SITE-SPECIFIC EARTHQUAKE ANALYSIS OF A WIND TURBINE INSTALLATION JACK-UP

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ABSTRACT
As per ISO 19905-1 [1], as part of the site-specific assessment for operating a jack-up, earthquake conditions shall be addressed. In highly seismic areas, dedicated earthquake analysis may be required and should include the assessment of potential for soil liquefaction if identified as a risk. For a recent wind farm development, a Site-Specific Earthquake Assessment (SSEA) was performed considering site conditions at all wind turbine locations combining the overall risks in various conditions, i.e. harbour and elevated on site, for the Wind Turbine Installation (WTI) jack-up. A quantitative risk analysis was performed, addressing site response analyses, liquefaction assessment, structural response and utilization including the effect of dynamic soil-foundation structure interaction, e.g. differential leg settlements. Additionally, post-earthquake survival conditions were addressed. This paper describes the methods used to quantify the overall risk.

KEY WORDS: WTI jack-up, earthquake, liquefaction, ISO 19905-1, site-specific assessment, leg settlement, quantitative risk analysis.

INTRODUCTION
Offshore wind farms, such as those developed in recent years in Europe, are frequently constructed using so called Wind Turbine Installation (WTI) jack-ups. These jack-ups commonly comprise a hull with four legs, jacking systems and a large offshore crane. As normally required, a site-specific assessment is to be performed in order to verify that the jack-up planned to be used is capable of withstanding the extreme environmental loading conditions, i.e. storm survival, and (crane-) operating conditions. As per ISO 19905-1 [1], as part of this site-specific assessment for operating a jack-up, earthquake conditions shall be addressed.

New wind farms are planned in other areas of the world, such as East Asia with recent developments offshore Taiwan and Japan. Unlike the offshore wind developments in Europe, those in Taiwan and Japan may be considered in highly seismic areas as can be observed from the seismic contour maps in ISO 19901-2 [2]. In highly seismic areas, a dedicated earthquake analysis is required and should include the assessment of potential for soil liquefaction if identified as a risk [3, 4].

For a recent project, Yunlin offshore wind farm development, such a dedicated earthquake analysis has been performed including a liquefaction assessment as this was identified early as a significant risk. The present paper aims to describe and discuss the analyses as performed. Topics highlighted are: site response analyses, liquefaction assessment, structural response and utilization including the effect of dynamic soil-foundation structure interaction, e.g. differential leg settlements.

THE PROJECT
The Yunlin offshore wind farm development is a large Offshore Wind Farm (OWF) off the west coast of Taiwan. The wind turbines, 80 in total, are planned to be installed by Fred Olsen Windcarrier by means of their Brave Tern WTI jack-up in two phases. The water depth varies between 8.6 m and 30.3 m across the site from locations with very dense layers of sand to locations with layers of silt to very silty sand to even silt and clay.

A maximum of 3 sets of wind turbines are loaded on the deck of the jack-up in the Taichung harbour. Next, the jack-up will sail on its own propulsion to the site where it will jack up to its operational airgap on the 1st location and install the Wind Turbine Generator (WTG) in approx. 24 hours, followed by the same sequence for 2 more wind turbines before returning to port to load the next sequence. For the WTI jack-up, preloading is a critical aspect which aims to warrant against overloading and excessive settlements in a storm and also plays a significant role in the foundation stability during an earthquake event.

From the seismic maps in ISO 19901-2, Annex B, latest edition, the seismic level at the site is significant with spectral values of $S_{a,map}(1.0s) = 0.3$g. Considering the level of seismicity combined with a risk of liquefaction identified for soils across the wind farm, a detailed Site-Specific Earthquake Assessment (SSEA) was initiated early 2019 and completed end of 2020.

THE ASSESSMENT PROCESS

The requirements of ISO 19901-2 are based on an accepted probability of failure due to an earthquake with a mean return period of 2,500-yr. This requirement is thus satisfied if loads, due to a 2,500-yr Return Period (RP) Extreme Level Earthquake (ALE) event, would not result in progressive collapse.

When evaluating the modes of operation of a jack-up, the number of sites and variations in both water depth and soil conditions in relation to the level of earthquake loading of the 2,500-yr earthquake, it is inevitable that this requirement cannot be satisfied in a deterministic approach, i.e. satisfy the requirements for every condition. This was also deemed unnecessary since the probability of a 2,500-yr earthquake in combination with a specific onerous configuration would be very low due to the time span in this configuration. Therefore, in early stage, the analysis was set-up with the intention to provide input for a Quantitative Risk Assessment (QRA) in order to verify that the probability of failure during the time span of the project would be acceptable to all parties involved. The intolerable limit for global failure of the jack-up during the entire wind farm installation was set to an annual probability of 10^{-3}. Global failure risks with an annual probability between 10^{-3} and 10^{-5} where considered to be in the ALARP region.

Structural integrity of the main crane was not explicitly included in the structural analysis. The probability of a significant earthquake in combination with a critical crane operation, i.e. crane limits with regards to hook load and outreach, is significantly reduced by the duration of this critical operation. The exposure risk was minimized by operational measures such as minimizing the time while operating the crane.

Harbour conditions and elevated operating conditions are included in detail, deriving their individual risk of failure considering the individual duration being in a specific condition. With 80 turbines and 1 harbour location, a total of 81 individual sites are considered, each site comprising different combinations of weight and airgap. Since the airgap and installation sequence could be subjected to changes pending the results of the QRA there were a total of 644 possible configurations to consider.

The QRA was to be performed for the probability of failure, i.e. to assess the structural integrity according to ALS principles considering the ALE event. This ALE assessment would require complex analysis to capture the failure mechanisms for the structure and foundation. For many reasons, such a fully integrated non-linear analysis of jack-up and soil was not achievable, amongst which:

- Fully integrated, 3D, analyses take a long time to run, in the order of weeks per case
- All 644 configurations need to be addressed
- ALS failure mechanisms are complex
- Integrating analysis of liquefiable soils is extremely challenging, so the risk of liquefaction was assessed separately

Therefore, the assessment was designed as follows:

- Non-linear site response analyses were performed, excluding structure
- Structural analysis of the jack-up based on ULS conditions in line with simplified methodology of a screening assessment including initial foundation stability analysis. Resulting probability of exceedance of the ULS criteria were subsequently translated into probability of failure.

- For critical sites, a dedicated settlement analysis was performed resulting in additional loads to the structural analysis
- Liquefaction assessment based on free-field conditions, i.e. excluding the effect of the jack-up, for each specific site

The risk of failure per site was set as the highest of the above as found and combined with the duration of operation for all sites and conditions into a total risk of global failure.

Additional requirement was taken that the jack-up shall be able to withstand a presumed 1.5 m differential vertical leg settlement with a reasonable weather window post-earthquake. Considering this as a damaged condition, the resulting weather condition was found acceptable with an approximate annual probability of non-exceedance of 95% for the site.

To optimise the amount of effort for this complex assessment, the WTG locations were clustered into five main groups (see Table 1) based on key aspects governing the foundation capacity and the magnitude of leg reaction, such as:

1. Spudcan penetration depth due to preload.
2. Leg length below jack up hull. This affects the structure's natural period and the leg reaction.
3. Soil type below spudcan. This governs the consolidation strength gain and cyclic degradation
4. Presence of an interbedded weak layer. A weaker layer interbedded within the soil below spudcan may impede the consolidation process or exhibit greater cyclic degradation.

The typical cone resistance profiles for each group below the predicted spudcan penetration depth are shown in Figure 1.

NON-LINEAR SITE RESPONSE ANALYSIS

Non-linear Site Response Analyses (SRA) were performed for all WTG locations. The mudline response spectra and the acceleration-time histories were derived for each location and subsequently fed into dynamic structural analyses. Assuming mudline instead of at spudcan penetration depth is expected to be conservative as the Peak Ground Acceleration (PGA) reduces with depth. Seismic induced cyclic shear stress profile was also derived for input in the liquefaction assessment per site.

A nonlinear time domain analysis was adopted as this approach is more appropriate than the equivalent linear approach when the seismic intensity is expected to be high, as observed at Yunlin site. The analyses were performed using the DEEPSOIL software [5].

The available soil information comprises Piezocone Penetrometer Test (PCPT) performed at each WTG location and boreholes at 9 locations. Static and cyclic laboratory tests (e.g. monotonic and cyclic simple shear, resonant column and bender element tests) were also performed on intact and reconstituted samples, which allow for characterisation of the soil properties required for performing SRA and assessing the liquefaction potential. Normalised relationships were derived for the key inputs used in the SRA (i.e. stiffness and damping) which allowed location-specific characterisation. A correlation was derived between the shear modulus and the normalised cone resistance (Figure 2). Generalised stiffness degradation and damping curves were also derived as a function of the effective vertical stress (Figure 3) and generally lies within the typical range for sandy soil [6]. This approach allowed for a more detailed seismic response assessment of each location. For DEEPSOIL analysis, the stiffness degradation curves were fitted using the MKZ hyperbolic model and the damping curves using a non-Masing MRDF-UIUC model [5].

