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ABSTRACT

Major earthquakes have resulted in devastating consequences in terms of human and economic loss. In almost all the earthquakes we observe the failure of structures, sometimes due to poor construction but also due to designers not identifying the specific geo-hazards (iIntensity of ground motion, faults, liquefaction, slopes etc) which affect these structures. In many cases these damages could have been avoided if the original design had correctly identified the geohazards at the site and incorporated the philosophy of performance based design. In this paper several examples will be presented where the different stages of risk assessment will be identified and possible solutions incorporated in the final design. The paper provides examples where existing studies and codes in certain countries may be storing up problems for the future.This paper also highlights some gaps in existing knowledge where more research is needed. Design examples will also cover the advantages of performing detailed design accounting for soil structure interaction effects. In many cases these will offer potential saving to the clients and thus provide value in seismic design. Examples are shown where structures which have accounted for the geohazards will be shown to perform satisfactorily during past earthquakes.

INTRODUCTION

The question “what is value is even more complex and more important than most people think. It is fundamentally a perception. But perception on its own does not have much importance if it is not backed by solid evidence. As Warren Buffet says” Price is what you pay and value is what you get”. This is even more important in current financial climate where our clients would like to have maximum value for their design. The fundamental assumption in seismic design is that the design codes provide a safe design which meets the limit states of collapse. The majority of seismic codes are written to prevent loss of life. This is achieved by designing a structure that is ductile, i.e. one that absorbs energy through cracking and plastic hinging, but without collapse. However, this means that significant damage can occur after a very large earthquake, which might make the structure un-usable. If this is unacceptable to the client, a more stringent performance level can be achieved, either by using a higher Importance factor (I) than is required by the code, or by using a performance based approach such as that given in FEMA368. However it can be argued that in certain type of structures we are not interested in collapse prevention for frequent earthquakes. This places emphasis on the idea that the fundamental corner stone of geotechnical seismic design should be the management of performance of the structure to various hazard levels. This paper will explore this idea through a typical design process and also places emphasis on what is a tolerable level of settlement for performance management? A slope may not fail if it has to sustain few mm of displacement or a foundation maybe allowed to deform. Questions like is it desirable to allow inelastic response in strong ground shaking above the foundation as repair to foundation can be extremely costly and difficult often comes across a designers mind. It is also noted that capacity design considerations for foundation design are required in Eurocode 8 and other codes such as NZS 3101. It is therefore good practice. However it is uncommon in US practice to consider "capacity design" for foundations and hence is not required in either UBC or IBC. Clearly there are cost and performance implications to this decision. Capacity design to include the overstrength factor will increase foundation costs, but ensure the foundations are serviceable following the design earthquake and ignoring capacity design may lead to plastic hinges forming in piles and hence the long term serviceability of the structure may be affected. This should not affect life safety requirements for the structure during its design life.

Methods for assessment of the seismic performance of geotechnical structures and soil-structure systems have evolved significantly over the last few years. This has partly been due to the improvement in understanding the fundamental soil behaviour and use of advanced numerical techniques to model complex soil structure interaction problems. The emphasis has also shifted from using simple soil models to complex constitutive models which can model soil behaviour under dynamic conditions In particular, the Performance Based Earthquake Engineering (PBEE) concept has emerged with the emphasis on performance based deign for geotechnical structures (ISO-23469, 2005). In broad terms, this general framework implies engineering evaluation and design of structures whose seismic performance meets the objectives of the modern society (Cubrinovski, 2009). This is a complicated task since the stress-strain behaviour of soils under earthquake loading is very complex involving effects of excess pore-water pressures and significant non linearity at high strains. The ground response usually involves other complex features such as:

- Modification of the ground motion due to site effects or local effects (earthquake excitation for engineering structures)
- Large ground deformation and excessive permanent ground displacements due to lateral spreading or soil movement.
- A significant loss of strength, instability and ground failure due to liquefaction etc.
- Soil-structure interaction effects.

In this paper attention is focused on Eurocode 8 design procedures as they become mandatory from March 2010 in Europe. Comparison will be made with other codes when deemed suitable.

DIFFICULTY IN FOLLOWING PERFORMANCE BASED DESIGN APPROACH

The fundamental assumption in applying performance based design criteria is that seismic loading is an imposed deformation and we need to quantify deformation demands for a chosen earthquake level. This is then followed by checking the imposed deformation against deformation limits at a global level as well as a local component level.

It is the intention to design a structure that will behave in certain ways when subjected to earthquake ground motion having a specific annual probability of being exceeded. For example the China code requires that there is no damage when subjected to a ground motion having an annual probability of 1 in 50 (= return period of 50 years OR a 63% chance of being exceeded in 50 years). It should also be demonstrated that under an extreme ground motion (annual probability of being exceeded of 1 in 2,500 or a return period of 2,500 years or about a 2% chance of being exceeded in 50 years) the structure will not collapse. Recent US rules have the same requirement for the extreme ground motion and also a requirement that there will not be a life safety issue with a 475 year return period (10% chance of being exceeded in 50 years). In addition to these requirements the client can impose more stringent requirements (on serviceability in significant ground motions, for example for manufacturing industries or Nuclear industry with expensive down time).

Most codes do not specify the deformation limit. The ISO 19901-2:2004 is quoted in Section 6.2.2 as: "During the ELE (Serviceability criteria) event, structural members and foundation components are permitted to sustain localised and limited non-linear behaviour (e.g. yielding in steel, tensile cracking in concrete"

Section 9.1 as: "The objectives of ELE design are to ensure that there is little or no damage to the structure during the ELE event and that there is an adequate margin of safety against major failures during larger events." A designer needs to interpret these performance criteria to acceptable deformation levels. An example performance criterion is presented in Table 1 for an offshore platform. This is developed in consultation with the client.

Table 1: Interpretation of performance criteria for ELE

Members	Performance Criteria
Primary structural members	elastic
Dynamic sliding at foundation level	$\pm 200mm$
Permanent slip at foundation level	$\pm 100mm$

For the collapse criteria (ALE) ISO 19901-2:2004 is quoted in section 6.2.3 as: ".e.g. structural elements are allowed to behave plastically, foundation piles are allowed to reach axial capacity or develop plastic behaviour, and skirt foundations are allowed to slide." and Section 9.2 as: "The objective of an ALE design check is to ensure that the global failure modes which can lead to high consequences such as loss of life or major environmental damage will be avoided." This can be interpreted as shown in Table 2.

