International Journal of Computational and Experimental Science and ENgineering (IJCESEN)
Vol. 6-No.3 (2020) pp. 180-188
http://dergipark.org.tr/en/pub/ijcesen

ISSN: 2149-9144

Research Article

Determination of Liquefaction Resistance and Allowable Bearing Capacity of Soils Based on V_S (Shear wave) velocity; Case Study: Isparta Süleyman Demirel Industrial Region Waste Treatment Facility

Emre TİMUR, Coşkun SARI*

Dokuz Eylül University, Engineering Faculty, Department of Geophysical Engineering, İzmir-Turkey

* **Corresponding Author** : coskun.sari@deu.edu.tr
ORCID: 0000-0002-0192-9300

Article Info:

DOI: 10.22399/ijcesen.744183
Received : 28 May 2020
Accepted : 17 November 2020

Keywords

Bearing capacity
Liquefaction analysis
Seismic refraction
Isparta

Abstract:

It was a very common procedure to investigate liquefaction risk with standard penetration test (SPT). However, this method has been lost its importance after the developments of conic penetration method in 1971, Becker penetration method and S-wave velocity measurements. S-wave velocity measurements could be very reasonable alternatives in order to carry out penetration tests for the gravelly and unconsolidated overburden soil investigations. In this study, S-wave velocity values were used in order to determine liquefaction resistance and allowable bearing capacity of soil where two different methods were applied to S-wave velocity values and the results were also compared. All the application steps of the methods were defined. Data were collected along 4 profiles for the ground investigations carried out for the Isparta Süleyman Demirel Industrial Region Waste Treatment Facility.

1. Introduction

Soil liquefaction defines an event where a saturated or partially saturated soil layer suddenly loses strength in response to an applied stress. Generally the reason of the stress is a earthquake shaking which forces the layer to behave like a liquid. This behaviour of the ground causes the buildings to lean to one side or a total collapse. Consequently liquefaction analyses of the shallow layers are very important on ground investigations.

Determination of the liquefaction resistance of layers is an important feature in geotechnical explorations especially in seismically risky areas. First procedure was developed by [1] using the data from the standard penetration test (SPT) correlated with a parameter called the cyclic stress ratio. This procedure has been revised and updated by several geotechnicians in time [2, 3, 4, 5, 6, 7]. Another method based on the cone penetration test (CPT) was developed by [8] in 1985 which also has been examined and updated [9, 10, 11, 12, 13].

The use of S-wave (V_S) velocity as an index of liquefaction resistance is soundly based because both S-wave velocity liquefaction resistances are similarly influenced by many factors. Some advantages of using S-wave velocity [14, 15, 16, 17] are that (1) the measurements are possible in soils that are hard to sample, such as gravelly soils where penetration tests may be unreliable; (2) measurements can also be performed on small laboratory speciments, allowing direct comparisons between laboratory and field behaviour; (3) S-wave velocity is a basic mechanical property of soil materials, directly related to small stress-strain shear modulus G_{max} given by $G_{max}=\rho V_S^2$ where ρ is the mass density of soil, V_S is the S-wave velocity; (4) G_{max} or V_S is normally a required property in earthquake site response and soil-structure interaction analyses; and (5) S-wave velocity can be measured by the spectral analysis of surface waves (SASW) or multichannel analysis of surface waves (MASW) techniques at sites where borings may not be permitted.

Three concerns when using S-wave velocity to evaluate liquefaction resistance are that (1) no samples are routinely obtained as a part of the testing procedure for soil classification and identification of nonliquefiable materials; (2) thin, low S-wave velocity strata may not be detected if the measurement interval is too large; and (3) measurements are made at small strains, whereas pore water pressure buildup and liquefaction are medium to high strain phenomena [18, 19, 20].

Secondly, the ultimate bearing capacity of a particular soil, under a shallow footing, was investigated theoretically by Prandtl [21] and Reissner [22] using the concept of plastic equilibrium as early as in 1921. The formulation however is slightly modified, generalized, and updated later by Meyerhof [23], Hansen [24], De Beer [25], and Sieffert et al. [26].