STRUCTURAL ANALYSIS

The structural analysis was based on linear ULS criteria and conditions, meaning linear elastic simplification of the structure and foundation, except for the inclusion of P-d effect. The seismic ULS event is the Extreme Level Earthquake (ELE), in accordance with ISO 19901-2 [2].

The ground accelerations were derived for 3 orthogonal directions: one vertical, and two in horizontal direction called the principal and minor direction applied at the environment, i.e. the soil, which is connected to the structure, i.e. footings, via foundation springs. Earthquake directional dependency was included.

The Time History Analysis (THA) method was used to perform the ELE analysis which considers time-domain simulations of scaled historical seismic events. The non-linear SRA directly resulted in acceleration records in three orthogonal directions at seabed level for 7 [2] different earthquakes for 56 representative sites within the OWF. The variation of the earthquake records over the 56 Yunlin sites was investigated by comparing the average of the 7 earthquake lateral response spectra. Based on the limited variation between the sites it was considered appropriate to use the SRA results of a single conservative site as representative for the entire OWF.

The acceleration records resulting from the SRA for 3 different return periods (30, 100, 475-yr), and intermediate return periods 150, 200 and 300-yr derived by scaling of- and matching to spectra, were all assessed in the time domain simulations.

The open source earthquake software OpenSeesPy [7, 8] was used for the time domain simulations. A structural analysis model was made within OpenSeesPy that represents the dynamic behaviour of the jack-up in elevated conditions, see Figure 4. The structural model is an FE-beam model, assuming a linear elastic stiffnesses of both the structure and foundation which is in line with an ULS principles. The dynamic behaviour of the jack-up is dominated by its elevated weight, foundation stiffness, leg stiffness and leg-to-hull stiffness. The P-Δ effect has been taken into account in the calculations. A Rayleigh damping ratio of 5% was applied to the FE model.

The added mass in vertical direction for the spudcan (surrounding water and soil) is significant and assumed proportional to its displacement, as is suggested by [9]. Due to the relatively large spudcans, foundation radiation damping as a result of wave propagation in the soil was expected. Additional vertical foundation radiation damping was accounted for by means of a dashpot between the foundation and support point and the damping values are derived in accordance with [1].

The global structural verifications were performed using the internal forces resulting from the design action with partial load and resistance factors in accordance with ISO 19905-1 [1]. The global structural verifications included:

- Leg strength at lower guide and footing level (yield and buckling strength)
- The loads on the jacking system verified against its maximum holding capacity
- The loads at the leg-to-hull interface to remain within the values as verified in design

Initial foundation verification was performed with foundation bearing and sliding based on the preloaded capacity and generic method from [1]. This check was performed to identify the critical locations; for these cases specific foundation stability assessment, including the effect of consolidation strength gain and cyclic degradation, was performed as discussed further in the next section.

Figure 5 presents typical earthquake acceleration records and Figure 6 and Figure 7 show the associated load responses at the lower guide and footing levels respectively together with the structural and initial foundation utilizations. Figure 8 shows a typical initial foundation verification.

The global structural and initial foundation verifications as mentioned above were performed for simulations with each of the 6 different return period ELE events and for all 644 configurations of the jack-up. The different structural and foundation verifications were translated to utilizations based on the maximum of 12 headings and average of 7 earthquakes.

Since the ELE analysis was based on linear stiffnesses, linearization between utilizations and the design response spectrum peak accelerations was found to be appropriate. Consequently, the logarithmic relation between design response spectrum and return period means that there is a logarithmic relation between utilizations and return period. By means of interpolation of the utilizations the return period at which full utilization (UC = 1.00) was estimated to occur, were derived, see Figure 9.

As described in the next section, resulting differential leg settlements were estimated when the loads exceeded the initial foundation capacity, both vertical as well as horizontal. Due to the inclination of the jack-up associated with the vertical differential leg settlement, additional leg loads and bending moments were generated due to the elevated weight of the jack-up. Due to horizontal differential leg settlement, a resulting shear force was found. Both effects were combined and represented by additional loading, which was applied as external static loads acting on the FE-model. The earthquake assessment was then performed, with the THA method and structural verification according to ULS principles.

FOUNDATION STABILITY

When the ELE loads on the foundation were found to be critical, i.e. the initial foundation bearing verification was governing for the ELE return period and the limiting return period was lower than 100-yr, additional analyses were performed according to ISO 19905-1 [1] to: (i) assess the foundation capacity including the effect of cyclic strength degradation (and consolidation strength gain) and (ii) assess the differential leg settlement when the seismic foundation load was exceeding the calculated capacity.

The jack-up and its foundation will experience excitations, when subjected to an earthquake, that potentially results in uneven settlements of the spudcans amongst its four legs, i.e. differential leg settlement, when the initial bearing capacity is exceeded. These differential settlements of the spudcans, which can be both in the horizontal plane as well as the vertical are, when limited, not an immediate risk to the structure as long as this will not lead to structural collapse. For this project, the consequences of the spudcan settlements are considered based on ELE typical foundation loads for the 100-yr RP earthquake.

The VH yield envelope adopted for the assessment of each foundation stability and calculation of additional leg settlement is based on ISO 19905-1 [1] formulation. This requires the definition of horizontal and vertical ultimate capacities, H_{max} and V_{max}.

As indicated earlier, V_{max} and H_{max} are calculated considering consolidation strength gain and cyclic loading strength degradation. The latter has been computed considering an approach based on an equivalent number of peak cycles (N_{eq}), where the N_{eq} represents the number of peak cycles that generates the same degree of damage produced by the entire seismic event. Since the soil strength is also affected by the shape of the loading cycles, an average 'Cyclic Load Ratio' (ratio between the cyclic and the average stress component) is introduced and calculated for each location.

For the calculation of the differential leg settlements, the effect of load-time history exceeding the yield envelope was estimated assuming the yield envelope being constant during the entire time history. Note that with this assumption, the number of cycles that exceed the foundation capacity is likely overestimated. This is because, in reality, the yield envelope does not have a constant size during the entire event; instead, it gradually shrinks from the intact conditions (at the beginning of the event) to the calculated size (at the end of the event) as cyclic degradation occurs.

The ultimate foundation vertical capacity (V_{max}) under the design seismic load is evaluated using the finite difference software FLAC [11]. The key input parameter, i.e. the operative strength profile during the earthquake was computed at each element of the mesh, using apposite constitutive relationship coded by NGI. A first step in the analysis is to verify that the assumed in-situ soil strength profiles yielded a capacity equal to the preload at all the investigated locations. After which, the following procedure was adopted in a separate analysis for each location:

- The vertical still-water pressure is applied in undrained conditions. This has the effect of generating excess pore pressures in the soil.
- The excess pore pressures are allowed to dissipate for a certain time, leading to an increase in undrained shear strength.
- The effect of cyclic degradation on the undrained shear strength is considered according to the relationships derived for the site based on cyclic laboratory tests.
- The shear strength of the soil is updated in FLAC to account for strength change due to consolidation and cyclic degradation, leading to an operative shear strength.
- The vertical seismic capacity is calculated by penetrating the foundation at a constant displacement rate until an equilibrium is found, that is, the reaction variation is within an acceptable tolerance.

An axisymmetric, wished-in-place model was built to calculate the spudcan vertical capacity, with the soil modelled as a frictionless Mohr-Coulomb material.

The ultimate foundation horizontal capacity (H_{max}) was computed with hand calculation based on the operative soil strength calculated with the procedure summarised above.

The key processes of the numerical calculation for deriving the operative strength are depicted on Figure 10.

For the assessment of the seismic induced foundation permanent differential displacement the foundation load-time histories from structural analyses were used. In the structural analysis, linear elastic springs were employed to represent the ground response. Consequently, this model allowed the foundation reactions to exceed the foundation capacity and was unable to capture the actual foundation displacement. To estimate the foundation settlement from the calculated foundation reaction, a linear elastic – perfectly plastic soil model was employed. It is assumed that only elastic deformation takes place when the foundation load is lower than the capacity. In the instance when the capacity is exceeded, irreversible plastic deformation is induced and accumulates with every load cycle exceeding the capacity.

