Evaluation of Some Geotechnical Properties and Liquefaction Potential from Seismic Parameters

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Abstract

A geophysical study using seismic wave velocities data, including compressional and shear wave velocity (V_p and V_s) values, for 14 sites has been carried out. These sites are located within the Mesopotamian plain and surroundings. Both seismic and geotechnical data have been conducted by the National Center for Construction Laboratories and Research (NCCLR) in Iraq. Some geotechnical parameters have been deduced from seismic velocities either from V_p or V_s. Correlations between seismic velocities (V_p and V_s) and geotechnical properties have been derived. These relations show direct proportionalities between V_p and V_s with standard penetration test (SPT-N value). LiuefyPro software has been utilized for two selected Iraqi sites to investigate the liquefaction potential. Input data of the program will be based on those derived from the compressional and shear wave velocities. The application shows a total settlement for saturated and dry sand of 32 mm for the first site while no settlement has been indicated for the second site. It was found that the high value of both wave velocities for a cohesionless fully saturated soil gives an indication that this soil is unable to liquefy and settle under earthquake excitation and vice versa.

تقييم بعض الخواص الجيوتكنيكية و احتمال التسييل من المعاملات الزلزالية

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الخلاصة

أجريت دراسة جيوفيزيائية باستعمال معلومات عن سرع الموجات الزلزالية تتضمن سرعة موجة الضغط و موجة القص (V_p و Vs) لأربعة عشر موقعا. و تقع هذه المواقع ضمن سهل وادي الرافدين و المناطق المحيطة به. و قد أجري كل من التحريات الجيوتكنيكية و الجيوفيزيائية من قبل المركز الوطني للمختبرات و البحوث الانشائية (NCCLR) في العراق. و قد تم الحصول على بعض المعاملات الجيوتكنيكية من السرع الزلزالية V_p و Vs. و بعدها أشتقت علاقات بين هذه السرع الزلزالية و الخواص الجيوتكنيكية. و قد بينت هذه العلاقات وجود تناسب مباشر بين V_p و Vs و قيم فحص الاختراق القياسي (N). و أستعمل برنامج الحاسبة LiuefyPro لتحليل موقعين منتخبين في العراق لايجاد احتمال التسييل فيهما. و تستند المعلومات المدخلة للبرنامج على المعادلات التي أشتقت من السرع الزلزالية (سرعة موجة الضغط و القص). و قد بينت التطبيقات حساب الهبوط كلي لرمل مشبع و اخر جاف حيث سجل هبوط مقداره (32 ملم) للموقع الأول و لم يسجل هبوط للموقع الثاني. و قد وجد أن السرع العالية لكلا نوعي السرع الزلزالية للتربة غير المتماسكة المشبعة كليا يعطي تصورا على أن هذه التربة غير قابلة للتسييل و الهبوط تحت تأثير الهزات الأرضية و العكس صحيح.

1. Introduction

There is an increasing application of seismic parameters for geotechnical determination for various underground constructions. Two demands arose in utilizing the correlation between seismic data and geotechnical properties (Domenico, 1984) [4]:

1. To improve the measuring procedure and to refine the interpretation techniques.
2. The analytical nature may increase the understanding of the significance of seismic parameters for geotechnical determinations.

Correlations among engineering properties come in many forms, but all have a common theme; specifically, the desired correlation utilizes a large database of results based on past experience.

Soil liquefaction and related ground failures (flow and deformation failures) are commonly associated with large earthquakes. During earthquakes, the shaking of ground may cause a loss of strength or stiffness which results in the settlement of buildings, landslides, failure of earth dams or other hazards. In common usage, liquefaction refers to the loss of strength in saturated cohesionless soils due to the build-up of pore water pressures during dynamic loading. This phenomenon is associated primarily, but not exclusively, with saturated cohesionless soils. In a more general manner, soil liquefaction has been defined as the transformation "*from a solid state to a liquefied state as a consequence of increased pore pressure and reduced effective stress*" (Bordare, 1988) [3]. A more precise definition of soil liquefaction is given by Sladen *et al*. (1985)[12] and National Research Council Committee (1985)[10] :"*Liquefaction is a phenomenon wherein a mass of soil loses a large percentage of its shear resistance, when subjected to transient or periodic loading, and flows in a manner resembling a liquid*." Thus, both flow and deformation failures are said to be liquefaction failures (Bodare, 1988) [3]. Flow failure describes the condition where a soil mass can deform continuously under a shear stress less than or equal to the static shear stress applied to it. Whereas, deformation failure involves unacceptable large permanent displacements or settlements during (and / or) immediately after shaking, but the earth masses remain stable following shaking without great changes in geometry. If a large increase in pore pressures occur within an earth mass as a result of an earthquake, significant cyclic and permanent deformation can occur (Bordare, 1988)[3].

From the seismological point of view, it is worth to mention that Iraq is surrounded by hazardous zones especially those located in the eastern (Zagros Zone) and northern (Taurus Zone) territories. Besides, the nearest historical earthquake to Baghdad city had been occurred in 1508 with magnitude value of 6.4 and 100 km epicentral distance (Ambrassys and Melville, 1982)[1].

The objective of the present paper is to evaluate some geotechnical properties from seismic parameters (particularly compression and shear wave velocities), and to evaluate the susceptibility of soil deposits to liquefaction for two selected sites using LiquefyPro software.

2. Site Locations and Descriptions

Data for 14 site locations over all Iraq were provided by the National Centre for Construction Laboratories and Research (NCCLR). These sites are located within the Mesopotamian plain and surroundings (Fig. 1). Several geotechnical properties at different depths have been obtained for each site such as compressional and shear wave velocities (V_p and V_s) , standard penetration number (N), fine contents, moisture content, plasticity index and ground water levels . The variation of V_p and V_s with depth are presented in Figures 2 and 3 respectively. In general, the obtained V_p and V_s values are within the ranges of 150 - 2500 m / s and 100 – 2000 m / s respectively with V_p / V_s about 1.26 – 2.5. The values of N versus depth (Fig. 4) are interlocked with each other, besides it is clearly noticed the direct relation between SPT (N-values) and depth. Moisture content values are ranging between 10 – 50 % which increases with increasing depth from earth surface. The average ground water level is around 2.75 m. Plasticity index, wet and dry unit weights values increase with increasing depth. An increase relation is noticed between V_p and V_s with fine contents, where sites with high percentage of sand give high wave velocities while sites with high percentage of fines give low velocities.

