

ASSESSMENT OF SEISMIC RISK FOR THE DESIGN OF OFFSHORE STRUCTURES IN LIQUEFIABLE SOIL

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ABSTRACT

The foundations of an offshore structure has to withstand a combination of complex design loads during its lifetime. These include wave loading, possibly ice loading as well as seismic loading if the platform is located in an area of moderate or greater seismic hazard. Furthermore in many cases the sea bed consists of loose silty sands and sandy silts, which could make it prone to liquefaction under sufficient seismic loads. In this paper the overall performance based methodology for designing an platform in a seismic zone will be discussed. Results from site response analyses will be presented for upper and lower bound estimates of soil properties. Comments will be made about the suitability of these correlations when in-homogenous soil is present. The use of deterministic and probabilistic methods to understand the triggering of liquefaction will be discussed. The role of fines on the liquefaction resistance of the soil will be discussed. The results of the 3D dynamic soil-structure interaction analyses for an offshore platform will be presented. It will be shown that the free field motion is different from the motion experienced under the presence of the foundation. In conclusion it will be shown that the seismic performance of the platform can be assessed and quantified if appropriate methods are used coupled with engineering judgments.

Keywords: offshore; liquefaction, dynamic soil-structure interaction; risk assessment

INTRODUCTION

As the earth's natural resources are exhausted the future of offshore hydrocarbon exploration and development lies at more marginal sites. Thus future production will require innovative sea bed design solutions to satisfy the stringent design criteria. This will result in greater challenges for the design of offshore foundations. Design of such foundations is complicated due to increased interaction between the soil the foundation and the platform superstructure. It is usually estimated that the cost of offshore foundations is generally 20-30% of the total cost of the project. In many cases traditional piled foundations can prove to be an uneconomic solution. A deep skirted shallow foundation, commonly referred to as a 'bucket' foundation is often an attractive solution.

Offshore foundations have to withstand a combination of complex design loads during their lifetime. These include wave loading, possibly ice loading as well as seismic loading if the platform is located in an area of moderate or greater seismic hazard. A broad range of geo-hazards, such as faulting, liquefaction, sea floor landslides, tsunamis, mud volcanoes etc. could potentially affect the chosen site and these need to be investigated fully. High quality geological and geotechnical interpretation is therefore required in order to identify these potential geo-hazards at the platform site and to mitigate project risks in areas such as foundation installation, foundation serviceability and stability, and

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seismic response of the platform. One of the most important geo-hazards is liquefaction, as the sea bed is often made of loose silty sands and sandy silts, which can make it prone to liquefaction under seismic loading. In such cases any liquefiable material beneath the foundation could impact on the overall dynamic stability of the system. The potential for liquefaction of seabed soils, particularly loose sands is therefore a major issue that should be addressed by the designers of offshore facilities.

In this paper the overall performance based methodology for designing an offshore platform in a seismic zone will be discussed. Results from site response analyses will be presented for various upper and lower bound estimations of the shear modulus based on different standard correlations. The results of the 3D dynamic soil-structure interaction analyses for the offshore platform carried out using the Arup in house program *Oasys LS DYNA* will be presented. It will be shown that the free field motion is different from the motion experienced under the presence of the foundation. The foundation filters the high frequency components and modifies the soil-structure interaction effects due to the flexibility of the soil. In conclusion it will be shown that the seismic performance of the platform could be assessed and quantified if appropriate methods are used coupled with engineering judgments.

SEISMIC DESIGN METHODOLOGY

The various steps needed in the seismic design of an offshore platform are shown in Figure 1. In the first stage the seismic design considerations have to be determined in accordance with ISO 19901-2 First Edition 2004: Petroleum and Natural Gas Industries – Specific Requirements for Offshore Structures – Part 2: Seismic Design Procedures and Criteria.

