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Modern Approaches in Soil Liquefaction Analysis

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ABSTRACT

There are various approaches for the determination of liquefaction potential of soil deposits. Most of these approaches are based on empirical studies which uses deterministic as well as the probabilistic methods. Although being able to estimate the potential for liquefaction-induced ground failure is important, it is desirable to correlate this probability with the severity of ground damage at the foundation or near the foundation. The study stems from the growing demands, about the effects of after liquefaction in the proposed buildings and foundations and uses a design procedure using performance based design procedures.

Apart from the previous empirical analysis to evaluate the liquefaction potential, non-linear finite element analysis are performed in this study. The analyses are computed for different seismic intensities.

This study aims to use the non-linear finite element analysis results with the new correlated engineering demand parameter, PPR_max, to evaluate the vulnerability curves for the designers to have a better understanding of the problem at hand just by using the appropriate earthquake intensity level.

Introduction

During the past three decades, a large volume of literature on geotechnical earthquake engineering, and in particular, on soil liquefaction, has been published. (Kramer, 1996) The theoretical base of the subject is out of this study and has been extensively investigated for the past years. Numerical and experimental approaches are developed and used.

Earthquake induced liquefaction is a phenomenon of temporary loss of shear strength of a soil during an earthquake. During past times, a large volume of literature on geotechnical earthquake engineering and in particular in soil liquefaction has been published.

Although determining the liquefaction potential of the soil is an important step in engineering approaches, the possible effects of the liquefaction phenomenon shall be more pronounced. Compared to the studies about the mechanism of soil liquefaction engineering, little work is done for the after effect of liquefaction. What is more important is to practising engineer is the effect and engineering outcome of liquefaction.

Liquefaction potential is generally evaluated by comparing consistent measures of earthquake loading and liquefaction resistance. Among the various methods for evaluating liquefaction

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potential of soils, the simplified procedure of Seed and Idriss (Youd and Idriss, 2001) is the most widely adopted procedure, which will not be presented here in detail but can be found in any textbook and database. The potential for liquefaction is then described in terms of a factor of safety against liquefaction,

FS = CRR/CSR (1)

Liquefaction potential index, IL, on the other hand is defined by Iwasaki et.al (1982);

$$I_L = \int_0^{20} F_1 W(z) dz \tag{2}$$

Where F1 is an index defined as: F1=1-Fs, if Fs =<1,0; and F1=1 if Fs>1, where Fs is the factor of safety against liquefaction triggering.

The integration is carried out from ground surface (z=0) to the depth of 20m, the depth below which the effect of liquefaction on the failure potential of the ground is considered negligible. W(z) is the weighting function of the depth, which is used to account for the effect of liquefaction at different depths to the failure potential of the ground.

Weighing function W(z) is defined by Iwasaki et al. (1982) as follows:

W(z)=10-0.5z (3)

Iwasaki et al., (1982), investigate 6 different earthquake case studies by which, they have encountered 64 post-liquefaction and 23 non-liquefied cases. Their work ends with the correlative study of liquefaction index and DSI index, which can be seen in Table 1,

Table 1.Probability of failure by liquefaction (Iwasaki et al, 1982)

Liquefaction Potential Index, IL	**Probability of Failure Based on Liquefaction**
IL = 0	Very low
IL<5	Low
5<IL<15	High
IL>15	Very High

Juang et al, (2005) analysed the CPT soundings and the seismic parameters from the Kocaeli and Chi-Chi earthquakes case studies, which they used to compute the liquefaction index and the probability of liquefaction-induced ground failure, PG. From their study, they have expanded the studies of Iwasaki et.al, and conclude with a correlation, in which the DSI index can be used as a damage measure. Their analyses end up with a correlation curve given in Figure 1, by which they determine the correlations of liquefaction potential index with the damage severity index given in Table 2.