3. Correlations among Data

Correlations between seismic velocities and geotechnical properties, when combining seismic investigations with drilling for the evaluation of soil/rock conditions for a project, it is very common to make an account of the ordinarily amount of boreholes that not all the seismic velocities recorded are represented by drilling results. In order to get the average values for velocities not concerned by the drilling, an attempt is made to find empirical average curves for existing relations between velocity and other properties.

For this purpose, the program "Curve Expert 1.3" which is a comprehensive curve fitting system for Windows has been used to evaluate the obtained fitting curves and empirical equations.

As the standard penetration test (SPT) is the most widely used field test and very valuable method of soil strength investigation, so correlations between seismic velocities and *N* values have been carried out. The relation of *N* values with V_p and V_s values is illustrated in Figures 5 and 6, respectively. The suggested empirical equations can take different parameters according to the points of SPT values that pass through. Thus, the distribution of these points is related to inaccuracy of SPT number due to ground water table levels and percent of soil composition (clay, silt, sand) and the existence of gravel.

The direct relation of V_p and SPT uncorrected values (*N*), given in Figure 5, is satisfied by the following empirical equation:

$$N = 61.22 \cdot (1 - e^{(-0.00137 \cdot V_p)}) \qquad (1)$$

Similarly, the relation of V_s and SPT, shown in Figure 6, is given by an empirical equation as follows:

$$N = 88.54 \cdot V_s^{(-40.7/V_s)} \qquad (2)$$

In general, Imai *et al.* (1976)[6] found an empirical relation between uncorrected *N* and V_s for sands:

$$V_s = 89.8\, N^{0.341} \quad \text{or} \quad N = (V_s / 89.8)^{2.952} \qquad (3)$$

The values of (*N*) used in equations (1) to (3) are uncorrected. This is usually done for such correlations because both the wave velocity and the standard penetration number (*N*) refer to field values.

Comparing the obtained empirical equations with those of Imai relations, it is clearly observed that higher values are given for SPT value than those obtained in the present study when $V_s > 400$ m/s, and vice versa for $V_s < 400$ m/s. That is because Imai equation is restricted to sandy soils only, whereas the obtained relations given in the present study represent a mixture of clay, silt, sand and gravel.

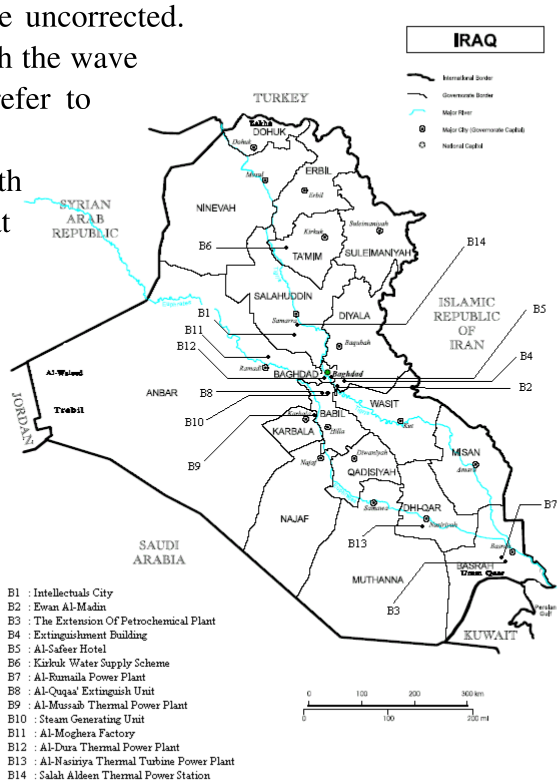


Fig. 1. Locations of sites included in the study.

4. Liquefaction Considerations

The character of ground motion, soil type and in situ stress conditions are the three primary factors controlling the development of cyclic mobility or liquefaction. Particle cementation, soil texture and aging – are important factors that can hinder particle arrangement (Seed, 1979)[13]. Soil deposited prior to the Holocene epoch (> 10,000 year old) are usually not prone to liquefaction (Youd and Perkins, 1978)[21] reasonably due to cementation at the grain contacts and increasing frictional resistance resulting from particle rearrangement and interlocking. Therefore, aging of the soil deposits must be accounted when evaluating liquefaction potential (Lion *et al.*, 2006) [8].

Stress history also plays an important role in determining the liquefaction resistance of a soil. Stress history may also contribute to the liquefaction resistance of older deposits. Overconsolidated soils having been subjected to greater static pressure in the past, are more resistant to liquefaction. In addition, the frictional resistance between soil grains is proportional to the effective confining stress. Consequently, the liquefaction resistance of a soil deposits increases with depth as the effective overburden pressure increases. Characteristics of the soil grains (distribution of sizes, shape, composition etc.) influence the susceptibility of a soil to liquefy (Seed, 1979)[13]. Relatively free draining soils such as GW, GP are much less likely to liquefy than SW, SP or SM. Dense granular soils under higher initial effective confining pressures (i.e lower water table beneath surface and deeper soils) are less likely to liquefy. Moreover, low plasticity fines may contribute to the liquefaction susceptibility of a soil. Koester (1992)[7] suggested that sandy soils with significant fines content may be inherently collapsible. Permeability also affects the liquefaction characteristics of a soil deposits. So liquefaction is more likely to occur in clean granular soils. Soils with significant contents of fine (< 0.075 mm.) are less likely to liquefy, especially when the fines are clays.

Wang (1979)[20] established that any clayey soil containing less than 15–20% particles by weight smaller than 0.005 mm and having water content (*wc)* to liquid limit (LL) ratio greater than 0.9 is susceptible to liquefaction. Based on these data, Seed and Idriss (1982)[17] stated that clayey soils could be susceptible to liquefaction only if all three of the following conditions are met: (1) percent of particles less than 0.005 mm(15%) , (2) LL < 35, and (3) *wc*/LL > 0.9 . This standard is known as the "Chinese criteria" due to its origin.

In recent years, a number of other investigators have examined the liquefaction susceptibility and response of soils with significant fines and fine-grained soils. Other investigators have used the "C" descriptor of the Unified Soil Classification System (USCS) as a tool to identify potentially liquefiable soils.