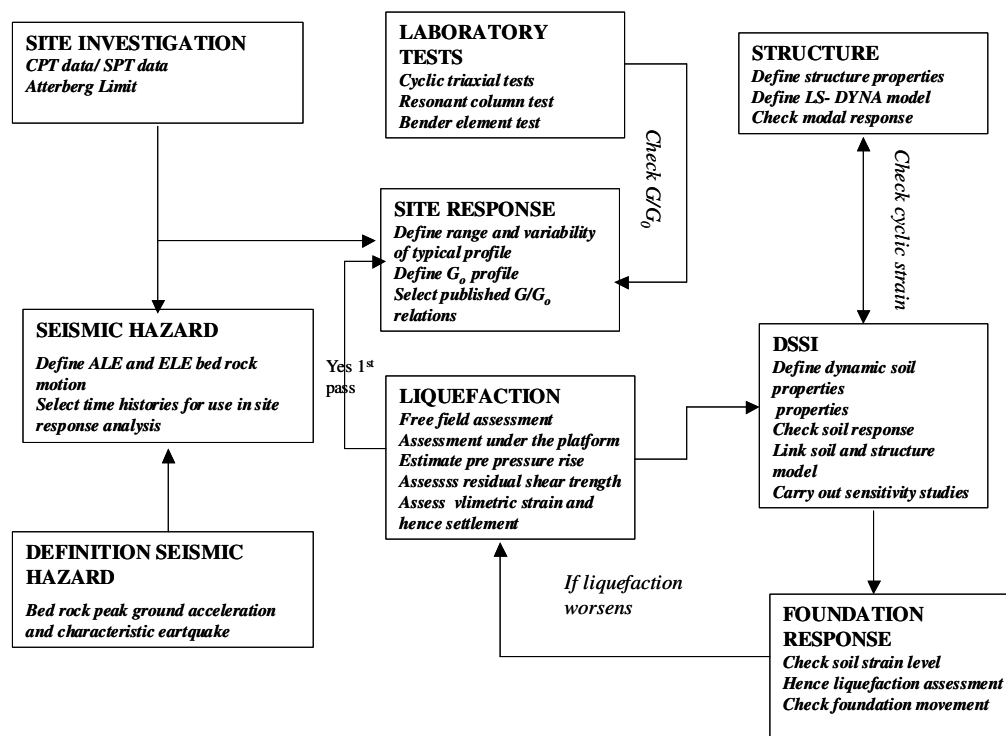


Figure 1: Overview of seismic design methodology

Two seismic design levels are usually considered:

- The Extreme Level Earthquake, ELE (formally known as the SLE)
- The Abnormal Level Earthquake, ALE (formally known as the DLE)

The ELE will have a reasonably low likelihood of exceedance during the structure life. The structure shall be designed such that it would sustain little or no damage during the ELE. The ALE is an

“intense earthquake of abnormal severity with a very low probability of occurring during the structure’s design service life”. The structure may sustain considerable damage during the ALE. However, the structure shall be designed such that structural collapse causing loss of life and/or major environmental damage is avoided, and overall structural integrity is maintained.

Thus a risk assessment for any such project should include evaluation of all geo-hazard effects. In the next section some of these risks will be evaluated for an offshore platform located for an area of moderate seismicity.

SEISMIC HAZARD

The seismic design criteria have been developed in accordance with the recommendations of ISO 19901-2. A site specific probabilistic seismic hazard assessment has to be undertaken for the site and the results should be presented in terms of uniform hazard response spectra as shown in Figure 2. Here the uniform hazard response spectra have been shown for return periods of 200, 2,000 and 20,000 yrs. The resulting ELE and ALE design spectra, which are also shown in Figure 2, can then be determined based on the site-specific hazard curves and for the target annual probability of exceedance specified in the code. This target annual probability of exceedance depends on the usage of the platform, whether the platform is manned and the seismic reserve capacity of the particular structure under consideration.