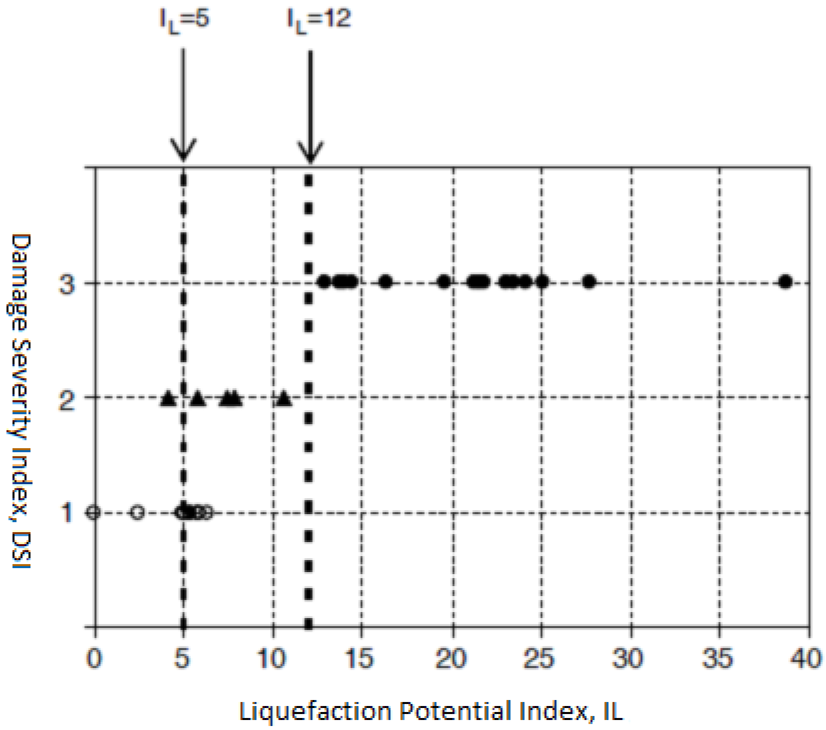


Figure 1. IL vs DSI, correlation graph (Juang et.al, 2005)

Table 2. Damage severity index (Juang et al, 2005)

Liquefaction Potential Index, IL	**Damage Measure, DSI (Damage Severity Index)**	**Damage Properties**	**Damage Details**
IL<=5	DSI=1	No Damage	No settlement ,tilting. No lateral movement or sand boils
5 < IL <=12	DSI=2	Few to No damage	Settlement <25 cm, Tits of buildings <3°; lateral movement < 10 cm
IL > 12	DSI=3	Big Damage	Settlement >=25 cm; Tilt >=3°; Lateral Movements >=10 cm, or collapse of buildings.

In this study, our aim is to correlate the liquefaction potential index with the pore pressure ratio determined from the analysis, and later extend the study further into vulnerability curves. The analyses were performed by using two-level scenario seismic level, which allow consideration of all magnitudes contributing to the peak ground acceleration at the site. In the current framework, the ground motion is characterized by an intensity measure IM, which is chosen to be the peak ground acceleration, PGA.

The effect of the IM on the model is expressed by the engineering demand parameter, maximum pore pressure ratio, PPR_max,in this case. The most important and meaningful part of the framework is the effect of liquefaction- damage measure, DM, which is expressed as the damage severity index (DSI) in this study.

Numerical Simulations

The numerical simulations were performed using the UBCSAND effective stress model (Puebla et.al.,1997) which has been implemented in the computer code Plaxis. UBCSAND is a fully nonlinear, effective stress soil constitutive model that is commonly used in practice.

The model capture the liquefaction triggering response observed in the field. For that purpose, repetitive analyses were done and the parameter Maximum pore pressure ratio is determined from these analyses by the following formula (4).

$$PPR\_\max = \frac{p'i - p'c}{pi} \tag{4}$$

In formula 4, p'i indicates the initial effective stress; p'c is the current effective mean stress of the soil cluster in question. The PPR-max values are taken directly from the analysis results as an average of the depth of the layer in question.