Table 2: Interpretation of performance criteria for ALE

Members	Performance Criteria
Primary structural members	May exhibit plastic degradation but must maintain a sufficient level of reserve strength to prevent catastrophic failures
Dynamic sliding at foundation level	is allowed to slide
Permanent slip at foundation level	is allowed

What is not so clear from the definition is the margin of safety that is required for these cases (Pappin 2009). For the no collapse rule it is usually interpreted as the building being able to be shown to be able to survive by calculation. As it is difficult to actually predict collapse, but rather we can show non-collapse with a margin that is not well quantified, it is clear that we are showing no collapse but we do not really know by how much. This is similar to the nuclear industry where we must show no loss of containment of the main vessels when subjected to the 1 in 10,000 year ground motion. Generally we do not go on to show when loss of containment will occur. Full Quantified Risk Assessments do need to consider these scenarios but this is beyond conventional Performance Based Design approach.

TYPICAL DESIGN METHODOLOGY

The various steps needed in the seismic design are shown in Figure 1. In the first stage the seismic design considerations have to be determined in accordance with the design codes being followed for the project. In this stage of the project all the possible geohazards need to be identified. This will include identification of possible faults, fault rupture, liquefaction, tsunami, landslides and all other hazards at the site. The desk study is a key element at this stage of the project and should be routinely undertaken for any project which is likely to have disastrous impact if it fails.

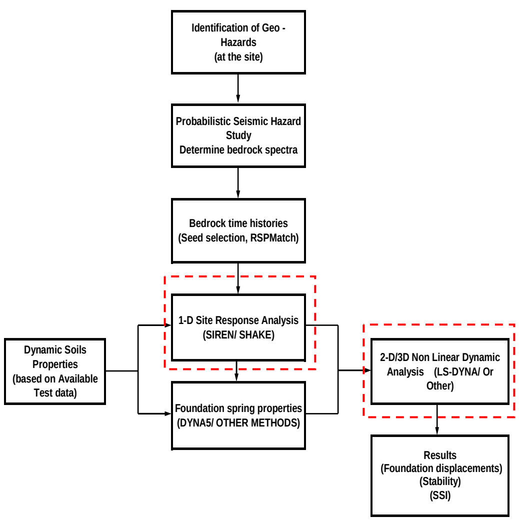

Fig.1: Typical Design methodology followed for a project.

EUROCODE 8 Vs SITE SPECIFIC SPECTRA

The definition of design ground acceleration on Type A (EN 1998:5-2004(E) ground having Vs, $_{30}$ greater than 800m/s is important in deriving the surface acceleration spectra. This is often based on the National Annexe or USGS hazard maps which define the PGA and spectral coordinates at short and long periods. This is then multiplied by selecting suitable S values (soil factors) to derive surface spectra.

Eurocode 8 allows the development of site specific spectra but cannot use the spectra if it falls below the code specified spectra as detailed in Section 10.6 (Part 1, EN-1998-1:2003). This clause can sometimes make it difficult to utilize the benefits obtained by deriving site specific spectra by performing a probabilistic seismic hazard assessment and site response analysis. We will illustrate this aspect with an example provided below.

There are two basic methods for assessing the seismic ground motion hazard in a particular region or at a specific site, namely deterministic methods and probabilistic methods. A full description of these methods is given in Reiter (1990). The methodology of probabilistic hazard assessment is very robust and adds value to the client by reducing the uncertainties associated with the definition of the hazard and can help in managing performance at different return periods in accordance with Table 1 and Table 2.

Figure 2 compares the Unified Hazard Response Spectra (UHRS, bedrock) obtained for moderately seismic sites in North Africa by PSHA analysis. The UHRS spectra are compared with the Eurocode Type 2 specified spectra for Ground Type A. This spectra can be used if the earthquake that contribute to the seismic hazard defined for the site for the purpose of probabilistic hazard assessment has a surface wave magnitude (Ms) greater than 5.5. It can be seen that the code specified spectra is often conservative especially for long periods and more so for longer return periods. This issue becomes significant in highly seismic areas where the cost of effective seismic protection can influence the project design basis. Thus it is important to offer a site specific probabilistic seismic hazard assessment as a value enhancing tool for the client for projects which are above a minimum threshold.

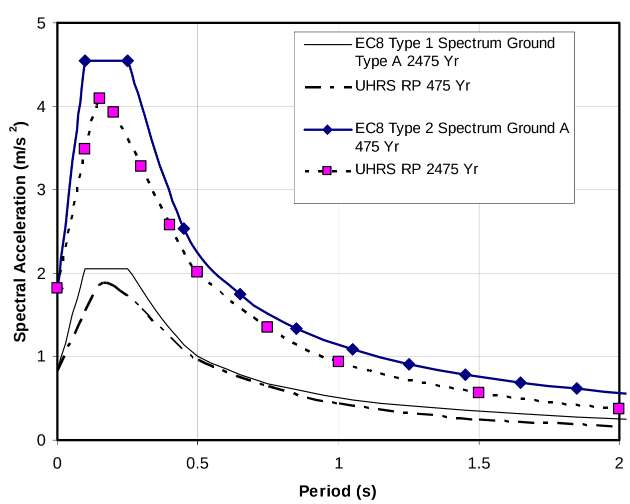

Fig.2: Comparison of bedrock spectra with PSHA analysis results

The surface spectra can be obtained by following several routes- either by multiplying the bedrock spectra with the Soil factors specified in the code. In Eurocode 8 the same S factor is used for all periods whereas in IBC 2003, 2006 specifies that different S factors can be used for long and short periods.

In the second case site specific spectra can be determined by performing site response analysis. In the next section an example will clarify this aspect.

SITE RESPONSE ANALYSIS – EXAMPLE OF VALUE

The key element of site response analysis is to capture the attenuation or amplification of vertically propagating shear waves as they pass through an idealized soil column. The elements needed for such an analysis include the dynamic soil properties, Modulus degradation curves, soil density, and earthquake time history which suit the tectonic regime of the site.

The idealised soil profile can be determined based on the site investigation data and geological history. A summary of the assumed soil profile is included in Table 3. It can be seen that a considerable thickness of soil overlies rockhead. Dynamic properties of the different layers were determined from the SPT data using various correlations (ex Wong & Pun, 1996). Standard degradation curves (EPRI 1993) were used to model the soil degradation due to seismic shaking.

Table 3: Assumed soil profile

Soil Profile	φ'	s_u (kPa)
Hydraulic Fill 0-8m(medium)	33°	
Hydraulic Fill (loose), 8-18m	28°	
Peat and Organic Clay, 18-22m	-	17
Sand (loose to medium), 22-72m	28°	
Glacial Sand (medium)	35°	

Selection of earthquake time histories

An in house Arup program (Grant et al. 2008) which selects the seed ground motion based on spectral matching to minimize the artificial manipulations to the seed records from the PEER database can be used for initial selection of the records. The scaling of the time histories to match the target design hazard spectra (hereafter referred to as the target spectrum) can be carried out using the software *RSPMatch2005*. This program performs a time domain modification of an acceleration time history to make it compatible with a user specified target spectrum. The methodology is based on that proposed by Lilhanand and Tseng (1987). The modification of the time history can be performed with a variety of different modification models (wavelets). The ease at which the program matches the target spectrum depends on the specific nature of the input time history, typically the initial frequency content and duration. This variability and effort required is reduced through careful seed selection procedure and initial scaling of the records, particularly over the long period. Figure 3 shows the time histories matched to the target bedrock spectrum. The target bedrock spectrum was obtained by matching the NYBD (New York Building Code) defined spectra to Type A ground.