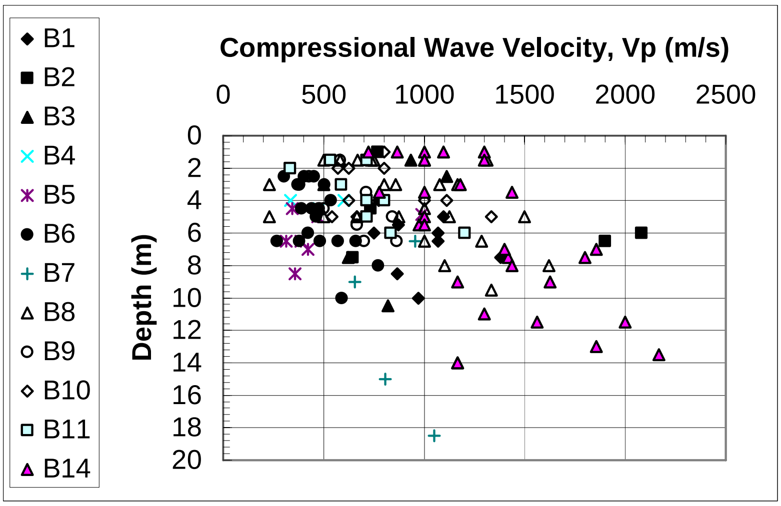


Fig. 2. Variation of compressional wave velocity with depth at the selected sites.

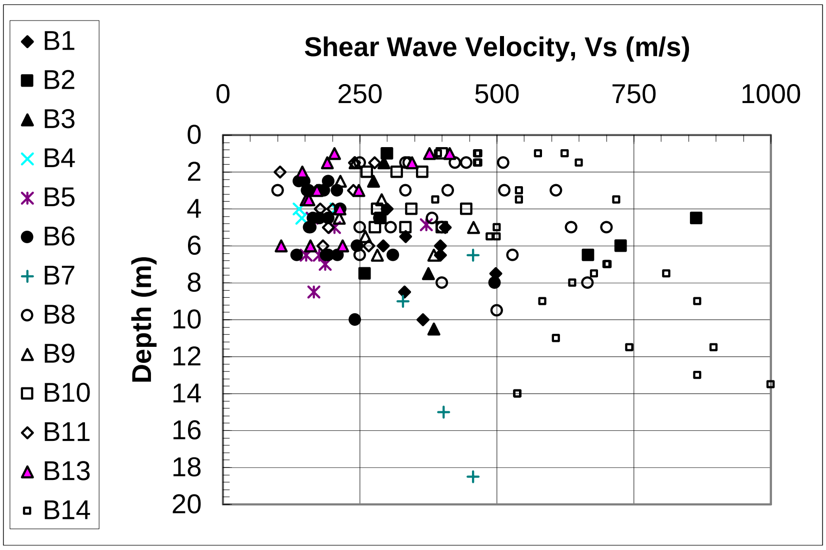


Fig. 3. Variation of shear wave velocity with depth at the selected sites.

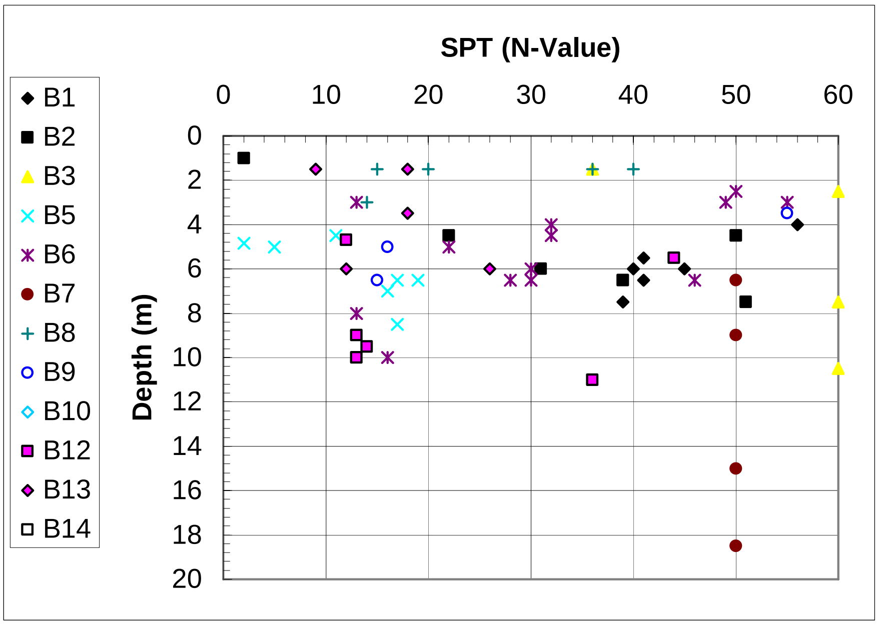


Fig. 4. Variation of the standard penetration number with depth at the selected sites.

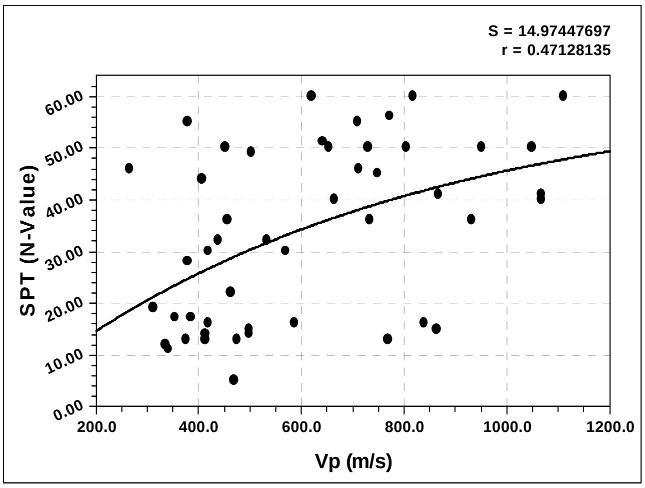


Fig. 5. Relationship between SPT (N value) and compressional wave velocity.