A potentially liquefiable site is chosen in the analysis, in where, various preventive measure against liquefaction were taken, for instances jet-grouting and cement injection. The site is situated in Izmir, along the Karsiyaka metro railway line. The site is very close to the sea side; therefore the water table level is taken as the surface level. In order to characterize the site, the soil investigation campaign is followed including:

- 4 boreholes
- 4 CPT
- 4 Geophysical Cross-Hole Tests, providing information on Vp and Vs wave velocities
- Geotechnical laboratory tests on undisturbed soil specimens

The description of the site properties and geotechnical soil model is given in Figure 2 with the layout of the cone penetration test results. The SPT resistances ranges from relatively low (N1 60 =10) to moderately high levels ($N1_{60}$=30)

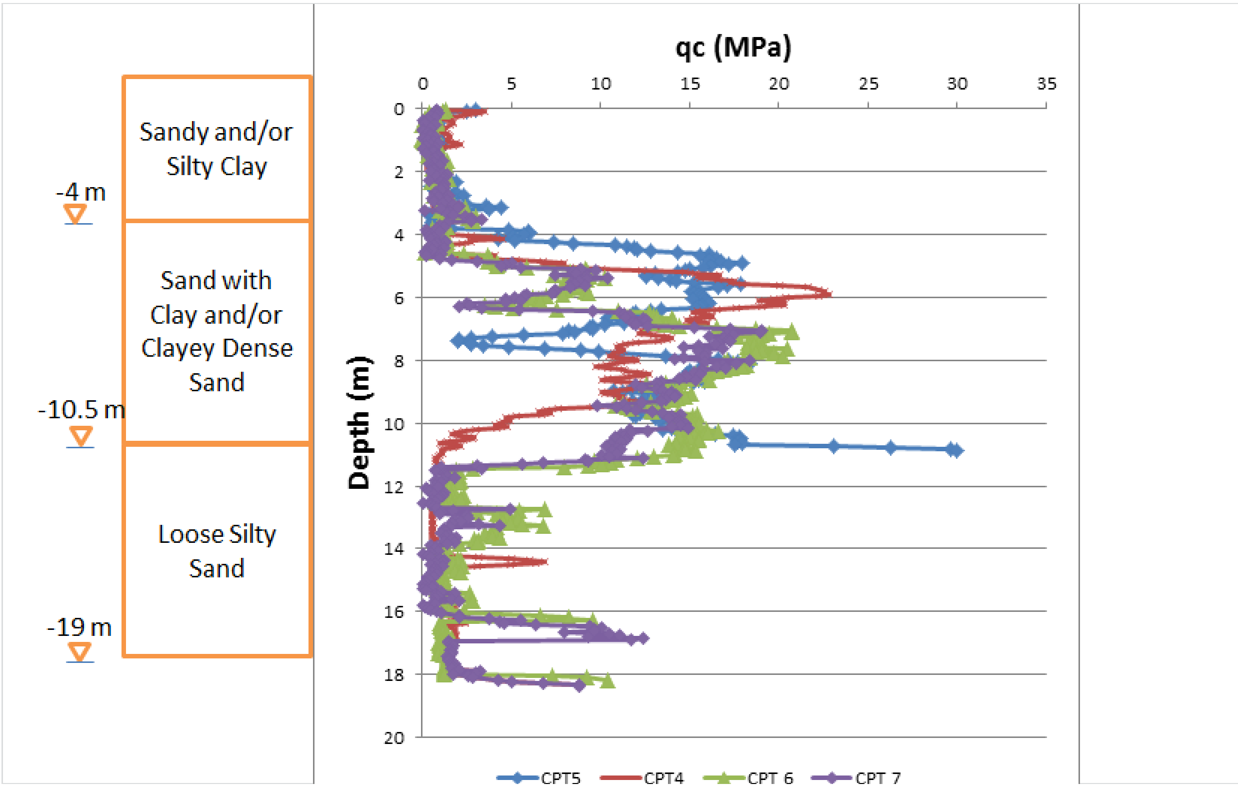


Figure 2. Geotechnical soil model and CPT test results from the site

In the numerical analyses, the nonlinear nature of the stress-strain behavior and some important mechanical behaviours like the volumetric change, dilation and pore-pressure generation are generated by the constitutive relations embedded in the UBCSAND model (Kramer and Elgamal, 2001; Beaty and Bryne, 1998; Petalas and Galavi, 2013) This model is generally accepted and used in liquefaction based problems and phenomenas among geotechnical engineers (NCHRP, 2012) This model, which is developed in British Columbia University, has been used and calibrated in many works (Puebla et al, 1997; Beaty and Bryne, 1998; Bryne et al, 2004; Sriskandamumar, 2004; Park, 2005) The theoretical basis of the model has verified many times in laboratuary consolidation, triaxial, simple shear, repetitive direct shear and centrifuge tests (Tsegane, 2010)