Newmark's sliding block concept [12] was employed to calculate the additional settlement, i.e. by double integrating the acceleration generated by the capacity exceeding loads with respect to time. The sliding acceleration is induced by the difference between the applied load (F) and the yield strength (F_{cap}) and calculated according to the second Newton's law of motion. For the condition where F is lower than F_{cap} with zero initial velocity, no acceleration is generated. For the condition where F is lower than F_{cap} but the foundations is in motion (initial velocity $\neq$ 0), the remaining foundation capacity ($F_{cap} - F$) will act to decelerate the foundation movement. This method was applied to estimate only the vertical settlement, considering each leg independent of the others. A calculation example for one leg is shown in Figure 11. Similarly, this method is applicable for horizontal sliding, however threating individual leg independent of other legs tend to unrealistically overpredict the differential horizontal displacement.

ALTERNATIVE SIMPLE SLIDING MODEL

When the above sliding block analyses would be applied to predict horizontal differential displacement between the jack-up's legs, the fact that each spudcan (leg) is essentially treated as an individual structure and foundation means that the effect of the presence of a jack-up structure connecting the four spudcans into one system is ignored. With differential horizontal displacement predicted between spudcans, it is clear that this would lead to load transfer from one spudcan to another and result in "leg splay" resulting in potentially high leg bending moments. The resulting difference in horizontal displacement were assessed to be physically impossible to occur.

An alternative simple sliding model was developed in order to account for the effect of the jack-up structure to differential horizontal displacement, and to include the effect of restoring load in the structural analysis.

A rudimentary calculation model was applied based on redistribution of loads once these exceeded the foundation capacity. These horizontal loads were redistributed from legs with high vertical load and associated low horizontal soil capacity, to legs with low vertical load and associated high horizontal capacity. The soil capacity was defined by the yield strength F_{cap} as determined by NGI. Spudcan displacements resulted in bending of the legs and reaction force. When spudcan horizontal and vertical load combination (F_h, F_v) fell outside the yield surface, instead of inducing horizontal inelastic displacements, the exceeding horizontal load (dF_h) was redistributed to the other legs for which the load combination still fell inside the yield surface, illustrated by the example Figure 12. The additional load resulting in leg bending moment due to inelastic horizontal differential displacements of spudcans was thus defined as:

$$dF_{h,leg} = \left(F_{cap,lowV} - F_{h,lowV}\right) < \left(F_{h,highV} - F_{cap,highV}\right) \qquad \textit{Equation 1}$$

where:

$dF_{h,leg}$ is the horizontal leg force per leg due to inelastic horizontal displacement resulting in a leg bending moment at hull

$F_{cap,lowV}$	is the horizontal capacity of the soil at legs with low vertical force
$F_{cap,highV}$	is the horizontal capacity of the soil at legs with high vertical force
$F_{h,lowV}$	is the horizontal load due to earthquake response on a windward leg at peak event
$F_{h,highV}$	is the horizontal load due to earthquake response on a leeward leg at peak event

VERIFICATION OF ALTERNATIVE SLIDING MODEL

In order to verify above, it was decided to build a simplified, essentially 2D, jack-up model that incorporates an elasto-plastic spudcan foundation model in order to provide a better estimate of the, effects of, horizontal differential settlements. Analyses were performed for a *Group 1* location, which is based on a seabed of sand and a water depth of around 20 m.

The analyses were performed using ABAQUS [13]. The earthquake accelerations were applied in a pure transverse (90 degrees) direction, i.e. 2D, taken from the structural analysis for this location. Earthquake acceleration levels representative of 100-yr RP and 500-yr RP were analysed.

The structure-soil-interaction was modelled by a combination of (uncoupled) elasto-plastic horizontal springs, purely elastic vertical springs and rotational free (i.e. pinned). Each spring was defined using a kinematic hardening element, similar as used in [14], which essentially behaves purely elastic until the yield surface is reached and close to fully plastic outside the yield surface. The foundation capacity was assumed to be defined based on NGI's yield surface (F_{cap}) for this location and to remain unchanged during the earthquake. That means that plastic deformations do not result in a change (increase or decrease) of the initial yield surface. The plastic capacities were matched to where the combined loads on the legs are found to hit the surface, i.e. $F_h \approx$ 6 MN for $F_v > V_{static}$ and $F_h = 16.2$ MN for all $F_v < V_{static}$. See for a plot of the spudcan loads and the applied yield surface – Figure 12.

As a result of the 100-yr RP loading, the development of inelastic horizontal displacement was found. As can be seen from Figure 13, due to the shape of the yield surface (different plastic load levels), legs on either side of the jack-up move outward resulting in "leg splay", or differential horizontal leg settlement, of about 13 cm which in turn results in a restoring horizontal load $dF_{h,leg} \approx 4$ MN per leg. The calculated horizontal load $dF_{h,leg}$ determined from the alternative sliding model was found as 5.3 MN.

By multiplying the accelerations by 2, a 500-yr RP earthquake was simulated. From this analysis inelastic displacements of about 20 cm and restoring load of 7.5 MN were found. When comparing the resulting leg bending moments from elasto-plastic analysis versus fully elastic analysis, it is shown that the additional bending moment compared to the purely elastic structural analysis as presented earlier, i.e. the change in bending moment, is important for structural verification. As shown in Figure 14, for 100-yr RP earthquake, the additional bending moment was about 65 MNm, however for the 500-yr RP earthquake the total moment was even 45 MNm *smaller* than in the purely elastic analysis. The latter observation is explained by the fact that in elastic response the horizontal loading on the soil is frequently outside the yield surface, moreover at the peak loading on all legs are in the plastic regime, which makes redistribution of forces in this instance impossible. There is development of inelastic differential displacements but the reduction of response compared to the fully elastic response is greater than the effect of inelastic displacements. This confirmed the result of the alternative simple sliding model yielding $dF_{h,leg} = 0$ MN for the 500-yr RP earthquake loads.

This confirmed the alternative rudimentary sliding model as conservative when added to the results of an elastic structural analysis in the peak loading of an earthquake.

LIQUEFACTION ASSESSMENT

Free field liquefaction assessment was performed for each WTG location. The potential for liquefaction is assessed by comparing the cyclic shear stress induced by the seismic events and the cyclic shear resistance along the soil profile. Sand layers will liquefy if the cyclic shear stress exceeds the cyclic shear resistance. These two stress parameters are traditionally expressed by normalising them with the effective vertical stress, and are denoted as Cyclic Stress Ratio (CSR) and Cyclic Resistance Ratio (CRR) (see [15] for more details). For free-field conditions, the cyclic stress ratio is defined as:

$$CSR = \frac{\tau_{cy,FF}}{\sigma'_{v0}} \qquad \textit{Equation 2}$$

In this assessment, $\tau_{cy,FF}$ is calculated based on the maximum shear stress, $\tau_{max,FF}$, obtained from the SRA for 100-yr RP. For each WTG location, the maximum shear stress profiles obtained from assessment using 14 time histories were extracted and the average (mean) profile was then computed. The equivalent free field cyclic shear stress was then estimated using the mean shear stress profile based on the following equation, as proposed in most liquefaction assessment methods, such as [16]:

$$\tau_{cy,FF} = 0.65 \cdot \tau_{max,FF} \qquad \textit{Equation 3}$$

The calculated CSR was also adjusted to a reference earthquake magnitude of M = 7.5 and effective vertical stress $\sigma'_v = 1$ atm to obtain $CSR_{M=7.5,\sigma'_v=1atm}$ value.

The above calculated $CSR_{M=7.5,\sigma'_v=1atm}$ value is compared against the cyclic resistance ratio at a reference earthquake magnitude of M = 7.5 and effective vertical stress $\sigma'_v = 1$ atm (i.e. $CRR_{M=7.5,\sigma'_v=1atm}$) for assessing the liquefaction potential. This value was obtained based on a correlation with the cone resistance q_{c1Ncs} (i.e. equivalent clean sand penetration resistance) as presented in [15]. To obtain a site specific correlation and potentially improve the calculated CRR, laboratory (i.e. cyclic simple shear) test results were interpreted and plotted together with the data and correlation proposed in [15]. The interpreted test results are presented on Figure 15 (cyan data points), together with the proposed site-specific design line for this project (black dashed line). The equation representing the proposed design line is provided in [15], with the $\varepsilon_{ln(R)}$ value being set to 0.13.

In addition to the liquefaction assessment under 100-yr RP seismic events, an additional assessment was performed to determine the liquefaction triggering return period, i.e. the return period at which the ratio of CRR/CSR is equal to 1. This was obtained by scaling the CSR using a scaling approach that was derived based the maximum PGA obtained from the SRA for different return periods. An example of the liquefaction assessment results from a selected location, in the forms of the calculated CSR for 100-yr RP, CRR and the liquefaction triggering return period profiles are presented in Figure 16. The liquefaction triggering return period at a representative depth at the WTG locations are used as one of the inputs in the quantitative risk assessment.