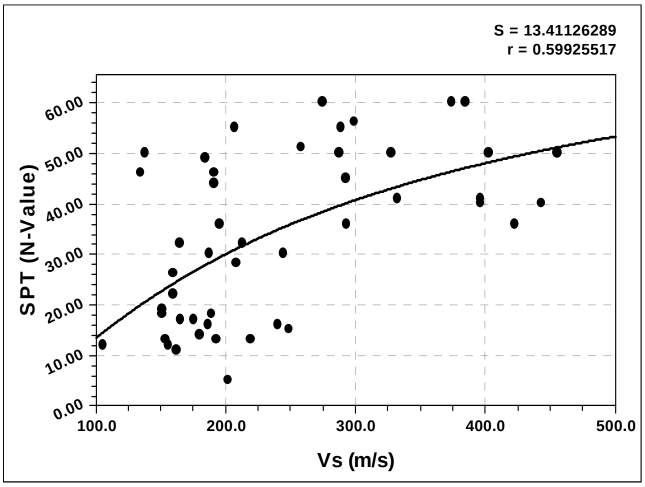


Fig. 6. Relationship between SPT (N value) and shear wave velocity.

5. Evaluation of Liquefaction Potential

The evaluation of potential for liquefaction in a given soil deposits during an earthquake is often assessed using in- situ penetration tests and empirical procedures. The most widely accepted procedure for evaluating liquefaction susceptibility, based on standard penetration test (SPT).

Liquefaction susceptibility at a site is commonly expressed in terms of safety factor versus the occurrence of liquefaction. This factor FS is defined as the ratio of available soil resistance to liquefaction, expressed in terms of cyclic stresses required to cause soil liquefaction, and the cyclic stresses generated by the design earthquake. Both of these parameters are commonly normalized with respect to the effective overburden stress at the depth in question.

$$FS = CRR / CSR \tag{4}$$

CRR: cyclic resistance ratio (soil strength) based on in–situ test data from SPT or CPT tests.

CSR: cyclic stress ratio (earthquake load) induced in the soil by an earthquake.

Consequently, a safety factor of about 1.2 is appropriate in engineering design. The precise factor has to be based on engineering judgment with appropriate consideration given to type and importance of structure and the potential for ground deformation.

Liquefaction susceptibility at a site is commonly expressed in terms of a factor of safety versus the occurrence of liquefaction. This factor is defined as the ratio between available soil resistance to liquefaction, expressed in terms of the cyclic stresses required to cause soil liquefaction, and the cyclic stresses generated by the design earthquake. Both of these parameters are commonly normalized with respect to the effective overburden stress at the depth in question. Because of difficulties in analytically modeling soil conditions at liquefiable sites, the use of empirical methods has become a standard procedure in routine engineering practice (Robertson, 1995)[11].

With the present state of knowledge the prediction of liquefaction is an approximation. However, there is general agreement that the current procedures work well for ground that is level or nearly level. The analysis for steeply sloping ground is less certain. Two basic approaches are used: one is based on standard penetration tests (SPT) and the other on the cone penetration test (CPT).

Although the curves drawn by Seed *et al.* (1985)[16] envelope most of the plotted data for liquefied sites, it is possible that liquefaction may have occurred beyond the enveloped data, but was not detected at ground surface. Consequently, a safety factor (1.2) is appropriate in engineering design. The factor to be used is based on engineering judgment with appropriate consideration given to type and importance of structure and the potential for ground deformation.

6. Liquefaction Analysis Using Computer Program

LiquefyPro is a software that evaluates liquefaction potential and calculates the settlement of soil deposits due to seismic loads. The user can choose between several different methods for liquefaction evaluation: one method for SPT and BPT (Beaker Penetration Test), and four methods for CPT data. Each method has different options that can be changed by the user. The options include fines correction, hammer type for SPT test, and average grain size (D_{50}) for CPT. The settlement analysis can be performed with two different methods.

7. Calculation Theory of the Computer Program

The calculation procedure is divided into four parts (LiquefyPro Manual 2004)[9]:

1. Calculation of *cyclic stress ratio* (CSR, earthquake “load”) induced in the soil by an earthquake.
2. Calculation of *cyclic resistance ratio* (CRR, soil “strength”) based on in-situ test data from SPT or CPT tests.
3. Evaluation of liquefaction potential by calculating a *factor of safety against liquefaction*, F.S., by dividing CRR by CRS.
4. Estimation of liquefaction-induced settlement.

8. Software Application for Liquefaction Potential

As aforementioned, the evaluation of liquefaction potential may be carried out by different approaches; in the present work, only the SPT (*N*-values) will be considered in liquefaction potential analysis due to the lack in the other data. Besides SPT is the most widely and common used field test method and very valuable method of soil strength investigation and should, however, be used as a guide due to its approximate results.

Considering liquefaction and settlement analysis of soil deposits due to seismic loads, one in general can choose between several methods for SPT and CPT data, as mentioned above. In the present work, Idriss and Seed method (1997)[5] had been applied for fines correction of SPT. For settlement analysis, Tokimatsu and Seed method (1987)[19] had been used. The required input data for SPT are depth, SPT blow counts (N- value), total unit weight (γ) and fines %. Besides other limited data such as peak horizontal ground acceleration for earthquake (PGA) 0.25 g, earthquake magnitude (6) and water table depth during earthquake (2 ft).

LiquefyPro software has been utilized for two selected Iraqi sites to investigate the liquefaction potential. The input data of the program will be based on those derived from the compression and shear wave velocities.

Finally, it may be stated that because of difficulties in analytically modelling soil conditions at a liquefied sites, the use of empirical approaches has become a standard procedure in routine engineering practice. With the present state of knowledge, the prediction of liquefaction is an approximation, however there is a general agreement that the current various procedures work well.

9. CSR - Cyclic Stress Ratio Computations

The earthquake demand is calculated by using Seed's method, first introduced in 1971 (Seed and Idriss, 1971)[15]. It has since evolved and been updated through summary papers by Seed and colleagues. Participants in a workshop on liquefaction evaluation arranged by NCEER reviewed the equation recently in 1996. The equation is as follows:

$$CSR = 0.65 \cdot \frac{\sigma_o}{\sigma'_o} \cdot a_{\max} \cdot r_d \tag{5}$$

where:

CSR is the cyclic stress ratio induced by a given earthquake,

0.65 is weighing factor, introduced by Seed, to calculate the number of uniform stress cycles required to produce the same pore water pressure increase as an irregular earthquake ground motion.