Table 4 shows the time histories selected for the analysis. It has been shown by Ghosh & Bhattarcharya (2008) that when spectral matching is performed tectonic origin of the time histories are not so important. It has been shown that the use of spectrum–compatible ground motion provides least variations in the response parameters. However it should also be kept in mind that these spectrum compatible motions can induce additional displacements and the records need to be baseline corrected before they are used for further analysis. The best results – in terms of successful and timely convergence to a solution, and minimal adjustment of the seed record – are obtained when the initial seed has a response spectrum which provides a good initial fit of the target spectrum.

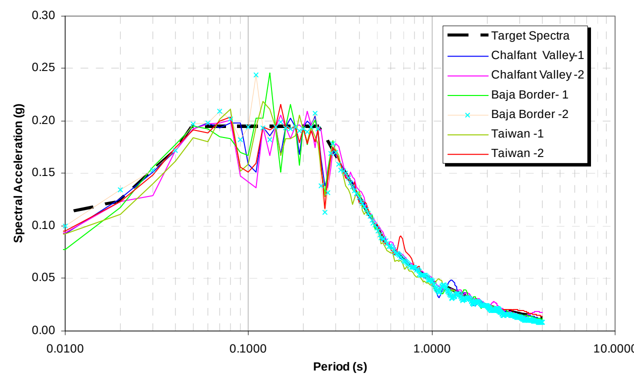

Fig 3: Selected time histories matched to target spectra.

Table 4. Selected earthquake time histories

Earthquake Name	Date	Mw
Chalfant Valley-02	1986-07-21	6.2
Baja California	1987-02-07	5.3
Chi-Chi, Taiwan	1999-09-25	6.3

Oasys SIREN (Henderson et al. 1990) was used for computing the site response analysis for the particular site. It is a non-linear time domain program. Previous studies by Henderson et al. (1990) and Heiderbrecht et al. (1990) indicate that *Oasys* SIREN gives similar results to those calculated by Shake (Schnabel et al. (1972)) for moderate levels of ground motion. However, at higher levels of ground motion, *Oasys* SIREN gives lower amplification due to the fact that it models the non-linear hysteretic behavior of the soil. Figure 4 shows the spectral acceleration at the surface for different type histories as well as IBC 2006 defined spectra for Site Class D and Site Class E. It can be seen that at low periods the code spectra is very conservative. Based on this analysis a design spectrum has been suggested which will reduce the base shear force by

30% for low period structures. This example demonstrates that there is value in performing site response analysis.

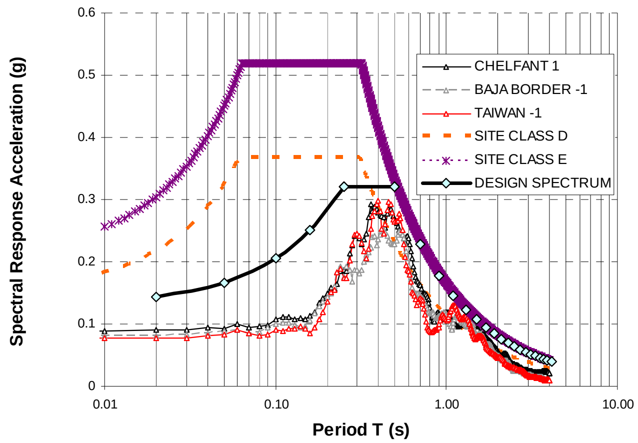

Fig. 4: Comparison of surface spectral acceleration

LIQUEFACTION EVALUATION IN MARGINAL SOILS

There is still no clear unified definition adopted for the term "liquefaction" due to its use by many researchers to describe different phenomena associated with pore pressure generation. Excellent summaries related to liquefaction analysis can be seen in various publications (Seed et al. 2003) and is not repeated here. The EERI monograph 'Soil Liquefaction during earthquakes" by Idriss & Boulanger (2008) provides a comprehensive summary of the State of the Art Knowledge.

There are two commonly used methods of assessing the likelihood of liquefaction occurring at a site;

- Standard Penetration Test (SPT) based methods
- Cone Penetration Test (CPT) based methods

The above methods utilize the soil properties, the geologic condition, elevation and the information about the earthquake shaking such as earthquake magnitude and site response to determine the likelihood of liquefaction. Liquefaction likelihood can be assessed deterministically, i.e. as a factor of safety of liquefaction or probabilistically where a probability of liquefaction occurring is calculated.

EN1998-5 Clause 4.1.4 provides guidance on assessing liquefaction potential. This states that liquefaction hazard may be neglected when the seismic action on the surface is less than 0.15g (1.5m/s^2) and either the soil is sufficiently dense (SPT $N_{1(60)} > 30$) or it has a clay content greater than 20% and plasticity index greater than 10%. However the triggering of liquefaction in marginal soil is still very difficult subject to tackle for most design engineers and will be discussed in this section through an example.

The site considered is characterized by an upper stratum of Made Ground of 6-9m thickness underlain by sands, gravels and cobbles interlayered with silty sandy clays, soft organic clays, loose silts and soft marine/lacustrine clays. Underlying the sands/gravels/clays is bedrock of interbedded limestone or mudstone or sandstone. The bedrock consists of variable strength rock with a weathered, fractured or shattered profile. Table 5 shows the ground profile based on SI investigation.

Table 5. Idealised soil profile for liquefaction analysis

Layer	Description	Thickness – range/average
Made Ground	Variable composition: limestone debris,	Variable thickness, but usually 2-3m
Superficial Deposits	Sands, sands & gravels & cobbles interlayered with relatively thick silty layers	2 to 25m / 7 to 15m
Flysch	Eocene Flysch: interbedded limestone, sandstone, mudstone, weathered, fractured and folded	Depth to bedrock – range/average 2 to 25m below ground level (bgl) / 10 to 20m bgl

There is between 8 and 16m of fill, silty sandy clay and gravel overlying bedrock in most of the site. The site is highly seismic and has a peak ground acceleration of 0.3g. The ground type according to Table 3.1 of EN1998-1 is interpreted to be Ground Type E. The PGA used for liquefaction analysis is thus 0.42g. The water table is between 0.5 to 2mbgl. The first step in deciding the potential for triggering of soil liquefaction is the determination of whether soils of potentially liquefiable nature are present in the site or not. Traditionally, clean sandy soils with low fines content have been most susceptible for seismically induced liquefaction.