QUANTITATIVE RISK ASSESSMENT

The ULS criteria and its associated ELE event are intended to assess a structure to ensure that no significant structural damage occurs. Therefore, the derived earthquake return period at which full utilization (UC = 1.00) occurs is the maximum allowable ELE return period. The QRA was to be performed for the probability of failure, i.e. to assess the structural integrity according to ALS principles considering the Abnormal Level Earthquake (ALE) event. The ALS requirements are intended to ensure sufficient reserve strength to sustain large inelastic displacements, including those of the spudcan foundation, without complete loss of structural integrity.

For the QRA it was therefore proposed to use an ALE-equivalent return period for the structural verifications with:

$$S_{a,ALE}(T_n, RP) = S_{a,ELE}(T_n, RP)C_r \qquad \textit{Equation 4}$$

With the logarithmic relation between the return period and response spectra for different periods, as discussed above, the ALE-equivalent return period was derived. In lieu of a dedicated push-over analysis, the seismic reserve capacity factor C_r was conservatively considered as 1.3. T_n is the natural period of the jack-up. The $S_{a,ELE}$ is the peak response of as SDOF system with a natural period T_n for an $S_{a,ELE}$ spectrum with a return period at which full utilization is expected, herewith again using a site-specific logarithmic relationship between spectra and return period derived from the SRA.

Estimation of the ALE-equivalent return period as the minimum from either the return period of structural utilization, including the differential leg settlement if applicable, or the return period of this differential leg settlement itself or the return period of soil liquefaction.

The QRA process considered the onerous ALE-equivalent return periods for the WTI jack-up itself for all different configurations, as well as limiting return periods for adjacent structures, such as an erected WTG offshore or erected towers at pre-assembly. At each individual location and for each condition, an exposure duration estimate was linked with the annual probability of the onerous failure mode. This was further used to establish the relative contribution from the individual periods of exposure to seismic activity to conclude the overall seismic risk profile for the project. The project could benefit from the numerous load cases analysed by scheduling less onerous vessel loading conditions at challenging locations in order to reduce risk levels both in the QRA and for offshore operations. The overall risk level concluded from the QRA was found to satisfy the project acceptance thresholds. Furthermore, the QRA served to inform the project on relevance and priority of operational risk reducing measures for the execution phase.

GENERAL DISCUSSION

Verification of survivability of WTI jack-ups is normally performed in line with guidance provided in ISO 19905-1. For assessing a WTI jack-up against seismic activity reference is given to ISO 19901-2 for specific requirements. The latter is for all practical purposes developed for the design of offshore structures. This poses both challenges and shortcomings when the aim is to evaluate the seismic capabilities of a WTI jack-up within this framework.

Firstly, the WTI jack-up is an existing unit and any verification needs to be performed to the capacities of this structure, as it is not technically nor economically attractive to re-design and upgrade an existing WTI jack-up to improve capacities against seismic loads.

Secondly, the WTI jack-up will take on a large range of conditions and properties during operations. To mention some variables such as water depth and soil conditions at location, loading condition of unit (i.e. total weight) and operating conditions (i.e. shift of centre of gravity), that all affect the dynamic properties of the system and results in an increasing number of load cases should one consider all such variables changing independently.

Thirdly, the Seismic Risk Category (SRC) is to be defined for the structure in order to determine the assessment methodology. The standard [2] assumes a fixed offshore structure constantly exposed to seismic activity throughout its lifetime. This is not well suited for WTI jack-up operations for which the unit is exposed to seismic events for reduced periods of time, in different configurations, and only during the installation phase of the offshore wind farm.

As noted the primary ISO requirement [1, 2] for a jack-up is not met. However, here a risk assessment is performed taking all different operational conditions into account which leads to a lower risk by definition. It would be good if this could be considered in ISO guidance.

Harbour conditions should be addressed properly as the low water depth can result in high earthquake loading and the jack-up is in port for prolonged period of time. In this project, the risk is reduced significantly by allowing low airgap such that with high tides the hull is even partly submerged. When submerged, the earthquake risk is significantly reduced. The foundation at the harbour location is prepared with a gravel bed in order to limit leg penetrations and liquefaction risks. The hull is jacked to the average tide level over the entire duration in the harbour, resulting in a maximum airgap of only 2.5 m during low tides and partial submersion in approx. 50% of the time.

DISCUSSION ON FOUNDATION STABILITY

Although free-field liquefaction potential represents an important aspect to be considered for high seismicity sites, it may not fully represent the conditions below foundation structures such as the spudcans, where the soil conditions may differ significantly. Some key aspects that should be considered when assessing the liquefaction potential for the soil below the spudcans include the following:

(i) The peak shear stress is mainly governed by a large static component (i.e. the static vertical pressure of the jack-up, which is more than 500 kPa) which is likely applied to the soil in drained manners. The seismic induced shear stress transferred from the foundation to the soil below the foundation is expected to be significantly smaller than the static component. This is different from free-field condition where the static shear stress component is relatively small, due to the soil weight alone.

(ii) Considering the above, the cyclic stress generated below the foundation is not well represented by the element test generally used to assess free-field liquefaction (i.e. 2-way cyclic tests) and specific tests (where the undrained cyclic shear component is applied after a static drained shear component) should be performed to better characterise and model such conditions.

(iii) Without specific tests being performed, the shear stress time history along the plane of maximum shear is likely to be closer to one-way conditions (i.e. with cyclic amplitude comparable to the average loading) than two-way conditions (i.e. cyclic loading with an average close to zero) that is generally applicable for free-field conditions.

(iv) The soil behaviour under one-way loading conditions is generally different from two-way loading conditions, with a much lower propensity to liquefaction and cyclic softening for one-way loading conditions. This behaviour is also observed for the soils at Yunlin based on the review of the laboratory tests performed for both loading conditions.

(v) In addition to the above consideration, the beneficial effect of consolidation strength gain following the application of static load and the dissipation of excess pore pressure should also be considered.

Based on the above discussion, the risk of liquefaction for the free-field conditions at Yunlin is considered more critical than for the conditions below the spudcan, and for this reason this has been addressed as the main priority. It is believed that this assumption is conservative.

Both pinned conditions as well as with moment restraint were considered. It is seen that, with high fixity, the resulting loads are the largest compared to pinned due to the shift of natural frequency and earthquake spectral energy. In reality, the soil-structure interaction will be somewhere in-between these two boundaries. It is assumed that by considering both boundaries, the structural analysis is conservative. Consideration of stiffness and hysteretic damping in non-linear spudcan-soil interaction [10, 14, 17] could be included to improve the accuracy of the analyses.

CONCLUSION

This paper describes and discusses the earthquake risk assessment as performed for the WTI jack-up planned to install the 80 turbines for the Yunlin project. As the basic requirement as per ISO 19905-1 [1] could not be met, a detailed QRA was performed considering a large number of conditions including liquefaction assessment in order to meet the set required probability of global failure.

It is shown that, as fully integrated analyses are impractical, elastic structural analysis and liquefaction assessment could be performed separately for many different configurations and conditions. For grouped locations, the estimated differential settlements based on loads from the jack-up could be estimated and transferred back into loads for the structural analysis. Finally, results are combined into one risk of global failure considering harbour and elevated operations.