σ_o is the total vertical overburden stress.

σ'_o is the effective vertical overburden stress.

$a_{\max}$ is the peak horizontal ground acceleration, PGA, unit is in g (≈ 0.25 g).

r_d is a stress reduction coefficient determined by formulas below.

r_d = 1.0-0.00765·z for z 9.15 m

r_d = 1.174-0.0267·z for 9.15 m < z 23 m

r_d = 0.744-0.008·z for 23 m < z 30 m

r_d = 0.5 for z > 30 m

CRR - Cyclic Resistance Ratio from SPT/BPT (Soil Strength)

The CRR liquefaction curves are developed for an earthquake magnitude of 7.5 and will hereafter called $CRR_{7.5}$. To take different magnitudes into account, the factor of safety against liquefaction is multiplied by a magnitude scaling factor (MSF). In the graphical output, the CSR is divided by the MSF to give an accurate view of the liquefied zone.

The computation of $CRR_{7.5}$ from SPT is described below. The BPT data is merely converted to SPT before following the SPT procedure to determine $CRR_{7.5}$.

1. **Step 1 - Correction of SPT Blow Count Data:**

   Because of their variability, sensitivity to test procedure, and uncertainty, SPT N-values have the potential to provide misleading assessments of liquefaction hazard, if the tests are not performed carefully. The engineer who wants to utilize the results of SPT N-values to estimate liquefaction potential should become familiar with the details of SPT sampling as given in ASTM D-1586 (ASTM, 1998) in order to avoid some of the major sources of error.

   The procedures that relate SPT N-values to liquefaction resistance use an SPT blow count that is normalized to an effective overburden pressure of 100 KPa (or 1.044 tons per square foot). This normalized SPT blow count is denoted as N_1, which is obtained by multiplying the uncorrected SPT blow count by a depth correction factor, Cn (*SP117*, 1999)[18].

2. **Step 2 - Fines Content Correction of SPT and CPT Data:**

   The CRR curves used in LiquefyPro are based on clean sand. To use these curves for soil containing fines such as silt and clay, the blow count data must be corrected for the fines content. Simplistically, one could say that a soil containing fines is more liquefaction resistant than a “clean” soil. Thus the blow count should be increased for the soil containing fines, which would increase its liquefaction resistance. The curve of Figure (7) shows the relationship between cyclic stress or resistance ratio and $(N_1)_{60}$ values for magnitude 7.5 of earthquakes. The curve is intended to divide zones corresponding to liquefaction and non-liquefaction. If a point plots above the curve, the site would be judged susceptible to liquefaction. If the point plots below the curve with an adequate margin of safety, the site is judged to be safe.

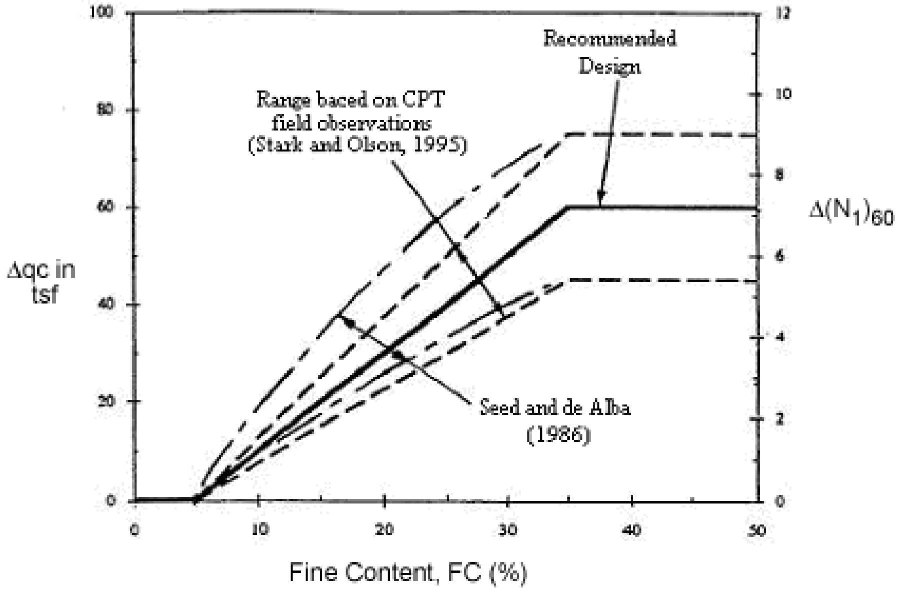


Fig. 7. SPT and CPT fines content correction factors (Seed, 1996) [14].

The fines content correction can be done with either one of the four options. The option can be chosen on the advanced input page in LiquefyPro. The second option (Seed, 1996)[14] used to correct fines content in the chosen site.

The fines content correction formulas below were developed by Seed (1996)[14]. This option is available only for SPT input and shown in Figure (8) (curve section at fines = 0 to 35%).

$(N_1)_{60f} = \alpha + \beta (N_1)_{60}$

= 0; = 1.0 for FC 5%

= $\exp[1.76 - (190/FC^2)]$; = $0.99 + FC^{1.5}/1000$ for 5 < FC < 35%

= 5.0 ; = 1.2 for FC 35%

where (N1)60f is the corrected blow count.

FC is the fines content in %.

3. Step 3 - Calculation of $CRR_{7.5}$

$CRR_{7.5}$ (Magnitude = 7.5) is determined using the formula below (Blake, 1997)[2].

$$CRR_{7.5} = \frac{a + c\cdot\chi + e\cdot\chi^2 + g\cdot\chi^3}{1 + b\cdot\chi + d\cdot\chi^2 + f\cdot\chi^3 + h\cdot\chi^4} \tag{6}$$

where,

$\chi = (N_1)_{60f}$

a = 0.048 , b = -0.1248 , c = -0.004721 , d = 0.009578 ,

e = 0.0006136 , f = -0.0003285 , g = $-1.673\cdot10^{-5}$, h = $3.714\cdot10^{-6}$

Factor of Safety as Ratio of CRR/CSR :

1. fs - User requested factor of safety :

A preliminary factor of safety can be applied to the CSR value in the program:

$$CSR_{fs} = CSR \cdot fs \tag{7}$$

where CSRfs – increased cyclic stress ratio (CSR) with user requested factor of safety. fs – user-requested factor of safety.

