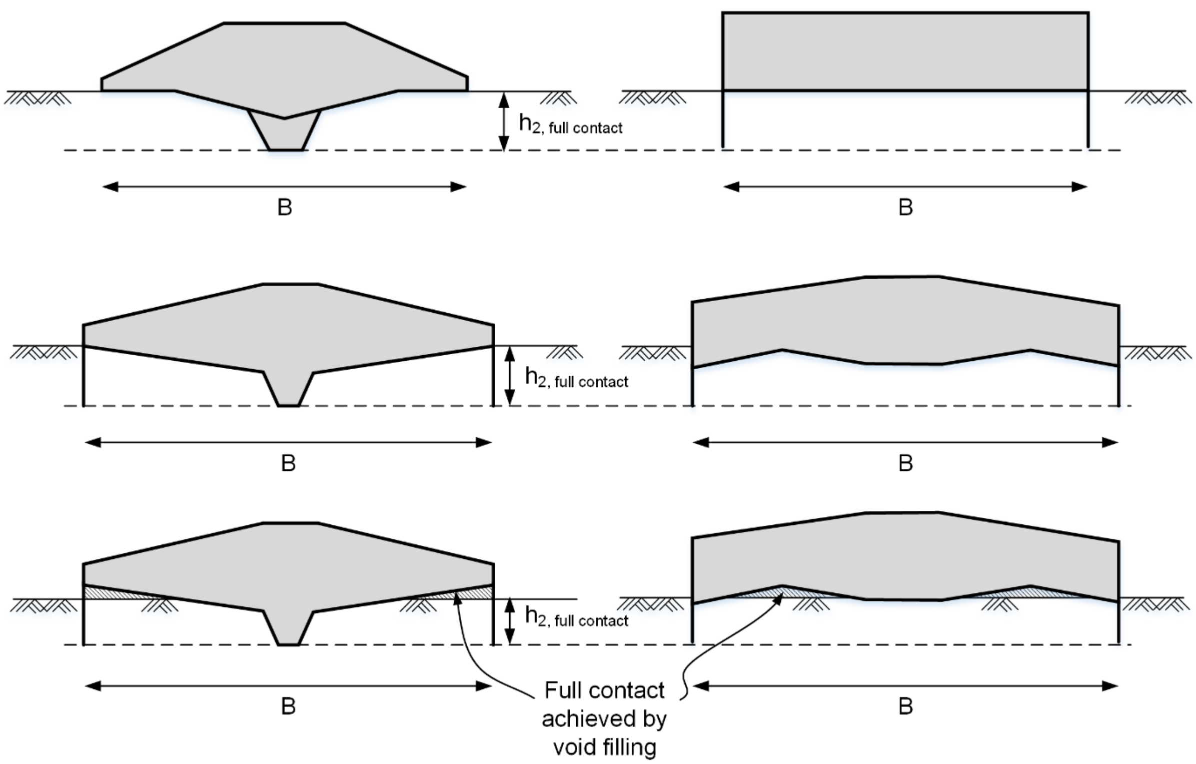

Figure 4 - Illustration of typical spudcan geometries and penetration conditions after preloading that are appropriate for use with calculated foundation capacities, with corresponding spudcan-soil contact diameter, *B*.

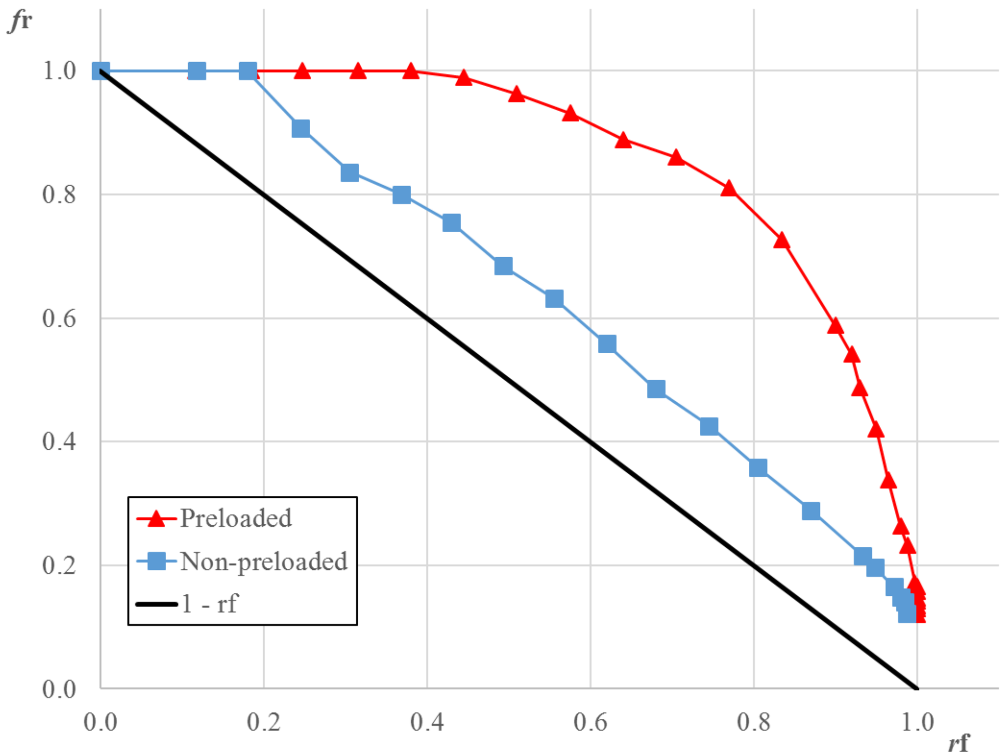

Figure 5 - Comparison of the secant rotational foundation stiffness degradation, f_r, with failure ratio, r_f, for a spudcan that has first been preloaded and unloaded to a stillwater load and a spudcan that has not been preloaded. Data inferred from results presented in [9].