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FIGURES

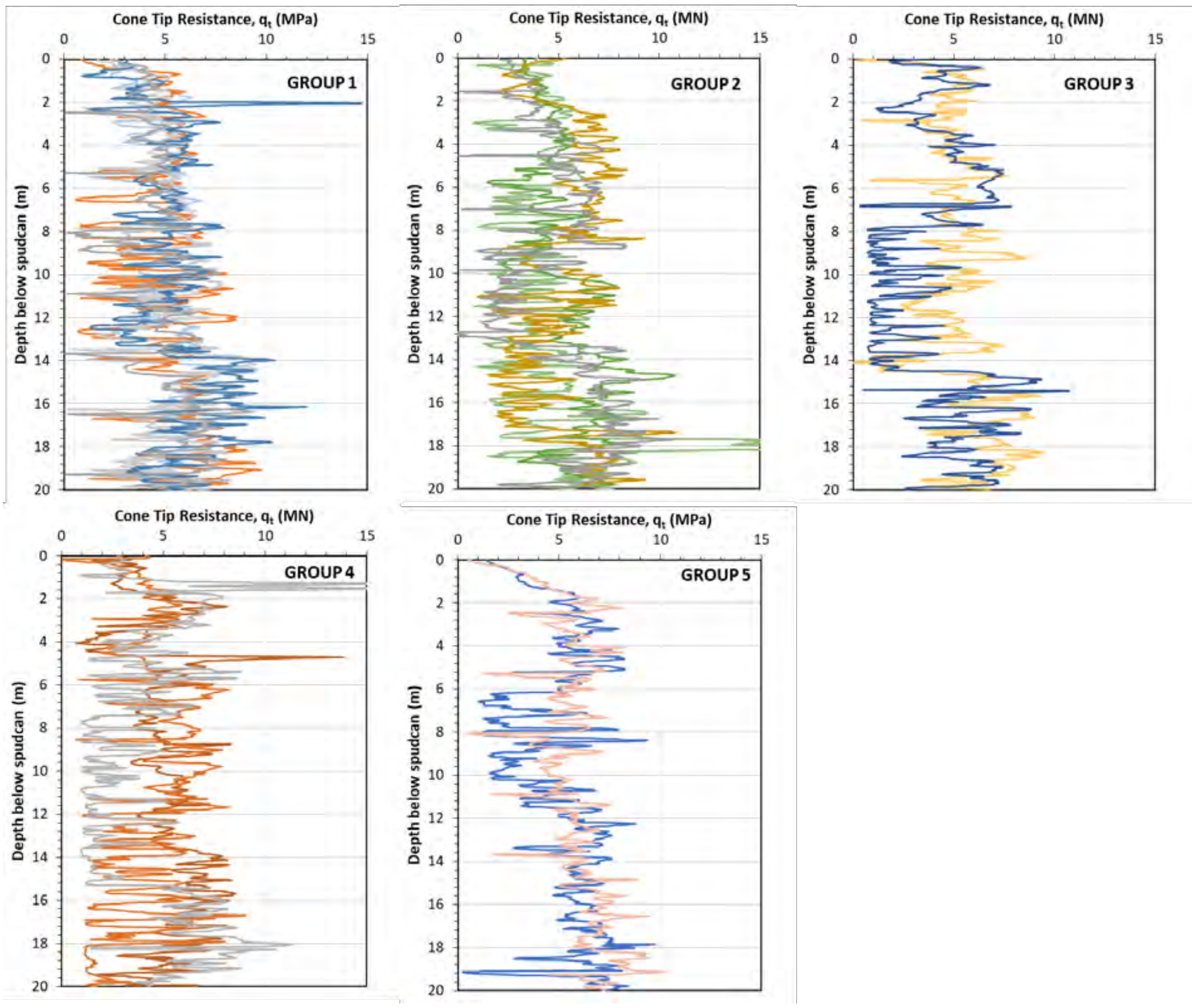

Figure 1 In-situ cone resistance profiles below predicted spudcan depth.

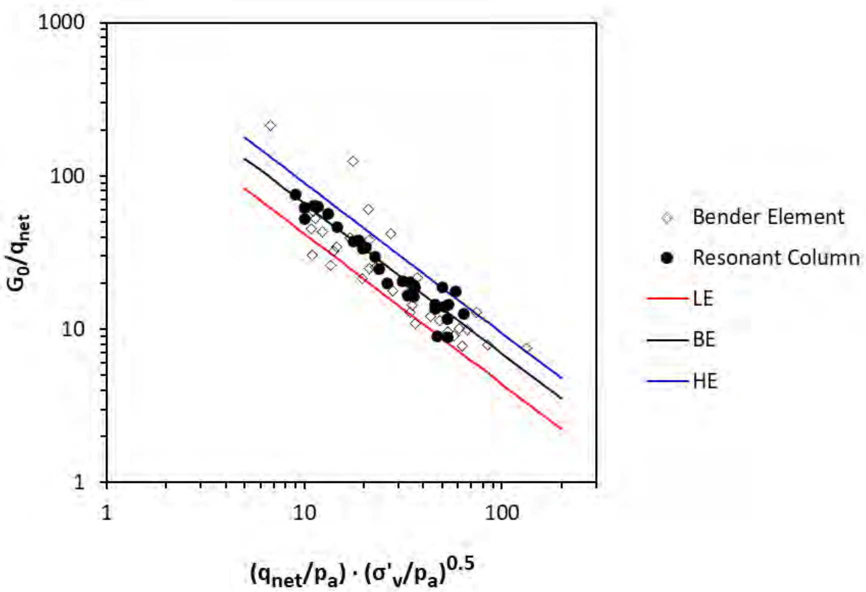

Figure 2 Correlation between shear modulus G_0 and net cone resistance q_{net}.

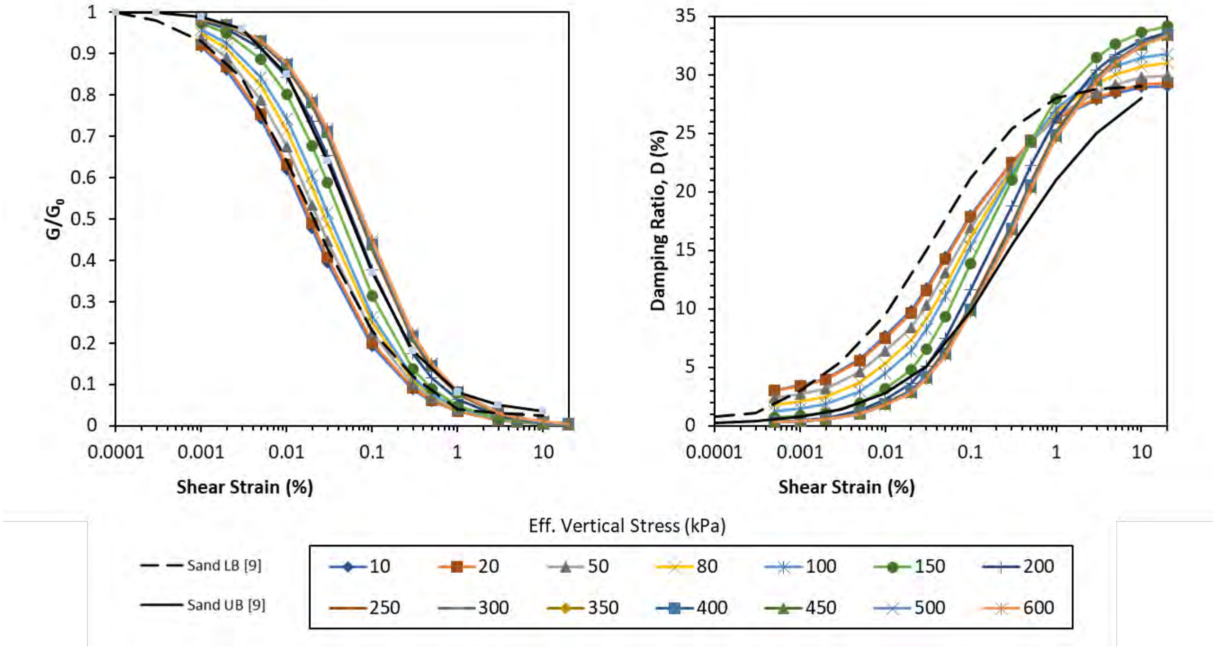

Figure 3 Generalised shear modulus degradation and damping curves as a function of vertical stress based on resonant column and simple shear tests data