A typical value of *fs* is 1.2. The larger the *fs*, the larger the CSR_{fs} and the more conservative of the liquefaction analysis. The selection of factor of safety also influences the settlement calculation as the CSR_{fs} value is used in the analysis.

2. **FS - Ratio of CRR/CSR :**

The ratio of CRR/CSR is defined as factor of safety for liquefaction potential:

$$F.S. = CRR / CSR_{fs} \tag{8}$$

F.S. is ultimate result of the liquefaction analysis.

If *F.S.* ≥ to 1, there is no potential of liquefaction; If F.S. < 1, there is a potential of liquefaction.

F.S. is different from fs, which is a user-defined value for increasing the value of CSR in order to provide a conservative liquefaction analysis.

Both *CRR* and *CSRfs* are limited to 2 and *F.S.* is limited to 5 in the program.

Settlement Calculations

LiquefyPro divides the soil deposit into very thin layers and calculates the settlement for each layer. The calculations are divided into two parts, dry soil settlement and saturated soil settlement. The soil above the groundwater table is referred to as dry soil and soil below the groundwater table is referred to as saturated soil. The total settlement at a certain depth is the sum of the settlements of the saturated and dry soil. The total settlement is presented in the graphical report as a cumulative settlement curve versus depth. LiquefyPro gives settlement in both liquefied and non-liquefied zones.

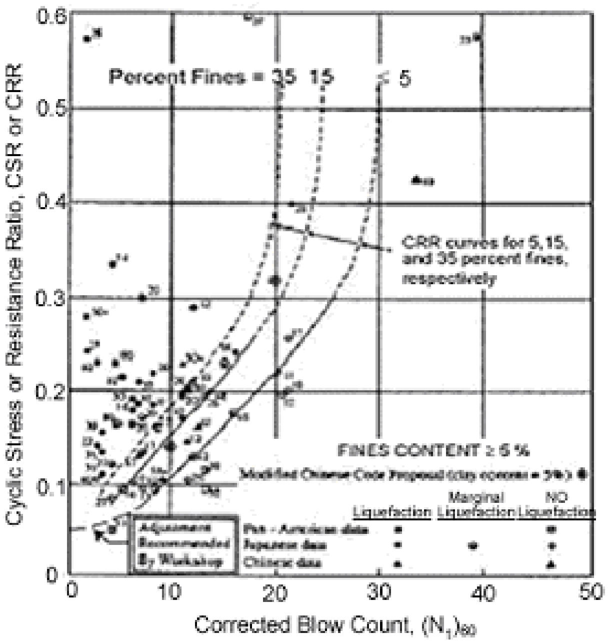


Fig. 8. Simplified base curve recommended for calculation of CRR from SPT data along with empirical liquefaction data (modified from Seed *et al*., 1985[16]).

Applications of the Program

In this section, the computer program LiquefyPro will be used to investigate the liquefaction potential of two sites. The input data of the program will be based on those derived from the compression and shear wave velocities.

Case No.1 (Al-Safeer Hotel):

The site of (Al-Safeer Hotel, B5) is chosen to check for liquefaction potential. The object of the investigation of the site is:

1. To assess the condition of subsurface soils underneath the hotel building which was built in 1955 and to define the cause of the structure movement.
2. To locate cavities and weak zones within the subsurface soils underneath the building.
3. To define the allowable bearing capacity of subsurface soils.

Below are the results of the application of the program:

Input Data:

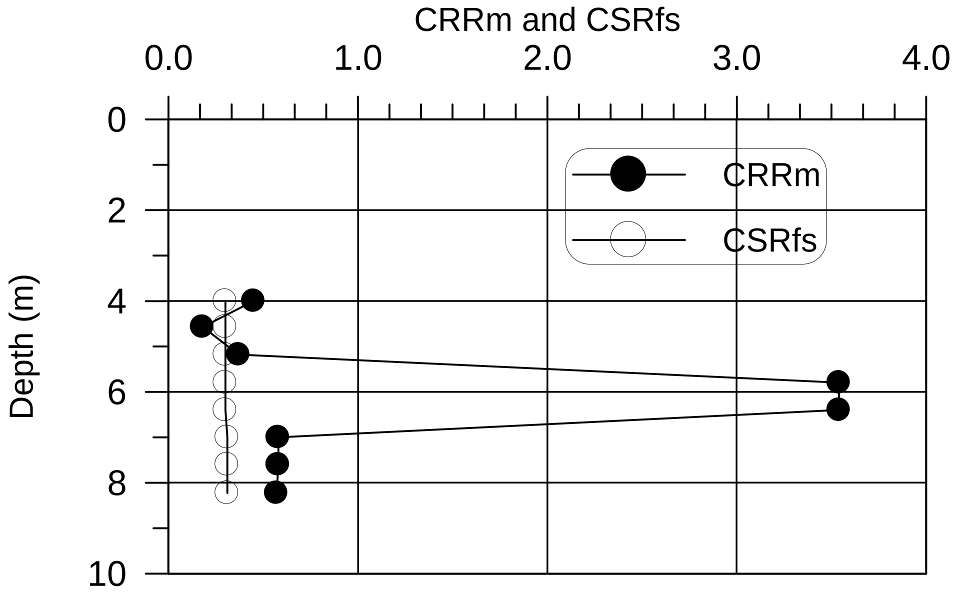
Depth (m)	SPT	(kN/m^3)	Fines %	Classification
4	11	17.7	75	CL
4.5	2	18.7	94	CL
4.9	5	19.1	97	CL
6	19	17.6	95	CL
6.5	17	18.5	85	CL
7	16	18.3	92	CL
8	17	18.3	80	CL