S-wave velocity surveys, represent the actual ground conditions, are much more efficient and reliable than the shear strength parameters measured in laboratory. In addition to seismic refraction survey, there are several other techniques of measuring V_S at the investigation site as defined by Stokoe and Woods [27] and updated by Tezcan et al. [28]. The reason is in-situ measured S-wave velocity indicates the actual unchanged condition of the soil layers. V_S also enables the observer to determine the allowable bearing capacity (q_a), which is also an important parameter for defining ground conditions, in a reliable way.

In this study, it was intended to observe the ground condition using seismic refraction surveys in Isparta/Turkey. Data were collected along 4 profiles and procedures of liquefaction analysis and allowable bearing capacity were applied in order to determine risky soil layers.

2. Evaluation Procedure of Liquefaction Analysis

The evaluation procedure of determination of liquefaction resistance requires the calculation of three parameters; (1) The level of cyclic loading on the soil caused by the earthquake, expressed as a cyclic stress ratio (CSR) [1, 2] (Figure 1); (2) stiffness of the soil, expressed as an overburden stress-corrected shear wave velocity [3, 29, 30, 31]; and (3) resistance of the soil to liquefaction, expressed as a cyclic resistance ratio (CRR) [1, 3, 32, 33].

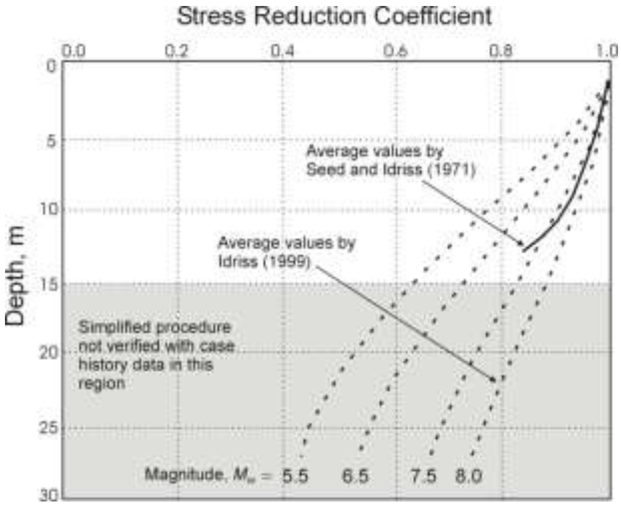


Figure 1. *Shear Stress Reduction Factor used to adjust for flexibility in soil profiles during earthquake shaking [2].*

The correction factor (K_c) is needed for high values of V_{S1} caused by cementation and aging. Figure 2 illustrates a method for estimating the value of K_c by using SPT blow counts. Figure 2a indicates the V_{S1}-$(N_1)_{60}$ correlation between silty sands implied by the recommended CRR-V_{S1} curves and CRR-$(N_1)_{60}$ curves [34]. The theoretical curves presented in Figure 2b were improved from them [3].

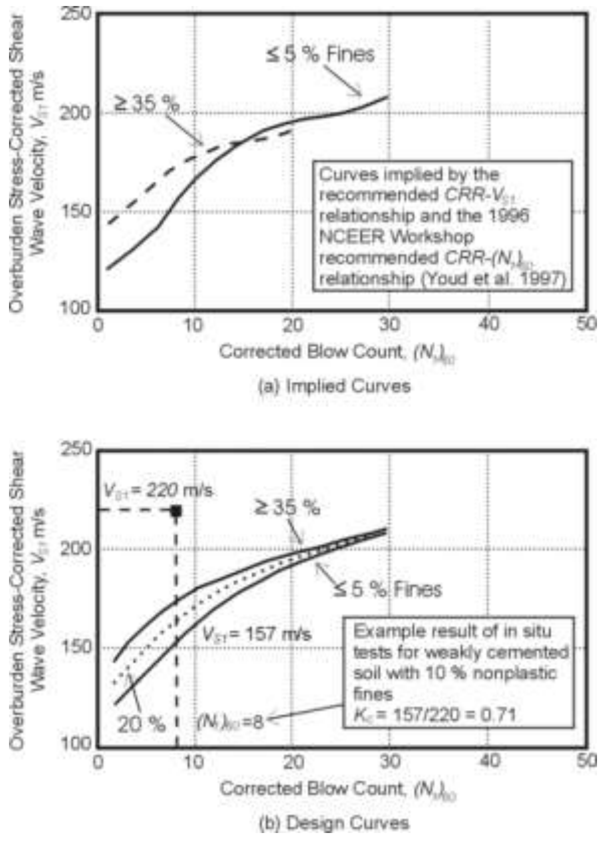


Figure 2. *Correlations between V_{S1} and $(N_1)_{60}$ and an example for determining correction factor K_c [3].*

In soils above the ground-water table, especially silty soils, negative pore pressures increase the effective state of stress and this effect should be considered in the estimation of $\sigma_V^{'}$ for correcting V_S and V_{S1}. The entire procedure is generally summarized in several steps in [3].