The constitutive soil parameters are gathered from the soil laboratory tests and the empirical equations which were calibrated by other works. Despite the diversity of the model, it can be used by very few parametres, which can be evaluated from the classical laboratuary tests and some well-defined empirical equations given in Table 3.

Table 3. Geotechnical model parametres used in the model

Name	Symbol	Unit	Method	Default
Constant volume friction angle	Φcv	(°)	CD TxC or DSS	-
Peak friction angle	Φp	(°)	CD TxC or DSS	-
Elastic Shear Modulus	K_G^e	-	Curve Fit	-
Cohesion	C	kPa	CD TxC or DSS	0
Elastic Plastic Modulus	K_G^p	-	Curve Fit	-
Elastic Bulk Modulus	K_B^p	-	Curve Fit	-
Elastic Shear Modulus Index	ne	-	Curve Fit	0.5
Elastic Bulk Modulus Index	me	-	Curve Fit	0.5
Plastic Shear Modulus Index	np	-	Curve Fit	0.5
Failure Ratio	rf	-	Curve Fit	0.9
Densification Factor	fac_{hard}	-	Curve Fit	1
Post Liquefaction Factor	fac_{post}	-	Curve Fit	0

A model view of the work in finite element program can be seen in Figure 3.

Figure 3. Model geometry used in the analyses

Two widely used mean return period seismic events are used in the analysis. The selected ground motions are then scaled in time domain in order to equate its response spectrum to the desired response spectrum which is locally derived in DLH codes and standards. These seismic levels and related spectral accelerations are given in Table 4. Ground motions and some of their seismic properties are given in Table 5 and Figure 4.

Table 4. Seismic levels and related spectral accelerations defined in the turkish building code (DLH)

Seismic Level	Spectral Acceleration(g) Ss=0.2 sn	Spectral Acceleration(g) S1=1 sn
D1 Level	0.42	0.12
D2 Level	0.81	0.26
D3 Level	1.91	0.89

Table 5. Peak ground surface acceleration hazard Information

No	Earthquake	Record Statio	Mw	475-yr ama	2475-yr ama
1	Düzce	Lamont 531	7,40	0,4	0,46
2	Lome Priata	Sierra	6,93	0,38	0,5
3	Irpınia	Auletta	6,00	0,4	0,44
4	Tabas	Tabas	7,35	0,38	0,39
5	Northridge	Lake Hughes	6,69	0,37	0,5
6	Kocaeli	Tekirdağ	7,51	0,37	0,43
7	San Fernando	Cedar Springs	6,61	0,41	0,58