Figure 5 shows the average particle size distribution curves for the sand samples recovered during site investigation compared with the criterion for liquefaction susceptibility developed by Tsuchida (1970). Figure 6 shows the Atterberg Limit data from boreholes plotted on a plasticity chart which incorporates the recommendations regarding the assessment of liquefiable soil types by Seed et al. (2003). This plot is based on experimental data and review of liquefaction field case histories, which show that low plasticity and non plastic silts may be liquefiable as they can not only cyclically liquefy, but they can also hold their water well and dissipate the excess pore pressures slowly due to their low permeability. The following zones are identified:
- Zone A soils are considered potentially susceptible to "classic cyclically induced liquefaction" if the water content w is greater than 80% of the Liquid Limit (LL);
- Zone B soils are considered potentially liquefiable with detailed laboratory testing recommended if w is greater than 85% of the LL;
- Zone C soils (outside Zones A and B) are considered generally not susceptible to classic cyclic liquefaction, although they should be checked for potential sensitivity.

Although a large proportion of the data was found to lie within

Zones A and B, only the data which verified the water content criterion (either w>0.8LL or w>0.85LL) was plotted. It can be seen that according to this method, some layers are potentially liquefiable.

Figure 7 plots the ratio of natural water content to that of the Plasticity Index based on the recommendation of Bray & Sanchio (2006). According to their database, the w_c /LL criterion for determining liquefaction susceptibility of fine-grained soils appears to be robust. For similar fine-grained soils at similar confining stresses, a lower w_c /LL ratio is representative of higher OCR and higher undrained shear strength. It is unlikely that plastic fine-grained soils with w_c/LL<0.8 are susceptible to liquefaction, and those with high w_c /LL ratios are prime candidates for liquefaction, especially if the soil is of low plasticity. In general it can be seen that fairly similar conclusion can be reached following either of the recommendations. In such cases other factors such as soil mineralogy, void ratio, overconsolidation ratio, age, etc. are also contributing factors to liquefaction susceptibility which need to be considered as well. We could provide cost savings in the project by asking for these advanced tests.

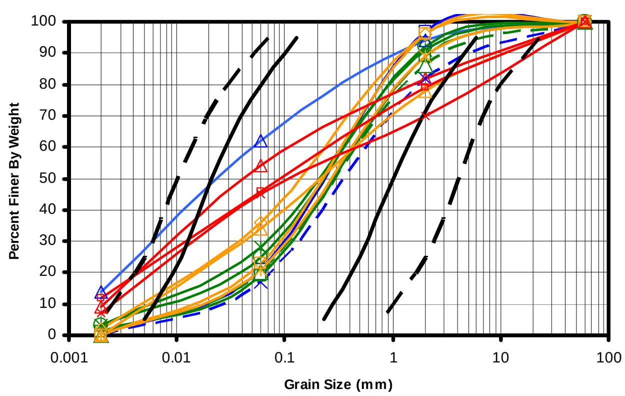

Fig 5:. PSD graphs for the soil samples compared with the limits of liquefiable soil.

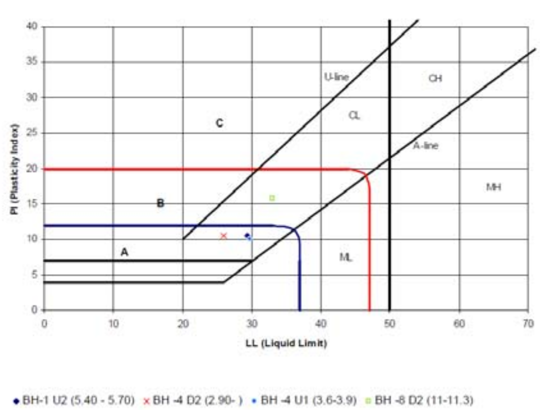

Fig. 6: Liquefaction susceptibility criteria based on Seed et al (2003) recommendations.

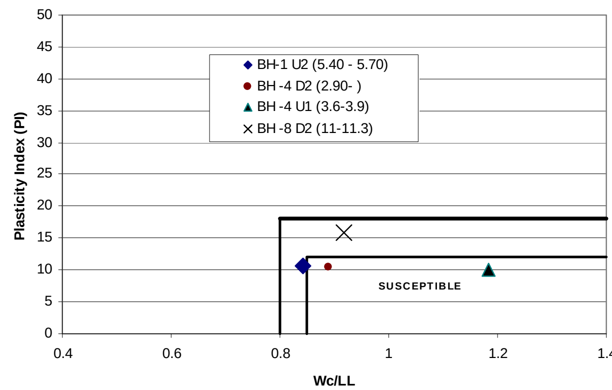

Fig. 7: Liquefaction susceptibility criteria based on Bray & Sanchio (2006).

<u>SPT based Liquefaction Assessment-</u>
This was carried out using the methodology proposed by Seed *et al.* (2003) based on the SPT data. This will be compared against the method proposed in Eurocode 8 especially for the cyclic resistance curves. The observed SPT N-values were corrected for overburden and instrument characteristics using various correction factors recommended by Seed *et al.* (2003):

$(N_1)_{60} = C_n N_{60}$

where Cn is a correction factor for overburden pressure, limited to 1.6 following Cetin et al. (2004), and

$N_{60} = C_e C_b C_r C_S N_{obs}$

where C_e, C_b, C_r and C_S are instrument-specific correction factors, and N_{obs} is the observed SPT blowcount. The following values were adopted: C_e = 1.0 (efficiency ratio correction), C_b = 1.05 (borehole diameter correction), C_s = 1.0 (sampler lining correction), C_r = 0.75 to 1.0 (depth-dependent rod length correction). Based on the SPT value, the average shear wave velocity for the top 12m was taken as 140m/s. This was used for calculating the value of r_d which is the non linear shear mass participation factor.

Further assumptions made in the Liquefaction Assessment

- Based on the ground investigations, a bulk unit weight of 20kN/m³ was assumed at all depths;
- For liquefaction assessment, magnitudes M_w 6.5, M_w 7.1 and M_w7.5 earthquake have been assumed;
- The upper few metres show gravel content in different proportions. It has been shown by Evans & Shenping (1995) that liquefaction resistance of sand-gravel composites may increase significantly with increasing gravel content and generally 40% gravel content is assumed to be the upper limit where liquefaction is possible. This criterion has been used to distinguish layers which have gravel/ sand composite and judge their liquefaction susceptibility.

Figure 8 shows the fines content from different boreholes. It can be seen that the fines content vary significantly across the site.