3. Evaluation Procedure of Allowable Bearing Capacity

The allowable bearing capacity, considering the limits for the parameters [35], can be calculated from the statements proposed by [34]. The change in allowable bearing capacity q_a, with respect to S-wave velocity V_S, is presented in Figure 3.

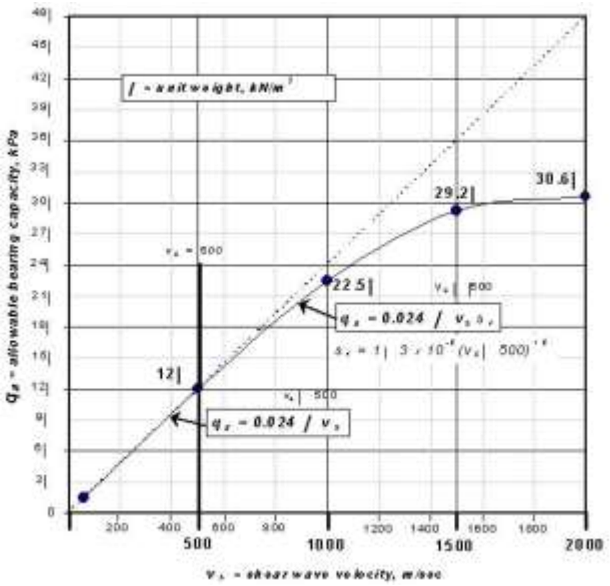


Figure 3. Allowable bearing capacity of soils based on S-wave velocity [41].

The P-wave velocity of a layer is influenced directly from the average unit weight γ. According to previous studies on this relationship, a corrected unit weight should be calculated by adding the effect of V_P [34, 36]. Also it can be clearly seen that the calculated unit weights are in consistence with the values measured in the laboratory (Figure 4).

4. Location and Geology of the Field

The survey area is located in the intersection point of the western and middle Taurus Mountains, near the Lakes (Göller) District of Turkey (Figure 5). The study area is located in the Gümüşgün resort in city of Isparta, on the crossroad of Ankara, Antalya and İzmir. It is 26 km from Isparta and 4 km from the local airport. It is planned to refine 4000-8000 m^3 contaminated water each day in the purification site.

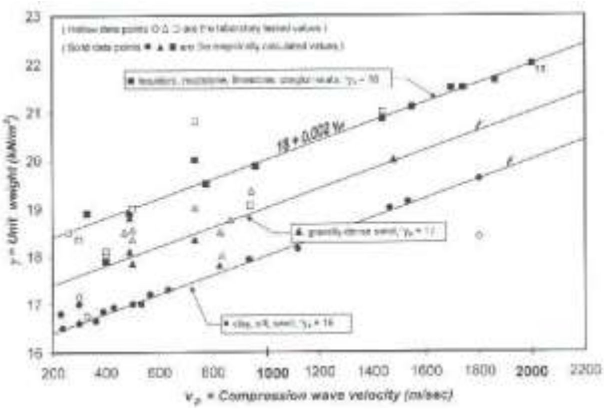


***Figure 4.** Unit weights based on S-wave velocities [34].*

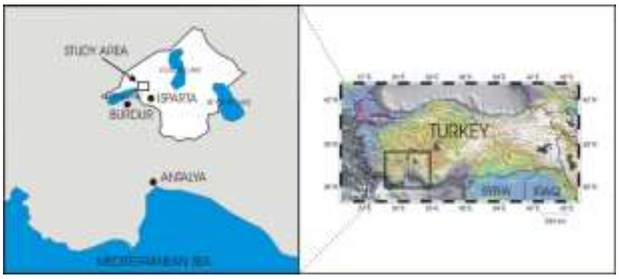


***Figure 5.** Location of the survey area.*

This area is defined by the rocks of Miocene, Pliocene and Pleistocene in age at the higher reliefs. High altitude areas, where this system exists, are broken apart in several levels by the faults and rivers. Water drainage system of the area was usually presented in the Pliocene. This system which is well adjusted with the orogenic and structural features and connected with the closed basin is partially preserved nearby the northern and north-western part (Göller Region) of the survey area (Figure 6).