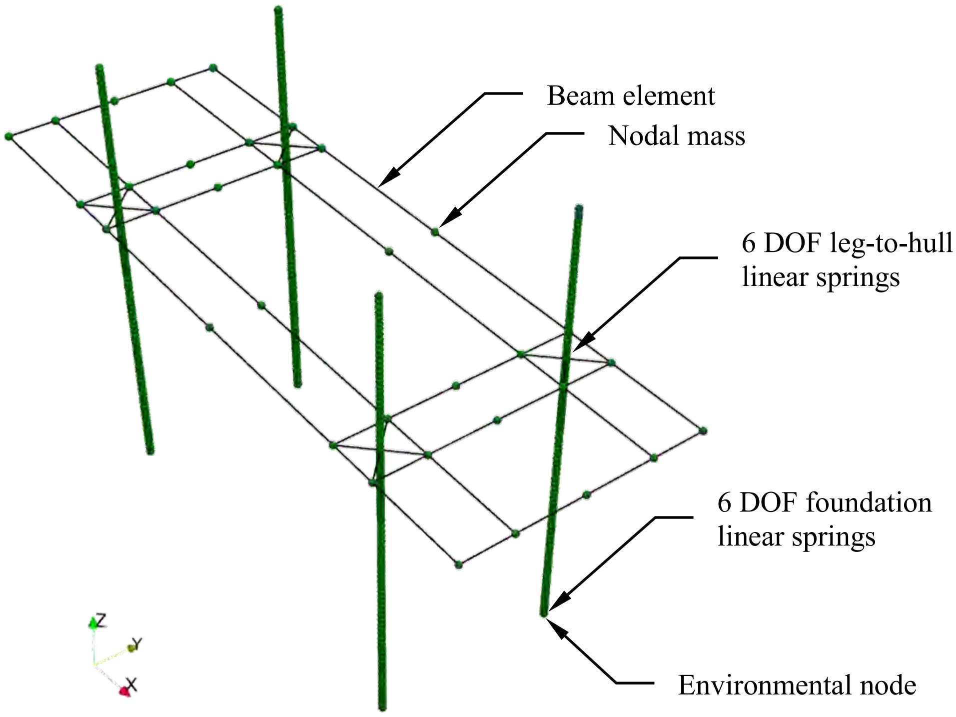

Figure 4 OpenSeesPy FE-model

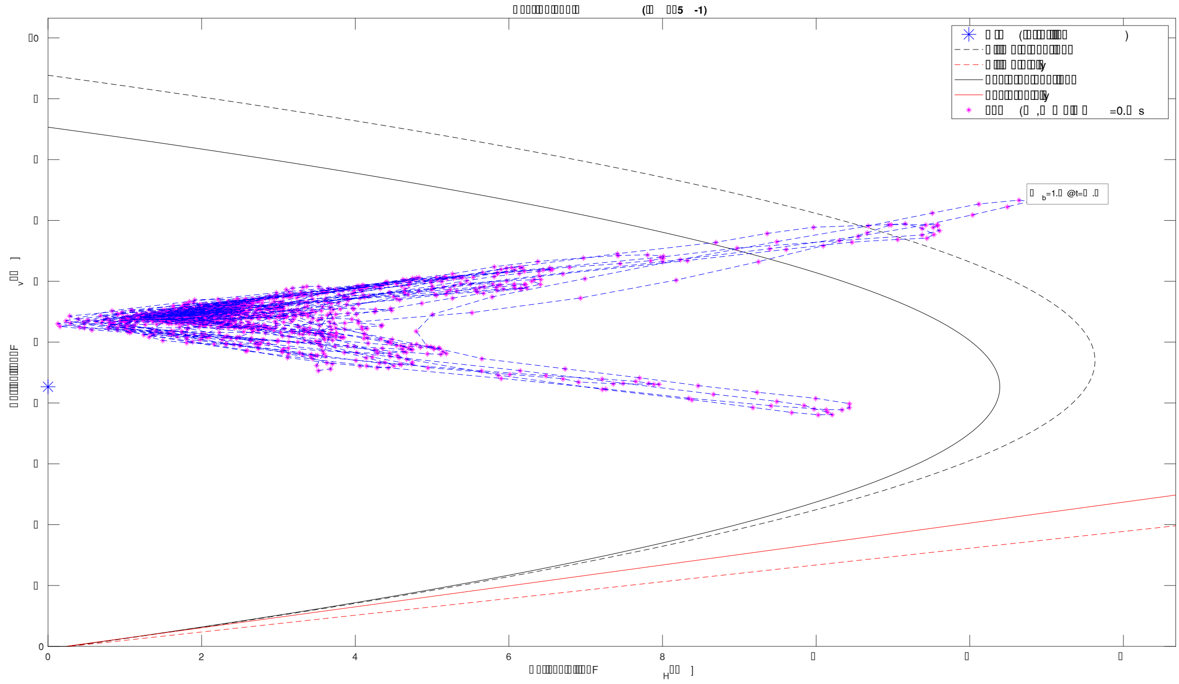

Figure 8 Typical foundation verifications

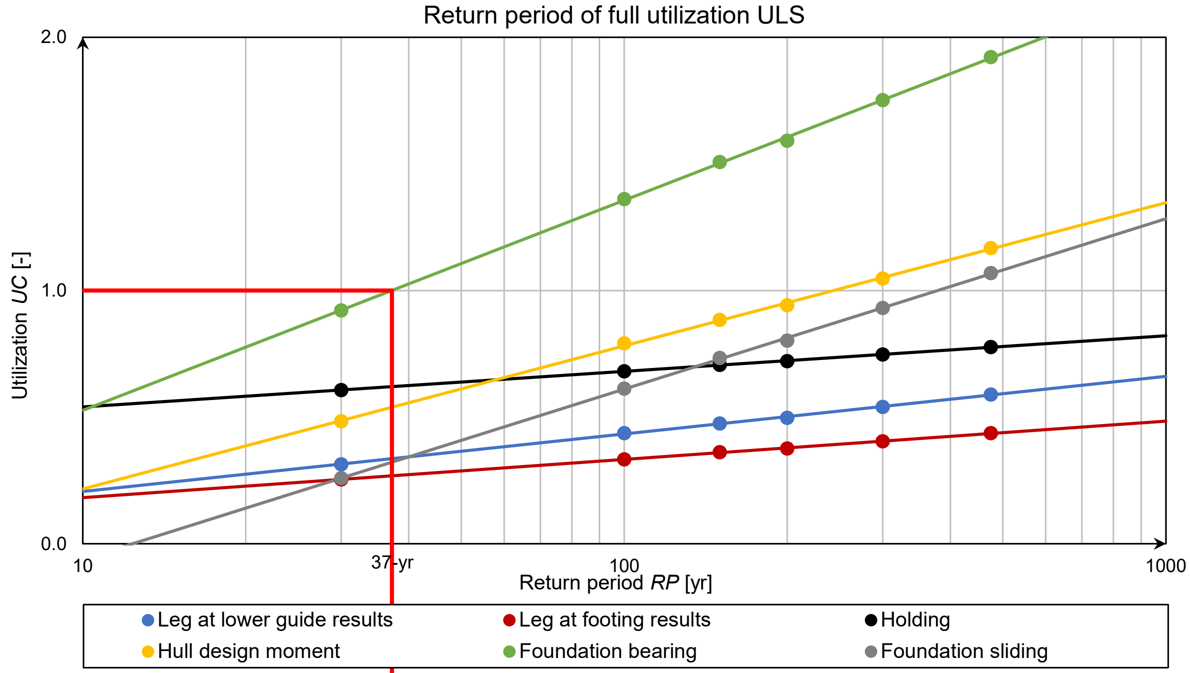

Figure 9 Linear interpolation of utilizations (data points are max of 12 headings and average of 7 earthquakes for a single site and configuration)