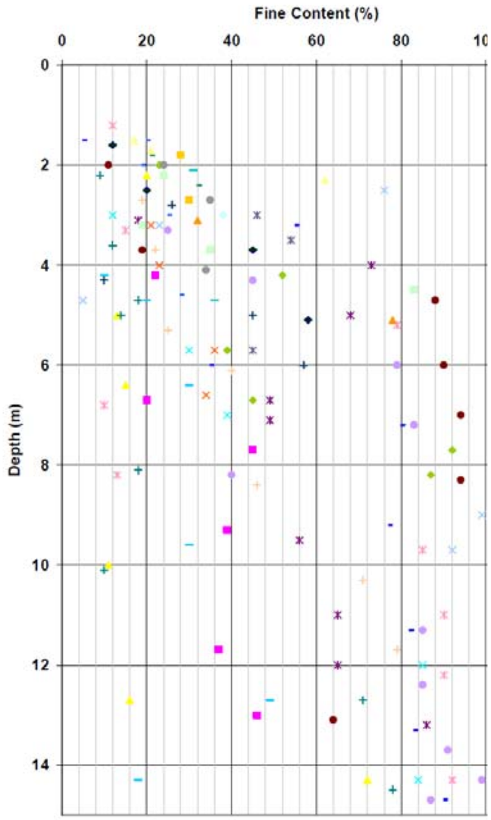

Fig. 8: Variation of fine content across the site

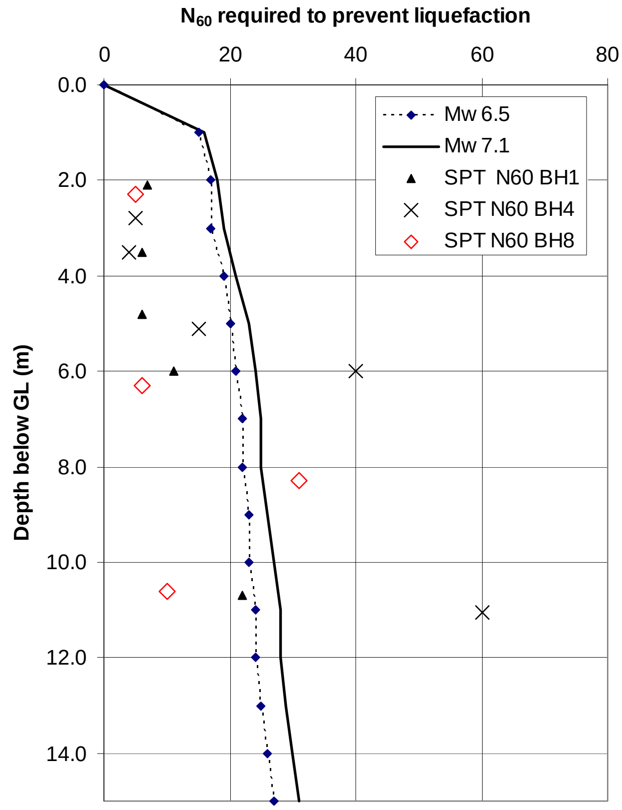

Fig. 9: Evaluation of Liquefaction potential following Seed et al.(2003).

Figure 9 shows the minimum target SPT values required to prevent liquefaction for the borehole and compares them to the corrected SPT N values. The corrected SPT points that fall to the right of the lines will not liquefy. Those that fall to the left will liquefy. It can be seen that between 2-6m there is some evidence of potential liquefaction for magnitude 6.5 earthquake. However it is often proposed that in detailed stage liquefaction potential should be determined using cyclic triaxial tests. These test results are often non conclusive in marginal soils –carbonate sand and need very careful interpretation.

Perhaps the important question to be asked is the likely consequences of liquefaction on the structure. Liquefaction-induced settlement could result in collapse or partial collapse of a structure, especially if there is significant differential settlement between adjacent structural elements. Liquefaction mitigation and performance criteria vary according to the acceptable level of risk for each structure type and human occupation considerations. Mitigation measures should be designed to either eliminate all liquefaction potential or to allow partial improvement of the soils provided the structure in question is designed to accommodate the resulting liquefaction-induced vertical and horizontal deformations following performance based design criteria.

In Figure 10 we compare the triggering curves recommend in Eurocode 8 with the triggering curves recommended by Seed et al. (2003). In EN 1998-5:2004 the SPT values are normalised to a reference overburden pressure of 100 kPa. For depths of less than 3m, the SPT values are reduced by 25% and the liquefaction potential may not be assessed for depths larger than 20m. The graph of EN 1998-5:2004 Annex B Figure B1 between stress ratios (CSR) causing liquefaction and $N_{1,60}$ values for clean and silty sands is used. It can be seen that for same magnitude and same cyclic stress ratio (CSR) Eurocode curves are unconservative. If we have a CSR of 0.2 and Fines content of 15%, the minimum SPT required to prevent the occurrence of liquefaction is 21 following Seed et al .2003. If we follow the Eurocode we need a minimum $N_{1(60)}$ of 14. This difference is reduced as the fines content is decreased. Possible reason for the differences can be attributed to the fact that the Eurocode curves are based on the curves recommended by Seed et al. (1985) and these curves have been adjusted following new case histories and recent research. It is recommended that the Eurocode curves are also duly updated.

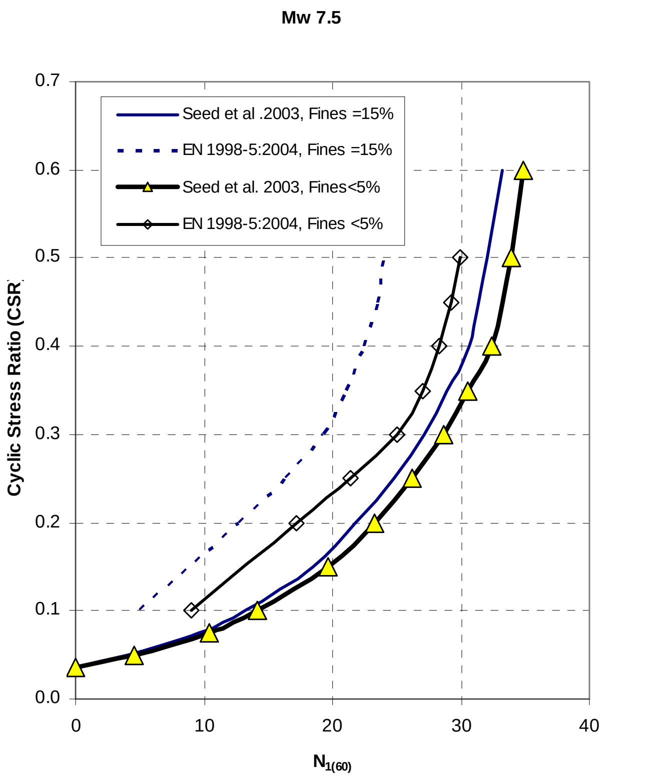

Fig. 10: Comparison of liquefaction triggering curves

It is known that stress conditions change completely in the vicinity of the structure and the self weight of the structure can be sometimes beneficial in reducing the liquefaction susceptibility. The high static shear stress underneath the foundation due to the structure weight inhibits the rise of the excess pore pressures to the free field levels. This creates transient flow conditions which prevent the dissipation of the excess pore pressures. This will be the intermediate condition until eventually there is liquefaction under the building which will lead to its instability.