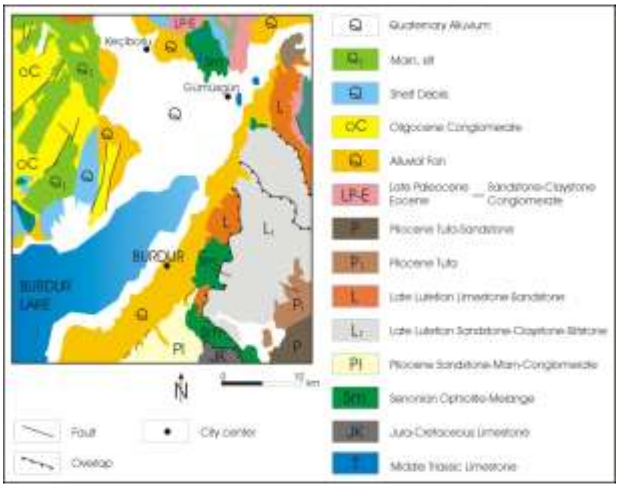


***Figure 6.** Geological map of the survey area.*

5. The Principles of Seismic Near-Surface Investigations

It is hereby the traditional seismic refraction technique was applied for the geotechnical engineering purposes. Shallow refraction technique is considered as one of the most effective method, which can be used, for the engineering purposes. Determining depth to the bedrock, the depth to groundwater, types of lithology, the lateral and vertical changes in lithology and investigating structural features such as micro faults and cracks are the main targets of shallow refraction applications.

Traditional interpretation of seismic refraction data has used a concept of layered horizons or zones where each horizon has a discrete seismic velocity. Interpretation methods based on the refraction of the first portion of the seismic wave have been known for many years [37]. The advent of hand-held calculators in the 1970's and personal computers by the 1980's, as well as the development of practical seismographs for civil engineering use, has made seismic refraction a practical geotechnical exploration tool for more than two decades.

The seismic refraction method consists of measuring (at known points along the surface of the ground) the travel times of seismic waves either P- or S-wave velocities generated by an impulsive energy source. The energy is detected, amplified and recorded by special equipments (seismographs).

At the seismic refraction study, data acquisition was designed to make effective use of the advanced data processing techniques. The seismic field data were collected using a 12 channel named PASI (Italy) engineering seismograph for refraction investigation. Each seismic refraction spread (profile) consists of a series of 12 channel geophones placed along the line at a set distance or geophone interval. 14 Hz geophones were used. The average shot spacing (a sledge hammer of 8 kg as a surface impact source) was about 1-10 meter length. Generally, for shallow depth investigation the sledgehammer is suitable because it is easy to operate, cheap, highly portable and safe. In case of soft ground, an impact plate firmly embedded in the ground is usually used. The locations of the refraction lines are displayed in Figure 7. The geophones were spaced at 3 m interval with a 3 m nearest offset from the source.

6. Application of Liquefaction Resistance Analysis

The procedure of liquefaction resistance analysis is given by [3] as below:

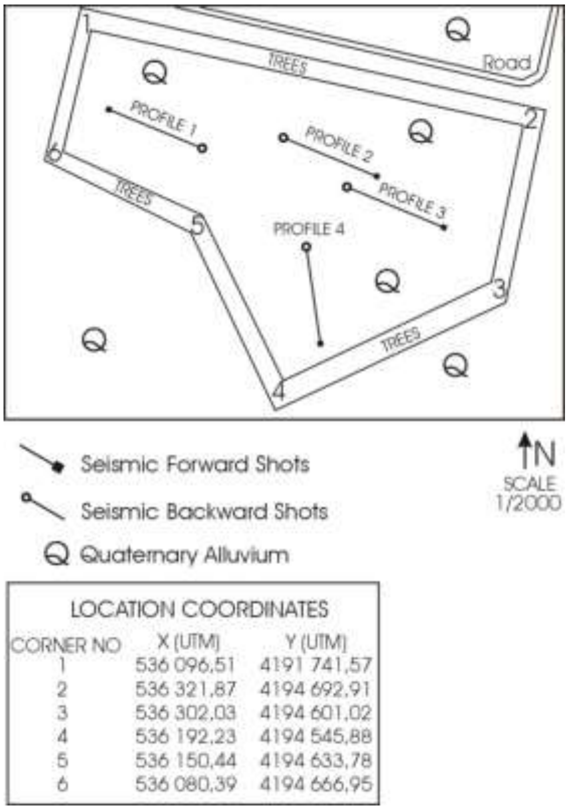


Figure 7. *Locations of the seismic profiles and drillings.*

1. Determination of S-wave velocity, fines content, densities (Table 1) and penetration resistance (if possible). Densities were calculated with the equation of Gardner et al. [38] $\rho = 0.31V_p^{0.25}$.

Table 1. *S-wave velocities, fines contents and densities of layers*

		V_S (m/sn)	Fines Content	ρ gr/cm³
Profile 1	1. Layer	121,24	FC ≥ 35%	1,30
	2. Layer	222,27	FC ≥ 35%	1,48
	3. Layer	425,35	FC ≥ 35%	1,69
Profile 2	1. Layer	134,56	FC ≥ 35%	1,26
	2. Layer	231,08	FC ≥ 35%	1,46
	3. Layer	464,74	FC ≥ 35%	1,82
Profile 3	1. Layer	134,04	FC ≥ 35%	1,22
	2. Layer	249,62	FC ≥ 35%	1,45
	3. Layer	347,71	FC ≥ 35%	1,78
Profile 4	1. Layer	113,8	FC ≥ 35%	1,26
	2. Layer	251,58	FC ≥ 35%	1,45
	3. Layer	498,53	FC ≥ 35%	1,80

2. Determination of water table, besides nothing of seasonal differences and pressures of artesian wells. As a result of insufficient data of water table, water level is assumed to be at the surface.
3. Determination of σ_v in all depths from the seismic data.

$$\sigma_V = \gamma . z$$
$$\sigma_V^{'} = \sigma_V - u \qquad (1)$$
$$u = \gamma_d . z$$

where z is the liquefaction depth, γ is the density, u is the pore water pressure, $\sigma_V^{'}$ is the initial effective vertical (overburden) stress at the depth in question, σ_V is the total overburden stress at the same depth [39]. Vertical stresses can be calculated from these equations (Table 2). However it could also be better to use the penetration tests to achieve better results, if possible.

Table 2. *Calculated vertical stresses.*

		σ_V (kg/cm²)	$\sigma_V^{'}$ (kg/cm²)
Profile 1	1. Layer	0,239	0,101
	2. Layer	1,065	0,444
	3. Layer	1,858	0,785
Profile 2	1. Layer	0,298	0,126
	2. Layer	1,260	0,53
	3. Layer	2,002	0,844
Profile 3	1. Layer	0,169	0,076
	2. Layer	1,001	0,451
	3. Layer	1,942	0,884
Profile 4	1. Layer	0,184	0,078
	2. Layer	1,045	0,456
	3. Layer	1,942	0,86

4. Correction of S-wave velocity measurements with C_V stress parameter. This parameter is accepted as 1.4 for surface layers.

$$V_{S1} = V_S C_V = V_S \left(\frac{P_a}{\sigma_V^{'}} \right)^{0.25} \qquad (2)$$

where $P_a = 100\,kPa$
for profile 1 and layer 2,
$\sigma_V^{'}$ = 1.065 kg/cm² = 104.4408 kPa
V_{S1} = 222.27x(100/104.4408)$^{0.25}$ = 219.87 m/s
All of the values are calculated in Table 3.

5. Determination of V_{S1}^{*} according to fines content. If fines content is not known than it is assumed to be 215 m/s. Fines content is greater than 35% in all layers (FC ≥ 35%). Thus V_{S1}^{*} = 200 m/s.