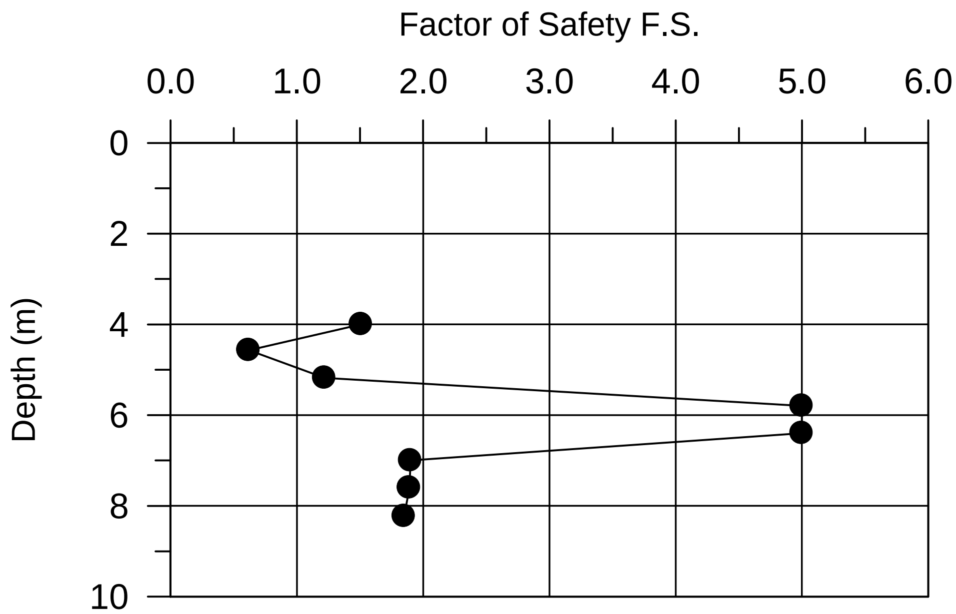
Output Results:

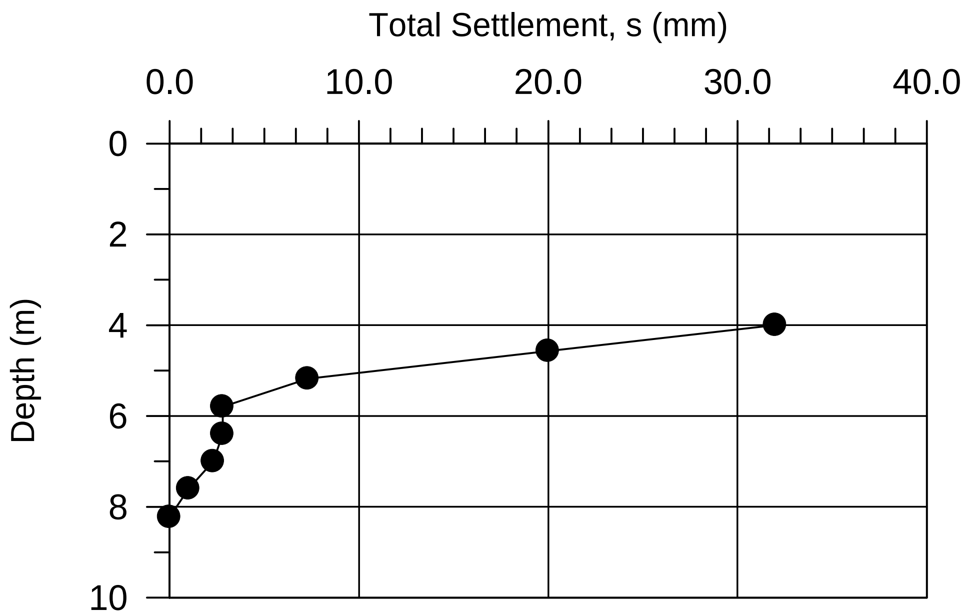
Settlement of saturated sands = 32 mm.
Settlement of dry sands = 0.00 mm.
Total settlement of saturated and dry sands = 32 mm.
Differential Settlement=16.2 to 21.2 mm.

Depth (m)	CRRm	CSRfs w/fs	F.S.= CRRm/CSRfs	S sat. mm	S dry. mm	S total. mm
4	0.45	0.3	1.51	32	0	32
4.57	0.18	0.3	0.62*	20	0	20
5.18	0.37	0.3	1.22	7.3	0	7.3
5.8	3.54	0.3	5	2.8	0	2.8
6.4	3.54	0.3	5	2.8	0	2.8
7	0.58	0.31	1.9	2.3	0	2.3
7.6	0.58	0.31	1.89	1	0	1
8.23	0.57	0.31	1.85	0	0	0

* (F.S. is limited to 5, CRR is limited to 2, CSR is limited to 2)
F.S.<1, Liquefaction Potential Zone


CRRm and CSRfs
0.0
1.0
2.0
3.0
4.0
0
2
4
6
8
10
Depth (m)
CRRm
CSRfs


Factor of Safety F.S.
0.0
1.0
2.0
3.0
4.0
5.0
6.0
0
2
4
6
8
10
Depth (m)


Total Settlement, s (mm)
0.0
10.0
20.0
30.0
40.0
0
2
4
6
8
10
Depth (m)

From above the following points are noticed:

1. A liquefaction potential zone appears near a depth of 4.5 m (F.S.<1), the main factor causes this result during calculation is S.P.T. N-value is equal to (2) at 4.5 m depth and (5) at 4.9 m depth. The low value of N is due to the high value of fine content.
2. The high values of fine content, low values of both S.P.T N-value and saturated unit weight, and the existence of cavities and weak zones, all these reasons caused low values of shear wave velocity in the site, and the values are between (152-371) m/sec.
3. The calculated settlement of the site by the program coincides with the measured settlement in the site.

Case No. 2 (Salah-Aldeen Thermal Power Station):

The site (Salah-Aldeen Thermal Power Station, B14) is located at the south of Samarra city. The topography of the site is slightly irregular.

Samarra area is located about 90 km, north west of Baghdad city, which is about 60-70 m above sea level on the left bank of Tigris river. Different geomorphologic shapes like slopes, valleys and plains are surrounding Samarra area.

The geological age of this area was at the Quaternary period, which is represented as Pleistocene alluvial deposit. The site in general is covered with recent deposits of river.

The results of the program for this case are given below:

Input Data:

Depth (m)	SPT	(kN/m^3)	Fines %
1	240	19.5	13
3.5	200	19.6	36
7.5	600	19.4	13
11	24	20	24

Output Results:

Settlement of saturated sands = 0.00 mm.

Settlement of dry sands = 0.00 mm.

Total settlement of saturated and dry sands = 0.00 mm.