Figure 11 shows the variation of Cyclic Stress Ratio (CSR) with depth for an offshore foundation and compares it with the free field value. The foundation was modeled by using *Oasys* LS-DYNA which is a non-linear explicit 3D finite element program capable of modelling highly non-linear and dynamic engineering problems. The skirt foundation was modelled as a rigid block of 8-noded solid elements. Non linear time history analysis was performed and it can be seen that the Cyclic Stress Ratio in presence of foundation is reduced and it can be beneficial to use this value for liquefaction assessment especially in marginal cases where the cost of remediation can be expensive.

In conclusion we can say that the complexity of liquefaction phenomena dictates that engineering judgment will always play a significant role in practice. Liquefaction in marginal soils with low cyclic stress ratio is always difficult to predict and we need to emphasize to the clients that 'value can be added to the analysis by using increasing level of sophistication which would contribute to the final decision making process.

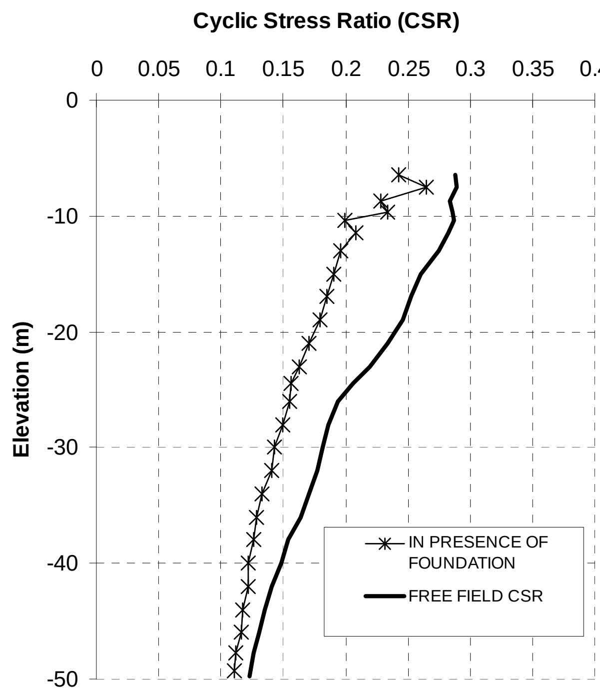

Fig. 11: Comparison of CSR for free field & underneath the structure

DYNAMIC SOIL STRUCTURE INTERACTION

The next step in the design process 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 both approaches, the mathematical complexity is enough to persuade most design engineers to ignore these effects. The motion recorded at the base of a structure or in the immediate vicinity is different from that which would have been recorded in the absence of the structure. However, very few instrumented cases studies exist which quantify this difference; numerical modifications are usually adhered to account for such effects. In most cases the free field surface level ground motion is selected as the control motion used at the foundation level neglecting the kinematic interaction effects. This assumption is based on the perceived beneficial role of SSI in reducing the seismic response. But there are numerous documented case histories where ignoring SSI has lead to oversimplification in the design leading to an unsafe design in foundation and superstructure (Gazetas et al. (1998)).

The impedance method is the more popular and in this method the founding media is represented by homogeneous, isotropic, linear elastic or visco - elastic half space extending to an infinite depth. Most of the foundation impedances have been derived assuming homogeneous half space conditions for the soil, which overpredict the damping for structures on actual soil profiles. The majority of the available solutions are for uniform deposits despite the fact that soil deposits are seldom uniform! Recorded strong motion in structures indicates that destructive shaking is often accompanied by non- linear response of the foundation soil (Luco et al., 1980, Trifunac et al., 2001 a,b). In such a case the validity of the impedance approach is questionable. Interactions are greatest for rigid structures resting on soft soil, which in some cases may be liquefiable. In this section we will present some examples where we have added increased value in our design by incorporating soil structure interaction effects in our design.

In order to investigate these effects Dynamic soil-structure interaction (DSSI) analysis was carried out using the Arup in-house program *Oasys* LS-DYNA to model the interaction of the foundation with the ground during strong seismic shaking. The purpose of the analysis was to derive foundation level spectra which could be used to calculate the forces in the superstructure. These spectra could be used for response spectrum analysis of the superstructure to predict drifts and displacements. The modified version of *Oasys* LS-DYNA, known as Ceap (Civil Engineering Application), was used in the dynamic soil-structure interaction (DSSI) analyses of the platform structure. The use and verification of the soil model 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.

Four DSSI analyses were carried out for ALE (Abnormal Level earthquake) 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 12 compares the response spectra of the ground motion of the foundation block, as well as the free field surface for longitudinal (X) directions for each time history. These spectra were derived for 3% damping.

A comparison of the different results shows that the response spectra are more or less identical beyond 0.7s 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 analyzed 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.

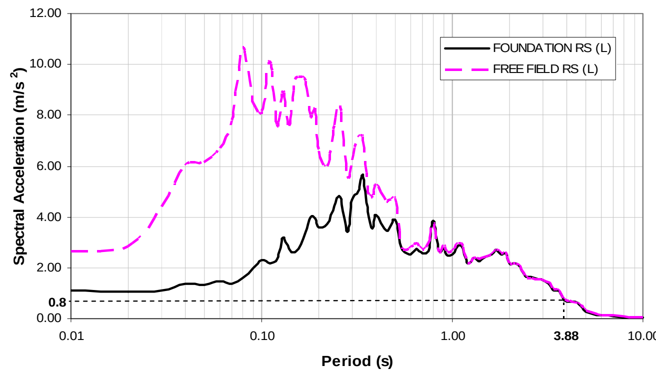

Fig. 12: Comparison of Free Field and Foundation Level spectra

Lubkowski et al. (2000) presented the analyses and results for the dynamic soil-structure interaction assessment of an ethylene tank, in the Philippines. The initial design had been carried out by two independent organizations, one designing the pre-stressed concrete piles the other the steel tank structure. The purpose of the analysis was to determine the effects of kinematic interaction on the piled foundations and/or potential cost savings for future projects, by considering the entire soil-structure system in a single analysis. Two levels of earthquake were considered, the Operating Basis Earthquake with a return period of 500 years and the Safe Shutdown Earthquake with a return period of 5000 years. Analyses were carried out for both upper and lower bound soil conditions for OBE and SSE motions. Figure 13 shows a typical bending moment envelope and the instantaneous bending moment diagram at 5 seconds for an actual pile.