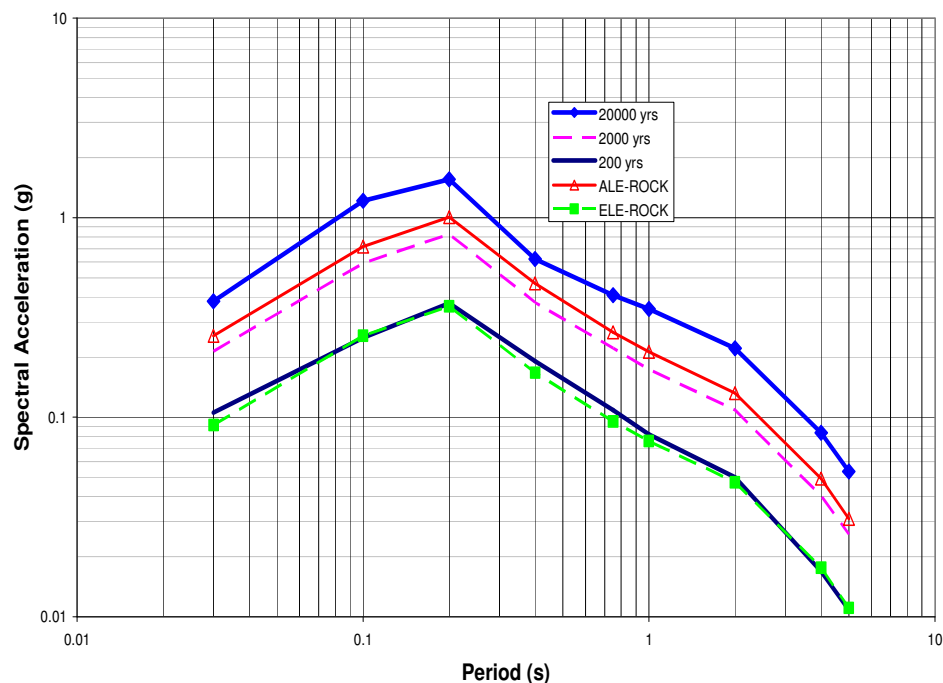


Figure 2: Example ALE and ELE rock hazard spectra

SITE RESPONSE

Seismic ground motion is altered as it passes from the underlying bedrock and through the overlying soil profile. During an earthquake, stress waves which are generated at some depth are modified as they travel through the various soil layers. The soil acts as a filter, amplifying energy at some frequencies and attenuating energy at others. The effect of this is often calculated by performing a site response analysis, preferably using non-linear programs such as *Oasys SIREN* (Pappin, 1990), especially where large strength and strain reductions can be expected in the soil.

The soil column is specified as a series of layers each with its own material properties, characterized by a stress-strain relationship and a bulk density. The program operates in the time domain enabling it to model non-linear soil properties with hysteretic damping. The following parameters are needed to evaluate the non linear site response.

- Soil unit weight (γ) –obtained from site investigation.
- Small strain shear modulus (G_0)
- Modulus degradation (G/G_0) curves as a function of shear strain
- Bed rock properties – Usually obtained from site investigation.
- Input time histories –These are usually spectrum compatible

Determination of small strain shear modulus

A geotechnical investigation is undertaken to determine the static and dynamic soil properties for the soil bed below the sea level. This usually consists of cone penetration tests and laboratory tests.

Relationships are available which allow the derivation of low strain shear modulus (G_0) from cone tip resistance (q_c), for example (Baldi et al (1989), Mayne and Rix (1993) and Rix and Stokoe (1991). Furthermore, Rix and Stokoe (1991) have also suggested that these relationships are probably a function of other factors, such as material type, median grain size etc.

Figure 3 shows the Best Estimate and Lower Bound design lines for G_0 to be used in the seismic assessment. In this example the reduction in G_0 corresponds to locations where thin silt lenses are present.

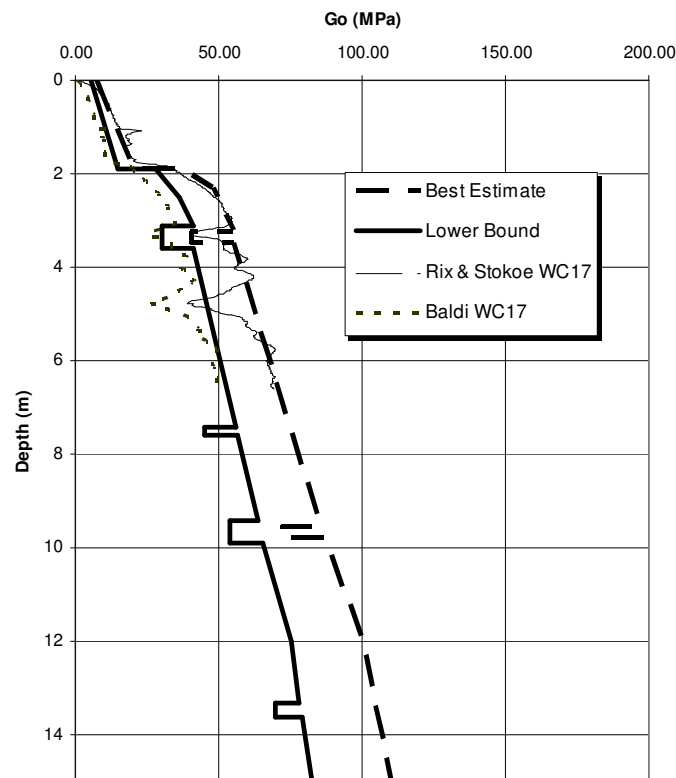


Figure 3: Example design profile adopted

LIQUEFACTION ASSESSMENT

Ideally liquefaction potential should be assessed using both the latest empirical methods and the results of good quality site-specific cyclic testing. The example below are based on the methodology proposed by Seed *et al* (2003) and Moss *et al* (2005). This methodology determines the probability of liquefaction, and the (recommended) CPT based procedure has been used. The factor of safety against liquefaction was calculated based on the Cyclic Resistance Ratio (CRR) for a 16 % probability of liquefaction. The following list describes the stages carried out in the liquefaction assessment:

- A series of site response analyses were performed to calculate the modification of peak ground acceleration (PGA) from bedrock to seabed level for both ALE and ELE events by using *Oasys* SIREN for pre selected input time histories. Distributions of maximum shear stress normalised with the effective overburden stress at that depth obtained from *Oasys* SIREN for the ALE is shown in Figure 4 for two components of input motion. These values have been compared with the shear stress ratio obtained from detailed DSSI (discussed later).

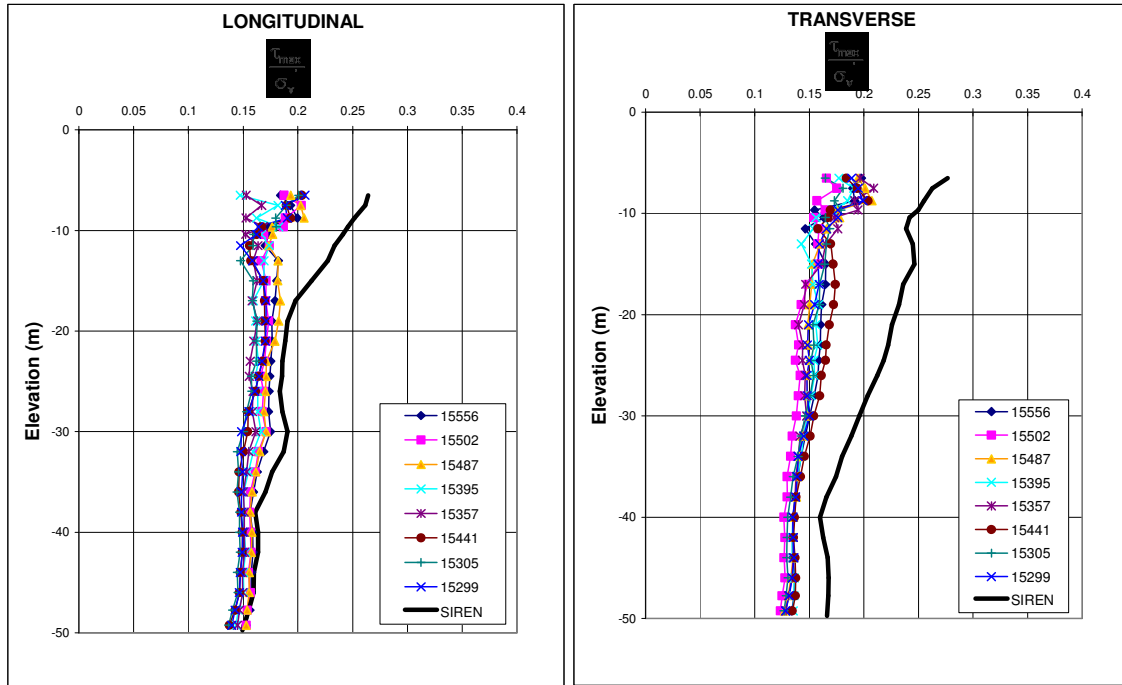


Figure 4: Maximum shear stress for ALE event obtained from SIREN

- The Cyclic Stress Ratio (CSR), which defines the shear stress applied to the soil during the earthquake, was calculated from the average shear stress generated due to the four selected earthquake time histories (obtained directly from the site response analysis). The CSR was calculated by using:

$$CSR = 0.65 \frac{\tau_{max}}{\sigma_v'}$$

The Cyclic Resistance Ratio (CRR) corresponding to a probability of liquefaction of 16% was determined by using the CPT data. This is determined as a function of the modified cone tip resistance, the friction ratio, the earthquake magnitude and the effective stress at the particular depth being considered.

- The deterministic factor of safety against liquefaction is computed as follows:

$$FS = \frac{Capacity}{Demand} = \frac{CRR_{7.5}}{CSR} DWF_M$$

The DWF_M (magnitude-correlated duration weighting factor) is intended to account for the fact that the magnitude at which the triggering curves were derived was 7.5 Mw.