Differential Settlement = 0.000 to 0.000 mm.

Depth (m)	CRRm	CSRfs w/fs	F.S.= CRRm/CSRfs	S_sat. mm	S_dry. mm	S_total. mm
1	3.54	0.2	5	0	0	0
2.5	3.54	0.6	5	0	0	0
4	3.54	0.27	5	0	0	0
5.57	3.54	0.28	5	0	0	0
7.1	3.54	0.28	5	0	0	0
8.63	3.54	0.28	5	0	0	0
10.1	3.54	0.28	5	0	0	0

* F.S.<1, Liquefaction Potential Zone

(F.S. is limited to 5, CRR is limited to 2, CSR is limited to 2)

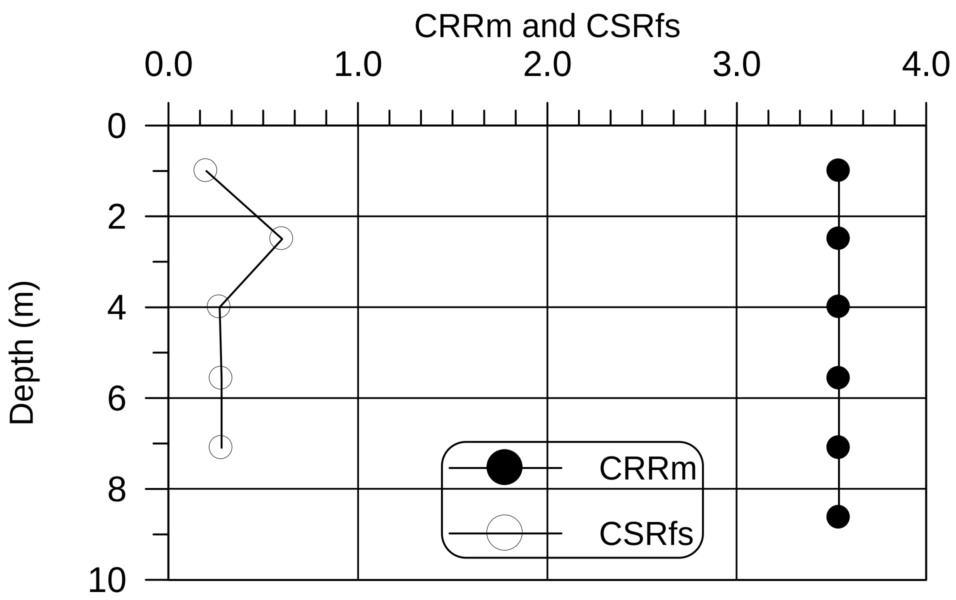
CRRm Cyclic resistance ratio from soils

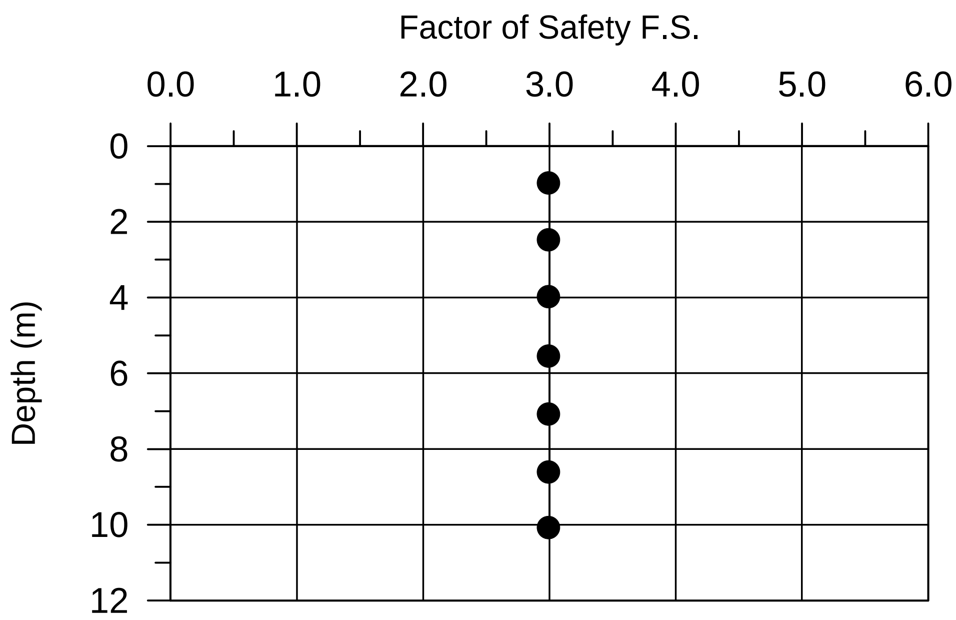
CSRfs Cyclic stress ratio induced by a given earthquake (with user request factor of safety)

F.S.	Factor of Safety against liquefaction, F.S.=CRRm/CSRfs
S_sat	Settlement from saturated sands
S_dry	Settlement from dry sands
S_total	Total settlement from saturated and dry sands
NoLiq	No-Liquefy Soils

From above the following points are noticed:

1. The high value of both γ_{sat} (reach 20 kN/m^3) and S.P.T (N-value), (reach 600 blows) make the soil of site B14 (Salah Al-Deen Thermal Power Station) difficult to liquefy.
2. No settlement occurs in this site.
3. The site has a high value of both V_p (reach 2166 m/s) and V_s (reach 1000 m/s).





7. Conclusions

Concluding remarks may be summarized as follows:

1. Direct relations have been obtained between V_p and V_s with the standard penetration test *N*-values. The relations take the form:

$$N = 61.22 \cdot (1 - e^{(-0.00137 \cdot V_p)})$$

$$N = 88.54 \cdot V_s^{(-40.7/V_s)}$$

2. Soil containing fines (clays and silts) are more resistant to liquefaction than sandy soils, thus the blow counts (*N*-values) increase for soils containing fines, and hence increase its liquefaction resistance. No liquefaction potential zones have been indicated deeper than 15 m as the liquefaction resistance of a soil deposits increases with depth as the effective overburden pressure does.
3. Although settlement is most commonly observed in liquefied zones, but it is also indicated in non–liquefied zones. The estimated total settlements range between 1 and 32 mm. This site also shows low values of SPT and low liquefaction resistance.

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