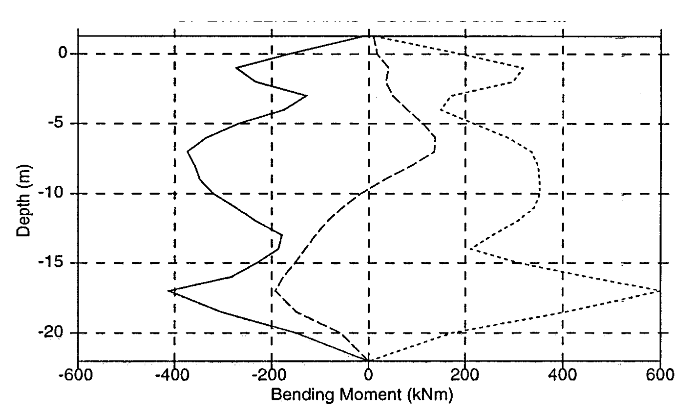

Fig. 13: Pile Bending Moments, Lower Bound Soil, SSE Input

The results indicate that under SSE loading and assuming lower bound soil conditions the ultimate bending moment capacity of the pile (modelled as elastic uncracked) is exceeded. The maximum bending moment in this case is found about 17m below ground level, at the change in soil from clayey silt to sandy silt. The assessment showed that the piles would behave in a plastic manner under the service level earthquake (OBE). The analyses also showed that the tank would remain elastic in the ultimate level earthquake (SSE), which suggests that significant cost savings could have been made in the original design of the steel tank. This example illustrates the point that advanced analysis can be very useful in reducing costs in some projects.

ADVANCED ANAYSIS SNAPSHOTS

In addition to this type of analysis there are certain design situations where we can offer value by performing advanced analysis. An example is presented below where excess pore water pressure was predicted for vibrations induced by wind turbines. The turbine had a monopile foundation and was installed in clayey soil having silty sandy bands.

The initial study was undertaken to develop a methodology for estimating the likely magnitude of pore pressures generated by turbine induced vibration and cyclic wave loading. This was done by first estimating the likely range of cyclic shear stresses and strains using dynamic soil structure interaction analyses, and then using the results of this analysis to assess whether the predicted soil cyclic shear stress/strain will be sufficient to cause permanent excess pore water pressures. The resulting pore pressure rise in the foundation soil will be in general a function of

- Level of shear strain generated in the soil
- Number of loading cycles
- Amplitude of vibration
- Initial density of the soil (Initial stress state of the soil)
- Soil type, drainage conditions

The load data is in the form of loading time history at mudline level for a period of 10 minutes. Two events are for normal operational conditions (wide banded excitation) and the third set of data is for idling conditions (narrow banded excitation) during an extreme event and includes data associated with a 50 year return period extreme wave as shown in Figure 14.

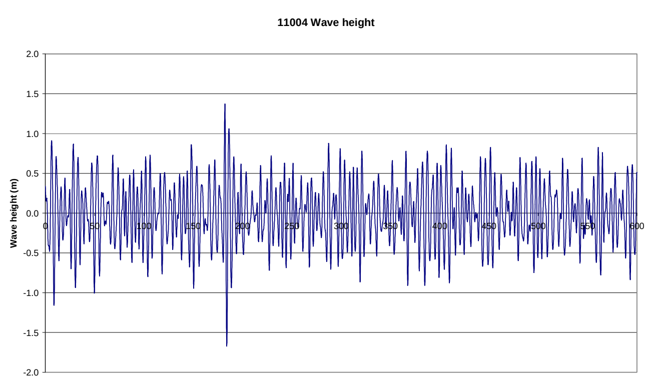

Fig. 14. Typical Loading condition for the foundation

To estimate excess pore pressures around the pile generated by cyclic loading requires detailed knowledge of cyclic shear stresses and shear strains within the supporting soil. Once cyclic shear stresses and/or cyclic shear strains have been determined laboratory test data can be used to estimate the likely magnitude of pore water pressures resulting from the these shear stresses/strains. The difficulty is finding a method that will provide reliable estimates of shear stresses/strain around a pile subject to cyclic loading.

Traditional methods of lateral pile analysis give the pile head displacement and rotation due to the shear loads and moments applied at the pile head, and also calculate displacements, moments and shear stress down the length of the pile. The problem of estimating the soil stresses and strains is overlooked in most common methods. A survey of literature on this subject shows that some of the popular methods include the strain wedge method. A number of methods have been used to derive the likely shear stresses/strains induced due to the vibrations and wave loading ranging from simple to complex methods. A preliminary 3D finite element model was developed which would capture the basic failure mechanism of the pile foundation. A snapshot of the model is shown in Figure 15. The pile was modelled as a hollow steel section with the average pile diameters taken across the depth. Perfect contact was assumed between the pile and the surrounding soil.

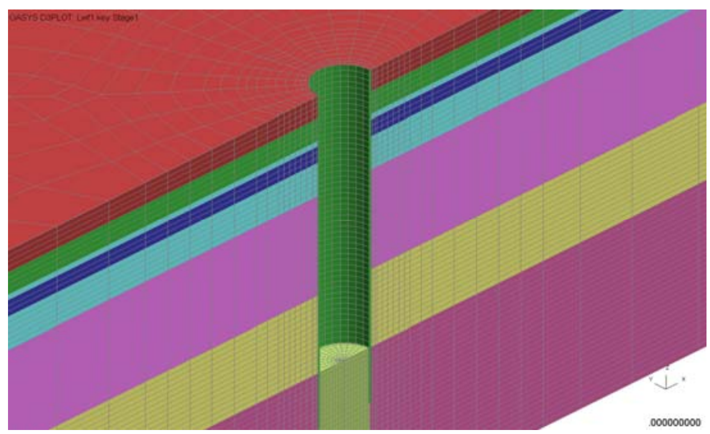

Fig. 15: Finite element model developed for analysis

This model was analyzed using Oasys LS DYNA. The analysis was run in various stages. In the first stage in situ soil gravity stresses were established in the soil mass. In the subsequent stage the pile was installed in the soil fabric (using the 'wished in place' approach) without changing the soil stresses. Once the stresses had been established the maximum horizontal load and moment was applied.

Figure 16 plots the variation of shear stress ratio (CSR) in the silt layers. For plotting purposes CSR is defined as follows the ratio of half of Von Mises to Effective Mean Stress. Figure 16 reveals that the peak shear stress ratio is generally less than 0.4.