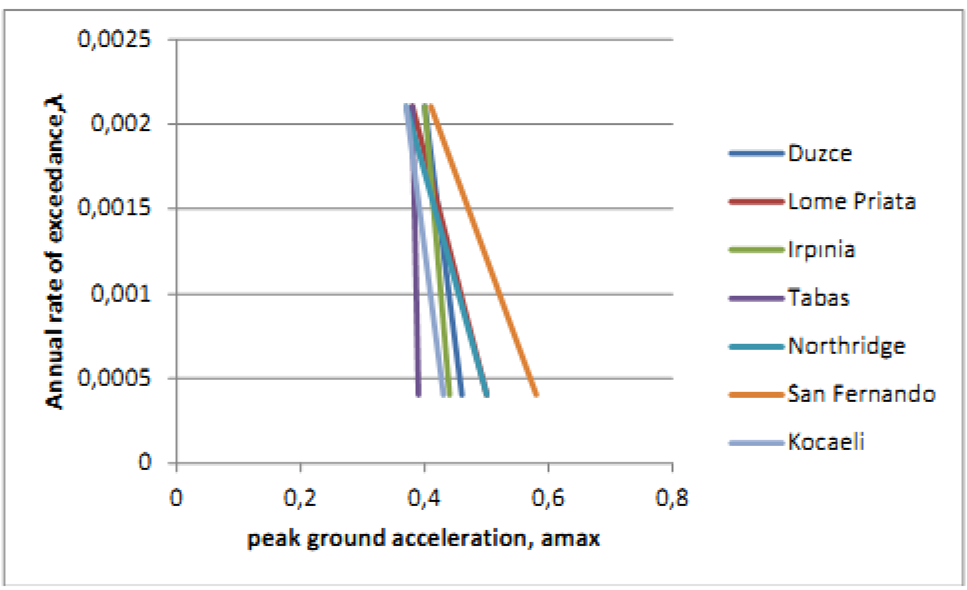


Figure 4. Seismic hazard curve for the site used in analyses

Analyses Results

A total of 28 time domain liquefaction analysis were done and the results are given for different seismic intensities (D2 and D3 defined in Table 3) in Table 6 and 7 below. In the tables, the first column is the liquefaction index, IL, calculated by using formula (2), for the two different soil layers (S1 and S2) which are prone the liquefaction. DSI, which is evaluated based on the IL value per Table 2, is given for the two different layers under the second column. And in the third column, the PPR_max values are listed which are calculated from the numerical analyses for the two different layers.