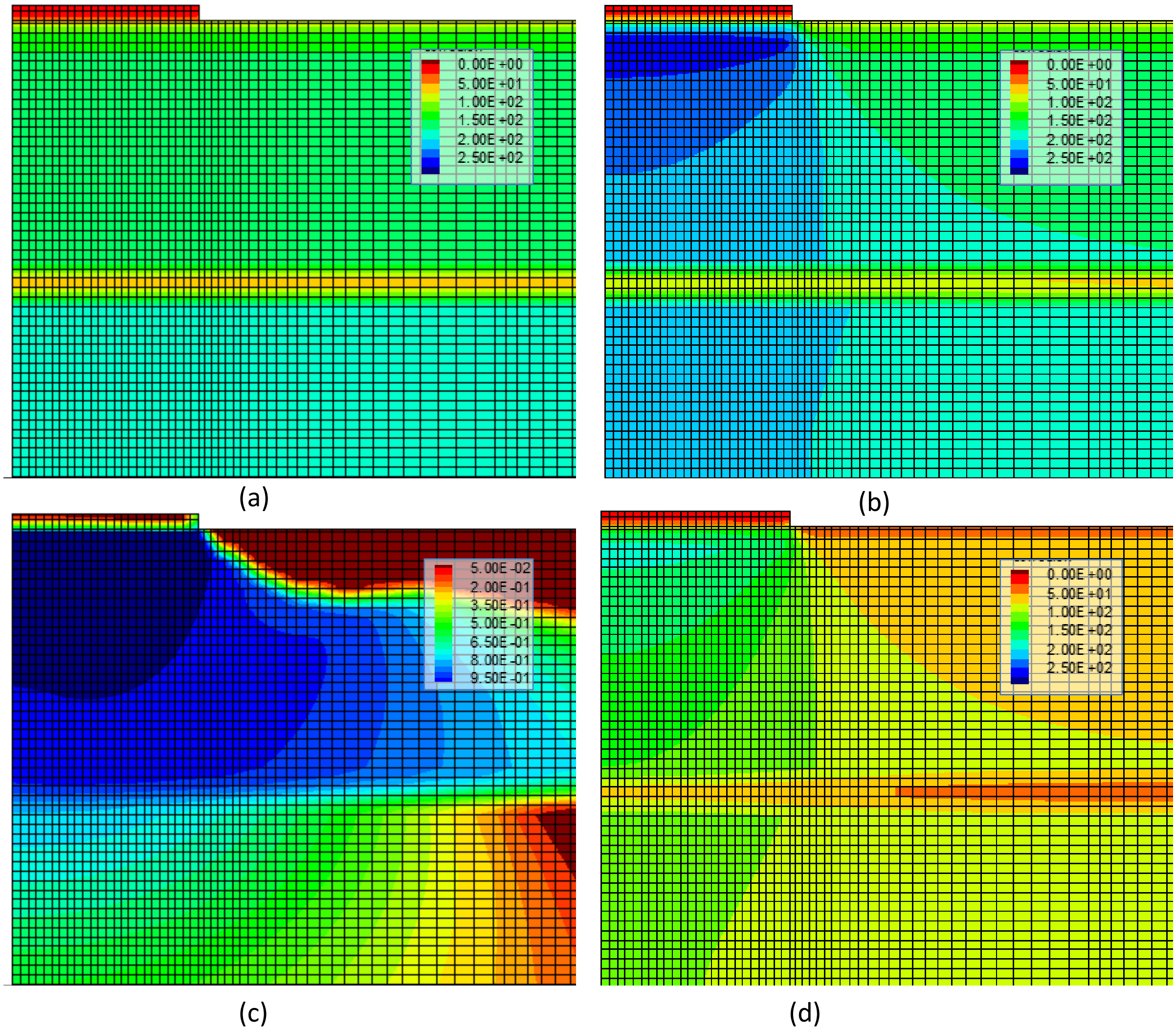

Figure 10 (a) In-situ undrained strength, (b) consolidated strength, (c) degree of consolidation, (d) consolidated cyclic strength

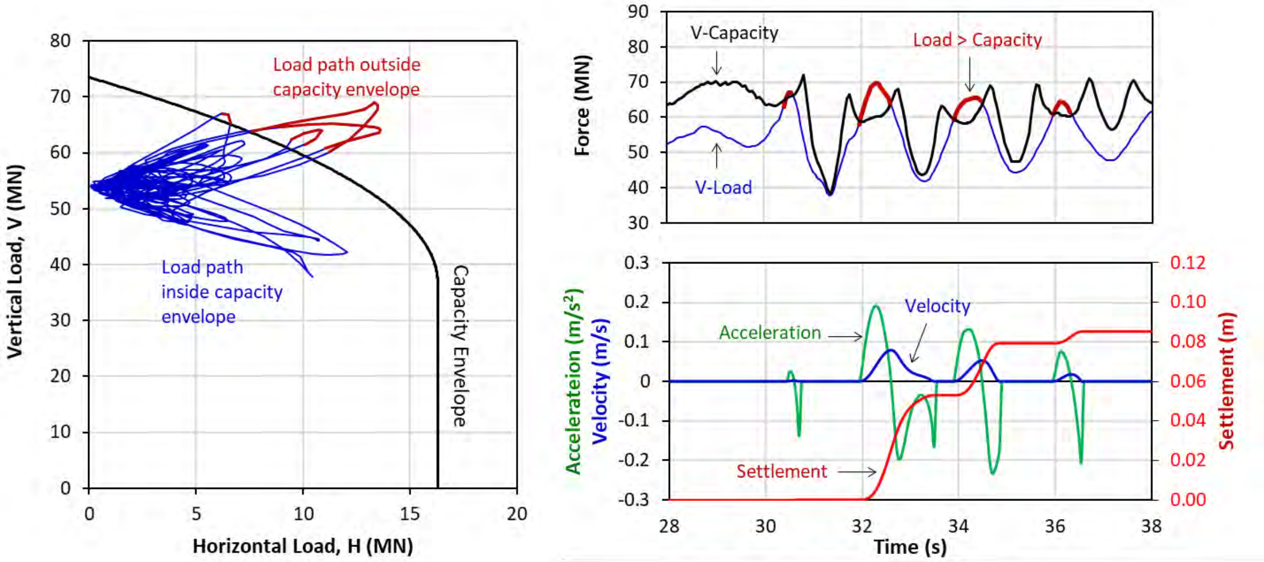

Figure 11 Calculation of additional leg settlement with Newmark sliding block concept

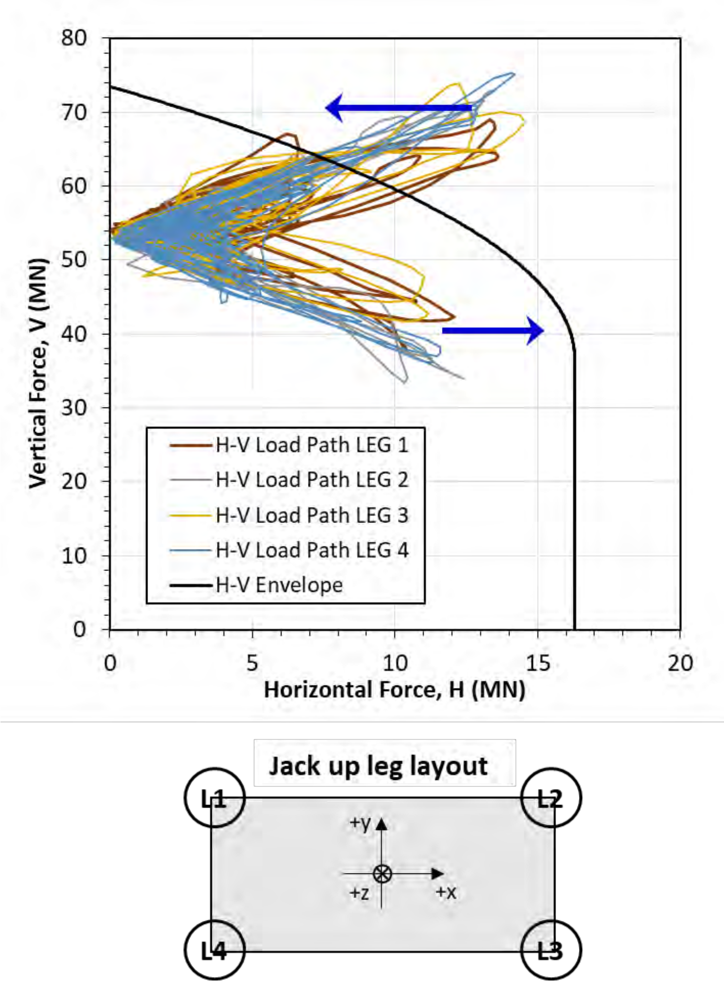

Figure 12 Example of foundation yield surface and footing loads

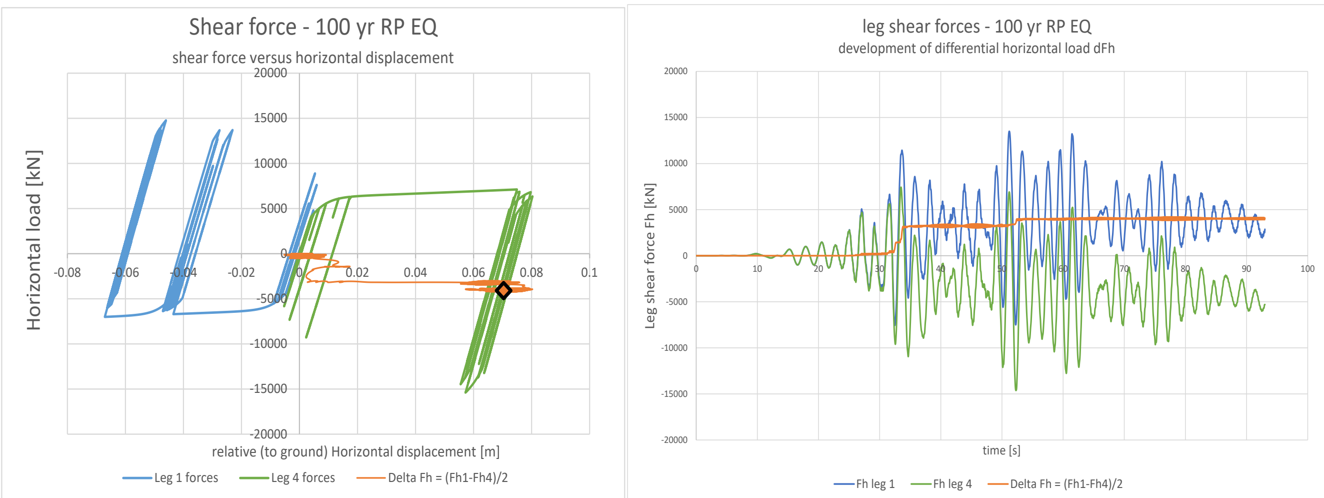

Figure 13 Development of differential inelastic horizontal displacement (differential leg settlement) and differential horizontal shear force between legs