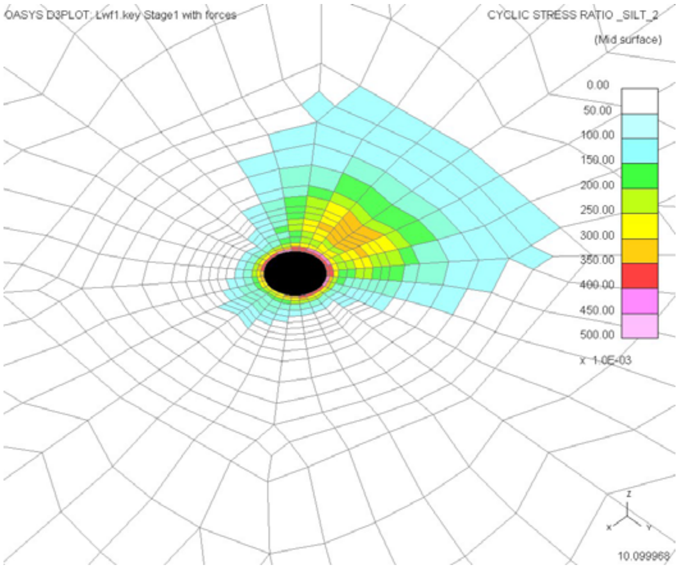

Fig. 16: Variation of CSR at areas near the pile foundation

The shear stress ratio was correlated with the excess pore pressure generated during cyclic loading. Dobry et al (1982) examined the pore pressure generation during undrained cyclic loading as a function of shear strain. They found that for a constant number of loading cycles to a constant shear strain the relationship between pore pressure generation and shear strain is essentially identical over a wide range of relative densities. Additionally pore pressure does not increase until a level of threshold strain is reached. It would have been ideal if advanced triaxial testing was done on the foundation material. However for the present project we did not have this information.

After a careful review of the published information and ensuring that the geotechnical properties are similar, a study carried out by Erten & Maher (1995) has been used to provide a correlation between excess pore pressures and cyclic shear strains. Erten & Maher (1995) performed a number of strain controlled cyclic triaxial tests on silty sands investigating the influence of silt content, plasticity of silt and the number of cycles on pore pressure generation potential. The tests were carried out on specimens formed by combining different proportions of sand, silt and low plasticity silty clay to provide results for materials with different percentages of non-plastic and low plasticity silt. The samples were tested within a cyclic shear strain range of 0.015-1.5% and the tests were carried to 1000 cycles or to initial liquefaction, which ever occurred first.

The authors were able to conclude the following:

- The tests demonstrated that a clear relationship existed between cyclic shear strains and pore pressure generation in silty sand. Also, it was observed that a threshold level of cyclic shear strain exists below which little pore pressure generation takes place irrespective of silt content, silt plasticity and number of cycles. The threshold strain is estimated to be of the order of 0.01%.
- There is a significant increase in the pore pressure for both non-plastic and low plasticity silty sands at strain levels above the threshold value. The pore pressures increase in non-plastic silty sands with up to 30% silt content. The pore pressure does not change significantly for low plasticity silty sands, for up to 60% silt content after which a significant reduction in pore pressures was observed.
- The effect of the number of loading cycles on the magnitude of pore pressures is a function of shearing strain and do not significantly influence the threshold strain level below which little pore pressures are generated. Increase in pore pressure is significant from 1 to 30 no. cycles and reaches a limiting value at approximately 100 cycles.

Using the data presented by Erten & Maher (1995), it is possible to obtain an estimate of the pore pressure generation potential of the silt/silty sand at the site. Figure 17 show the excess pore pressures reported plotted as a function of the cyclic shear strain. The charts from Erten & Maher (1995) can be used to correlate pore pressures to the shear strains obtained from previous analysis. From the previous analysis the shear strains are in the range of the 0.6%-0.4%. However the particle size distribution curves and laboratory results show the presence of plastic fines in some of the samples and these tend to reduce the excess pore pressure rise in silty soil. Thus the likely range of pore pressure rise is estimated to be in the range 0.4-0.5 for the top silt layer. For other layers the pore pressure ratio is likely to be less than this. This information was helpful to the client is assessing whether the generation of excess pore water pressure would be a problem in stability assessment of the foundation. This is an example of adding Value during our design work.

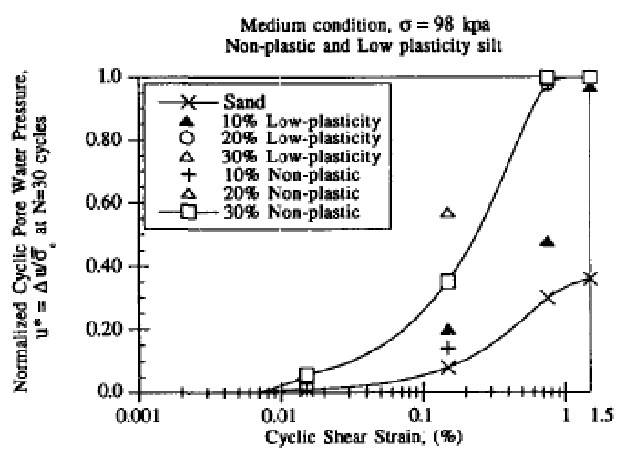

Fig. 17: Normalised pore pressure changes vs. cyclic shear strain for non plastic and low plasticity silt at N=30 cycles

CONCLUSIONS

In this paper we have presented the importance of correctly identifying the hazards and designing the structure to cope with these hazards. The key points that have been discussed in the paper are

- Performance based design criteria should be the corner stone in earthquake geotechnical design.
- However it is noted that it is difficult task to predict factor of safety against these performance criteria.
- Some examples were demonstrated where the performance matrix has been developed during design.
- The usefulness of Probabilistic Seismic Hazard Assessment (PSHA) was demonstrated by using an example. It is noted that most code defined spectra are conservative.
- An example was presented to highlight the benefits of site response analysis to derive surface spectra. It was also highlighted that certain codes (Eurocode-8) do not allow the site specific spectra to be used if it is below the code defined spectra.
- Liquefaction assessment was discussed for marginal soils. It was shown that it is still very challenging to predict liquefaction susceptibility in silty soils with low plasticity. The important issue was to understand the consequences of liquefaction. Additional level of complexity will always add value to the analysis.
- Some examples were shown where accounting for dynamic soil structure interaction (DSSI) is beneficial in saving costs and providing a better design.
- It was also shown that in some cases we have to analyze a problem using unconventional methods to obtain a solution.

In conclusion we can say that earthquakes are often very clever in finding mistakes in design. An inadequate design will be 'caught out' and will lead to costly remediation works. However we cannot expect our structures to work efficiently for beyond design events like the Boxing Day Tsunami.

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