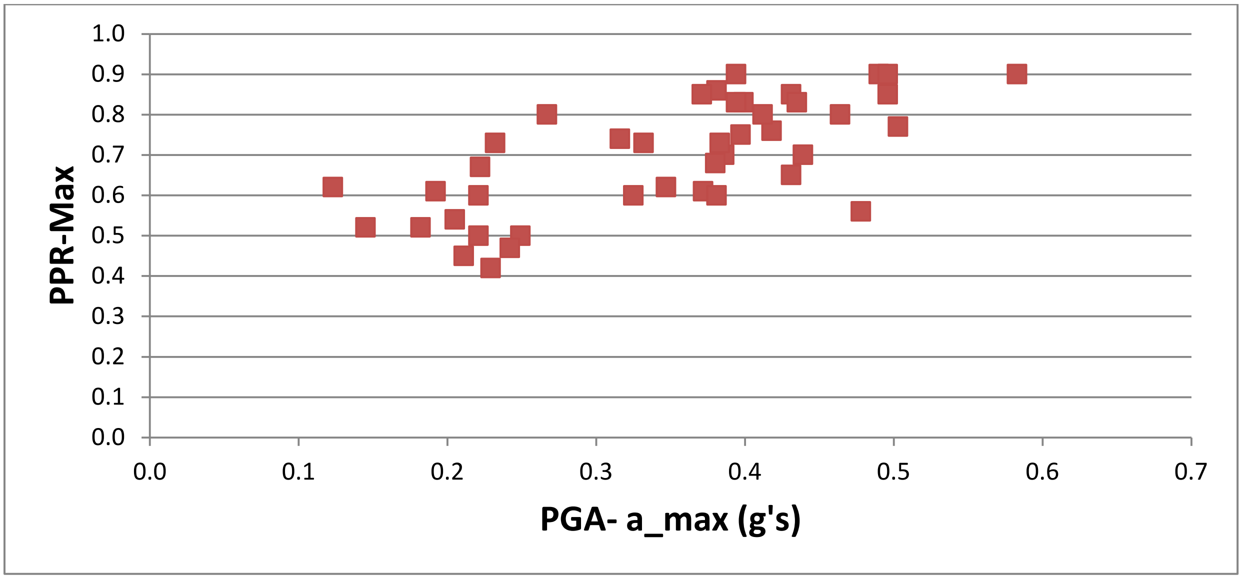
Table 6. Case D3 ground motions

LI-Liquefaction Index		DSI-Damage Severity Index		PPR_max Max. Pore Pressure Ratio	
S1	S2	S1	S2	S-1	S-2
5,67	3,90	2	1	0,90	0,50
5,40	3,71	2	1	0,85	0,50
5,13	3,51	2	1	0,90	0,58
4,68	3,06	1	1	0,90	0,62
1,35	0,46	1	1	0,83	0,43
1,98	1,04	1	1	0,83	0,46
4,95	3,32	1	1	0,83	0,58
3,42	2,08	1	1	0,90	0,59
4,41	2,93	1	1	0,76	0,55
5,22	3,51	2	1	0,77	0,54
5,58	3,84	2	1	0,65	0,45
5,94	4,10	2	1	0,56	0,44
4,68	3,12	1	1	0,85	0,57
4,68	3,12	1	1	0,90	0,59

Table 7. Case D2 ground motions

LI-Liquefaction Index		DSI-Damage Severity Index		PPR_max Max. Pore Pressure Ratio	
S1	S2	S1	S2	S-1	S-2
4,23	2,73	1	1	0,75	0,40
5,13	3,51	2	1	0,75	0,56
3,51	2,21	1	1	0,80	0,55
3,24	2,02	1	1	0,70	0,45
1,98	1,04	1	1	0,70	0,44
-	-	1	1	0,60	0,40
4,23	2,73	1	1	0,73	0,54
4,86	3,25	1	1	0,86	0,55
3,87	2,47	1	1	0,61	0,50
3,96	2,54	1	1	0,73	0,47
4,77	3,19	1	1	0,62	0,46
5,13	3,45	1	1	0,60	0,44
3,33	2,08	1	1	0,68	0,57
3,24	1,95	1	1	0,85	0,56

The two different engineering demand parameters, PPR_max and IL; are tabulated and their conformity with the seismic intensity parameter, PGA is given in Figure 5 below. From the figures, it can be concluded that the PPR_max is a better engineering demand parameter and more compatible with the PGA value which is a supportive point in selecting a different and more conformative engineering demand parameter for the further performance based analyses.