- The factor of safety against liquefaction was determined and compared with the probabilistic results in order to identify layers that would liquefy under ALE and ELE events. A liquefaction probability of 16% is considered equivalent to the deterministic limit, i.e. factor of safety of 1.0 against liquefaction.

Typical results for CPT WC06 shown in Figure 5, showing probability of liquefaction for ELE and ALE. The liquefying layers are identified to be those where the factor of safety against liquefaction falls below 1.0. It is seen that layers of different thickness are shown to liquefy across all depths. The results indicate that liquefaction is unlikely to occur due to the ELE but is likely to occur in discrete layers and locations in the ALE. Thus it is possible to draw cross sections and determine the extent of liquefiable soil in the proposed site location. If the scale of liquefaction is deemed to be a problem then suitable mitigation measures should be incorporated in the design to reduce the risk to a minimum.

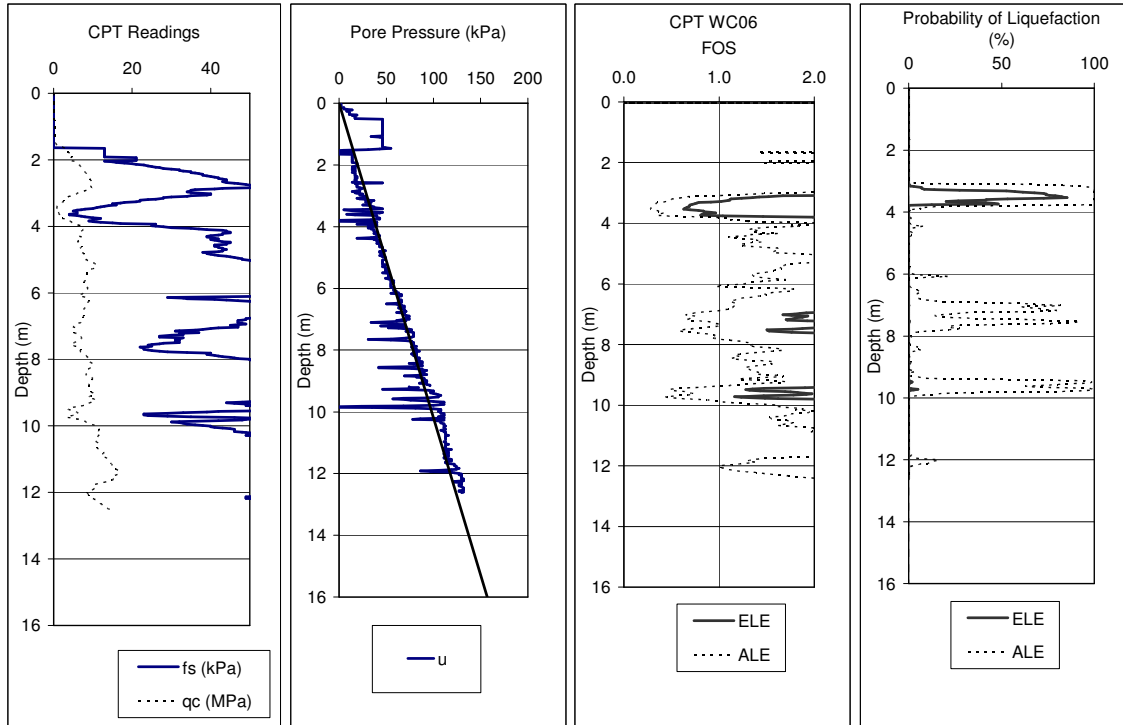


Figure 5: An example of probabilistic and deterministic assessment of liquefaction

DYNAMIC SOIL STRUCTURE INTERACTION

The next step in the design process for such offshore foundations is to evaluate the dynamic soil structure interaction effects. Generally two mechanisms of interaction take place between the structure, foundation and soil, namely inertial and kinematic interaction. In the present project dynamic soil-structure interaction (DSSI) analysis was carried out using the Arup in-house program *Oasys LS-DYNA* to obtain the following data for design:

- Seismic motion at the foundation and response spectra for use in the (separate) response spectrum analysis of the platform superstructure;

- Maximum shear stress to vertical stress ratio with depth below the foundation, to check liquefaction potential with the structure weight included (as opposed to free field liquefaction);

Oasys LS-DYNA is a non-linear explicit 3D finite element program capable of modelling highly non-linear and dynamic engineering problems. The use and verification of the soil model used for DSSI analyses is described in Lubkowski, (1996). A 3D finite element model was generated with soil and simplified representation of the platform foundation and super-structure. Non-linear time history analyses were performed by applying the ground motions as velocity time histories to the boundary of the model.