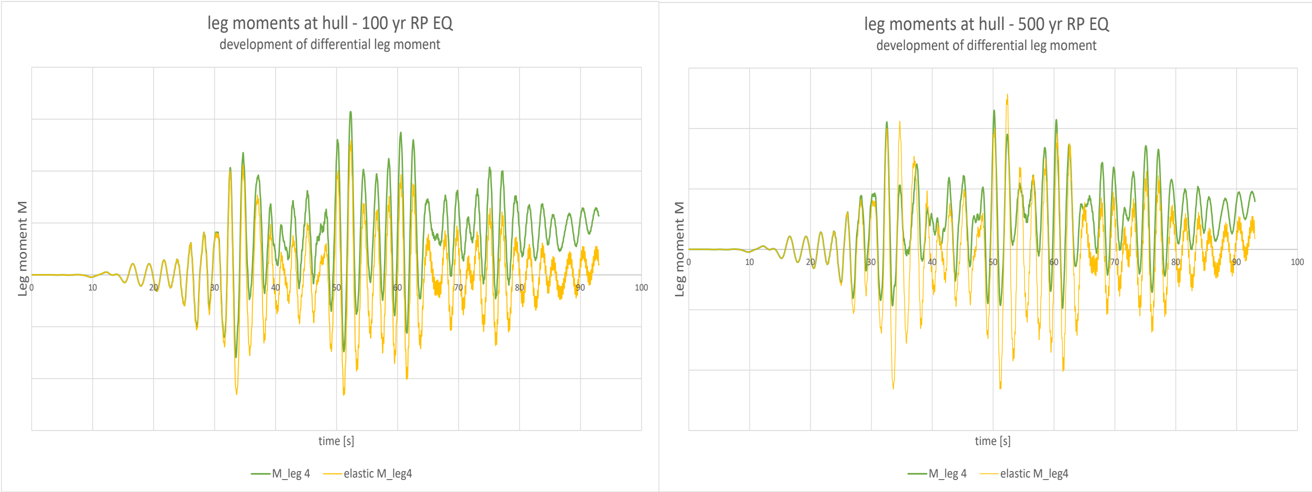

Figure 14 Comparison between resulting leg moments from the elastic analysis (yellow) versus analysis including elasto-plastic soil springs (green). For 100-yr RP EQ the elasto-plastic analysis results in an increase of peak leg moment (due to horizontal differential settlement), for 500-yr RP EQ to a reduction.

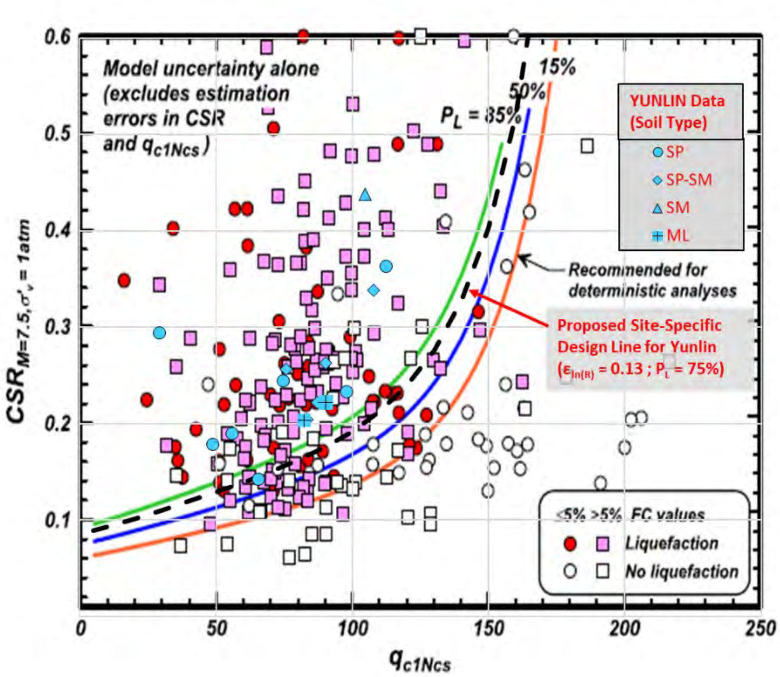

Figure 15 Proposed site-specific CRR design curve based on YUNLIN cyclic DSS tests, plotted with the general design curves from [6]

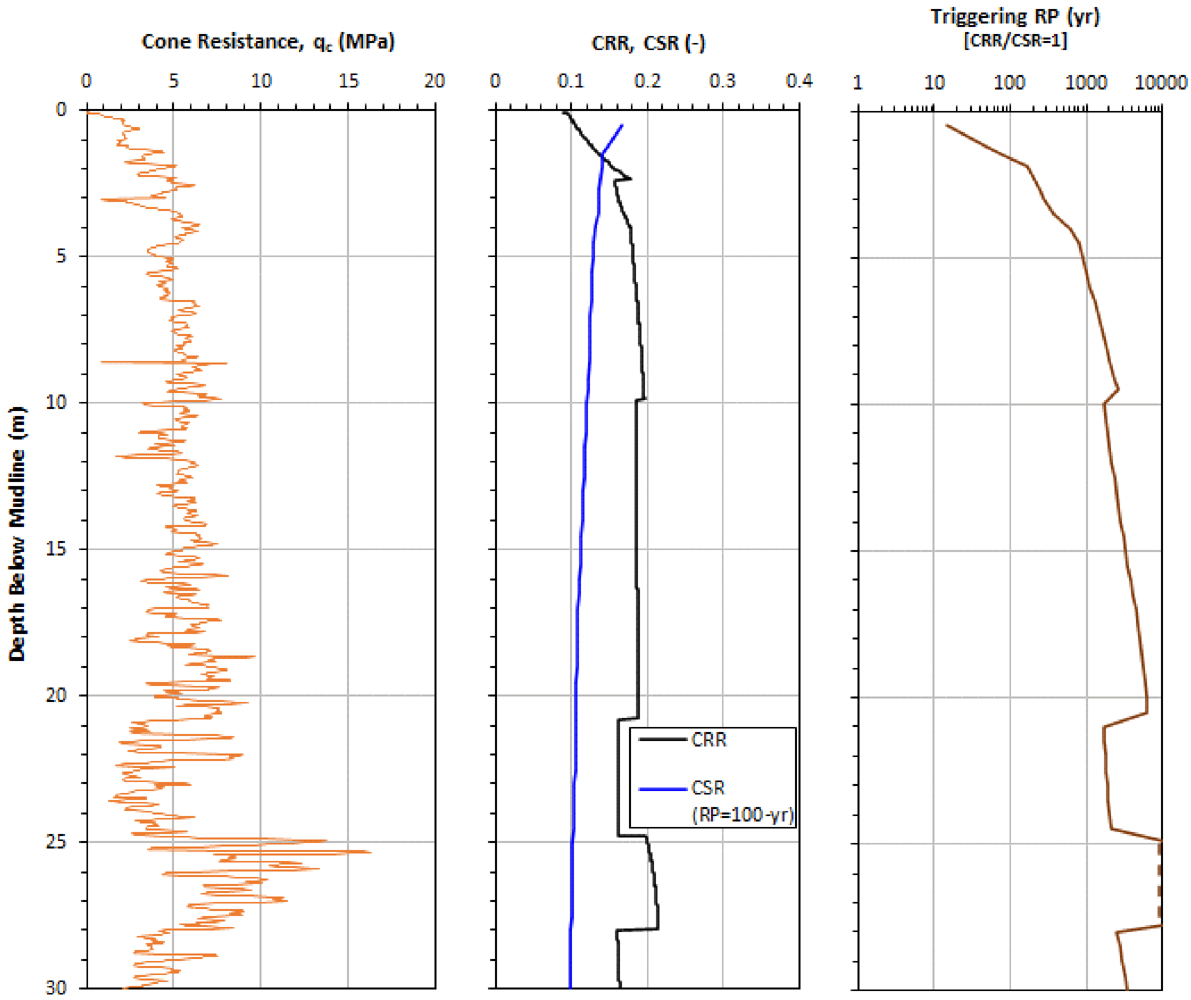

Figure 16 Example of liquefaction assessment results

TABLES

Table 1 Example of liquefaction assessment results

Group	Spudcan Penetration Depth (m)	Soils Below Spudcan
1	≤ 1	Sand
2	1 – 3	Sand
3	4 – 6	Sand
4	2 – 5	Predominantly sand with silt/clay
5	≤ 1	Sand with weak interbedded layer