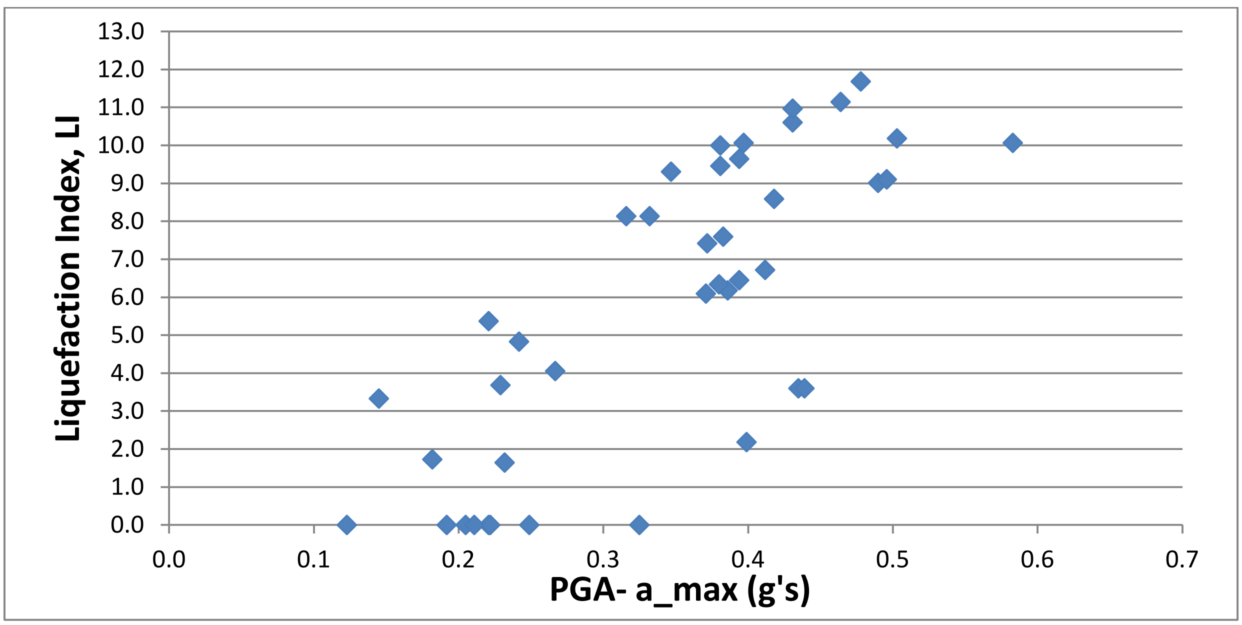
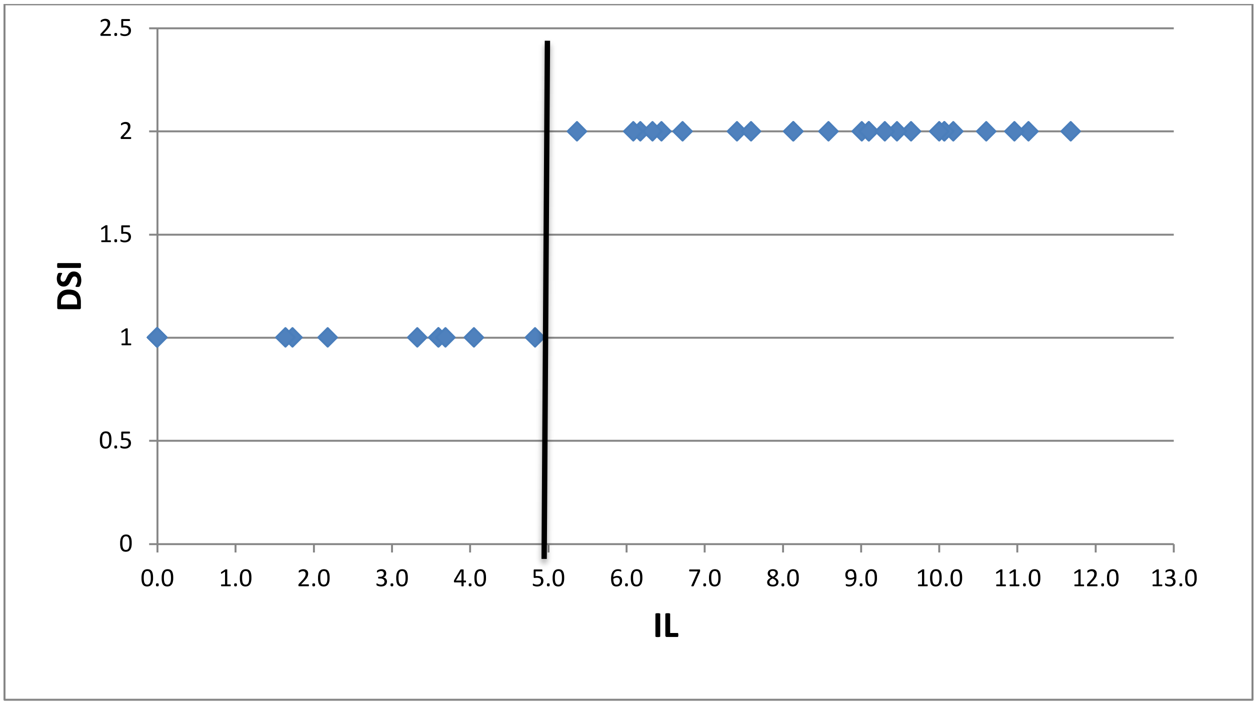


Figure 5. Engineering demand parameters against seismic intensity parameter

After the analyses, to be able to tabulate the results, IL is first generated with the DSI. To be able to see the change in DSI with respect to PPR_max; a second graph is generated. From the result of the analyses, the value of PPR_max = 0.47 is correlated with the IL=5 as given in Figure 6. The analyses conclude with the following relationship:

$PPR_{max} \leq 0.47 \Longrightarrow DSI = 1$

$PPR_{max} > 0.47 \Longrightarrow DSI = 2$



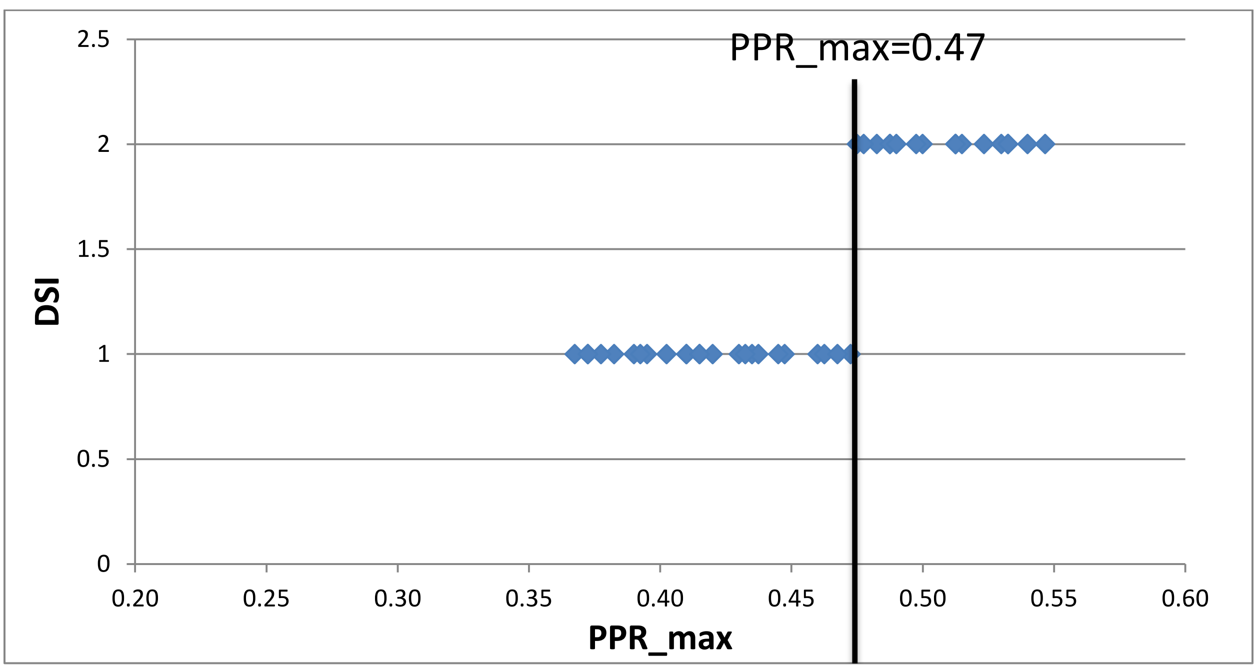


Figure 6. DSI versus IL and PPR_max

By combining the ground motion hazard with the seismic response analyses, the demand hazard can be obtained from the below formulation.

$$\lambda_{EDP}(edp) = \sum_{im} P[EDP > edp\ IM = im]\Delta\lambda_{IM}(im) \tag{5}$$

In equation 5, $[EDP > edp\ IM = im]$ is the complementary cumulative distribution function for EDP/IM, which gives the probability of exceeding the EDP parameter with the given IM; an λ_{EDP} is the annual rate of exceedance.

Conclusions

As stated before, damage severity index and liquefaction index parameters are correlated by following several case studies and analysing the effects of liquefaction on surface and foundations. Therefore, this study will build upon these investigated issues and tries to connect the seismic intensity parameter with a new and more conformative engineering design parameter, PPR_max. The results will also give insight in to the local site conditions and the possible severity of liquefaction damage in local environment.

To sum up, this study defines the beginning of the general framework, which still need to incorporate the range of peak ground accelerations in the analyses. The engineering design parameter is calibrated against the known and validated design parameter liquefaction index by using full dynamic analyses taking into account the latest modern techniques and theoretical accounts.

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