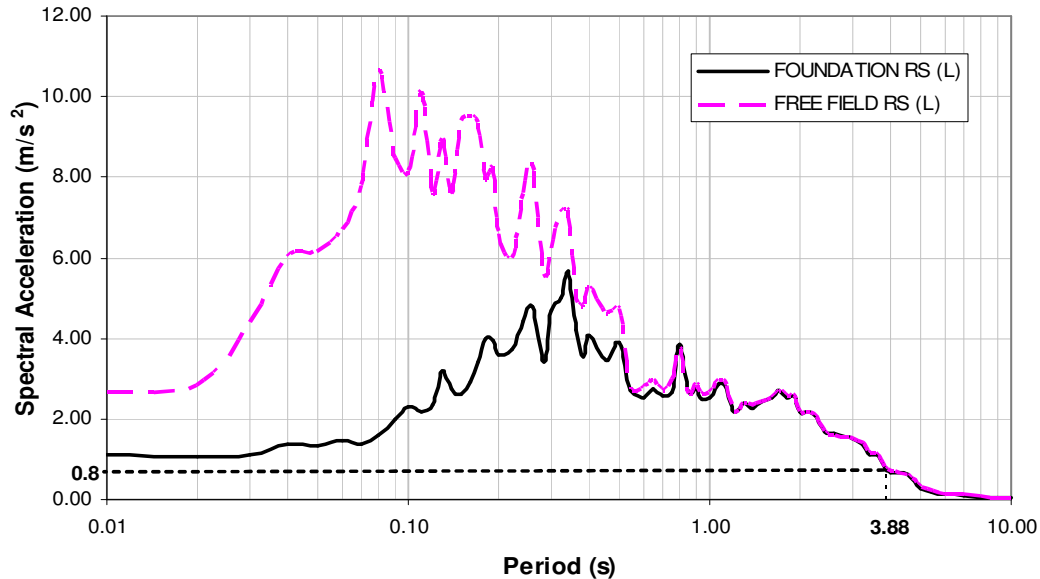


Figure 6: Comparison of free field and foundation response spectra

Four DSSI analyses were carried out for ALE seismic level and for best estimate soil properties. This was to derive the ground motion for input into structural analysis for the separate response spectrum analysis of the platform superstructure. Figures 6 compares the response spectra of the ground motion of the foundation block, as well as the free field surface for longitudinal directions for each time history. These spectrums were derived for 3% damping. A comparison of the different results shows that the response spectra are more or less identical beyond 0.7 s period. It can be seen that the foundation block filters the low period components of the free field ground motion resulting in lower spectral response in the periods below 0.7 s. This has implications on the shear response of the foundation analysed using response spectrum analysis method, where the free field ground motion derived response spectrum would give (very) conservative estimates of foundation response. This also highlights the importance of correct evaluation of DSSI effects.

The free field liquefaction assessment carried out using conventional site response analysis provides the cyclic stress ratio (CSR) for comparison with the cyclic resistance ratio (CRR) based on the soil properties. The presence of a structure results in an increase in (vertical) soil pressures above the in-situ (free field) conditions, although this effect diminishes with depth. The presence of a platform therefore has an effect on the site response characteristics below the foundation and therefore on the liquefaction resistance, which is different to that of the free field conditions. In order to assess the impact of the platform on liquefaction potential below skirt tip level, the maximum shear stress ratio (maximum of the shear stress to vertical effective stress ratio) was deduced from the four analyses shown in Figure 4. The SIREN curve represents the free field response without the effect of the structure. It is clear that the presence of the foundation significantly reduces the maximum shear stress

ratio (i.e. CSR) from that of the free field over the upper 10 m below the foundation and this has implication in the liquefaction assessment.

CONCLUSIONS

In this paper the overall performance based methodology for designing an offshore platform in a seismic zone has been discussed. Results from specific site response analyses has been presented for various upper and lower bound estimations of the shear modulus using different standard correlations. The results of the 3D dynamic soil-structure interaction analyses using *Oasys* LS-DYNA for the offshore platform have been presented. In conclusion it is shown that the seismic performance of the platform can be assessed and quantified if appropriate methods are used coupled with appropriate engineering judgment.

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