Table 3. *C_V and V_{S1} values.*

		V_S	σ_V (kg/cm²)	$\sigma_V^{'}$ (kPa)	C_V	V_{S1}
Profile 1	1. Layer	121,2	0,23	23,43	1,4	169,7
	2. Layer	222,2	1,06	104.44	0,99	219,8
	3. Layer	425,3	1,85	182,20	0,9	382,8
Profile 2	1. Layer	134,5	0,29	29,22	1,4	188,3
	2. Layer	231,0	1,26	123,56	0,94	219,1
	3. Layer	464,7	2,00	196,32	0,9	418,2
Profile 3	1. Layer	134,0	0,16	16,57	1,4	187,6
	2. Layer	249,6	1,00	98,16	1,0	250,7
	3. Layer	347,7	1,94	190,44	0,9	312,9
Profile 4	1. Layer	113,8	0,18	18,04	1,4	159,3
	2. Layer	251,5	1,04	102,47	0,99	250,0
	3. Layer	498,5	1,94	190,44	0,9	448,6

6. Determination of K_c. If the soil is un-cemented and <10.000 years old, than it is 1. If the condition is not known than it is 0.6 (Table 4).

   If there is an abnormal increase in V_S of the third layer it makes appropriate to apply K_c correction. Also it is known that the bottom layers are of the Pliocene.

Table 4. *K_c, age corrected velocities and CRR values*

		K_C	V_{S1age}	CRR
Profile 1	1. Layer	-	-	-
	2. Layer	0,8	175,896	0,143183
	3. Layer	0,7	267,9705	0,117173
Profile 2	1. Layer	-	-	-
	2. Layer	0,75	164,3807	0,141423
	3. Layer	0,7	292,7862	0,164634
Profile 3	1. Layer	-	-	-
	2. Layer	0,75	188,084	0,340638
	3. Layer	0,75	234,7043	0,030219
Profile 4	1. Layer	-	-	-
	2. Layer	0,75	187,5332	0,328283
	3. Layer	0,65	291,6401	0,162523

7. Determination of earthquake plan and estimation of a_{max}.

   Although a_{max} is not known it is possible to estimate a value from Esteva's maximum horizontal ground surface acceleration diagram (Figure 8).

   It is assumed that the magnitude is 7.0, and also the seismic risk is assumed as maximum so epicenter distance is taken 0.

   So a_{max}=532.2 cm/s².

a_{max} = 532,2 cm/s^2 = 5,322 m/s^2 = 0,543 g $\Rightarrow$ M_w = 7.0

8. Determination of CSR of all layers under the water table.

r_d can be calculated from the figure of [1] (Figure 1).

CSR = $0.65(\alpha_{max}/g)(\sigma_v/\sigma_v')r_d$

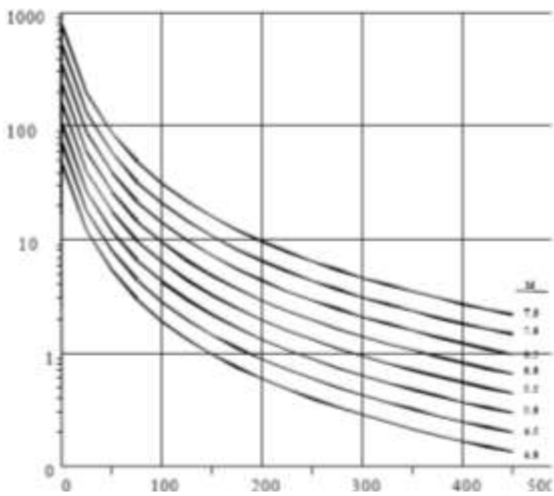
As an example for Profile 1 and 1. Layer; for 1.38 m r_d=0.98 (from Figure 8).

CSR = 0.65(5.322/9.81)(0.239/0.101)0.98 = 0,8185 can be found. Table 5 indicates all the profiles and layers.

Table 5. *r_d and CSR values.*

		r_d	CSR
Profile 1	1. Layer	0,98	0,8185
	2. Layer	0,90	0,7619
	3. Layer	0,80	0,6683
Profile 2	1. Layer	0,96	0,8014
	2. Layer	0,88	0,7384
	3. Layer	0,76	0,6363
Profile 3	1. Layer	0,99	0,7770
	2. Layer	0,91	0,7129
	3. Layer	0,79	0,6125
Profile 4	1. Layer	0,97	0,8076
	2. Layer	0,89	0,7199
	3. Layer	0,78	0,6217

Figure 8. *Esteva's maximum acceleration diagram for*



epicenter distances [40].

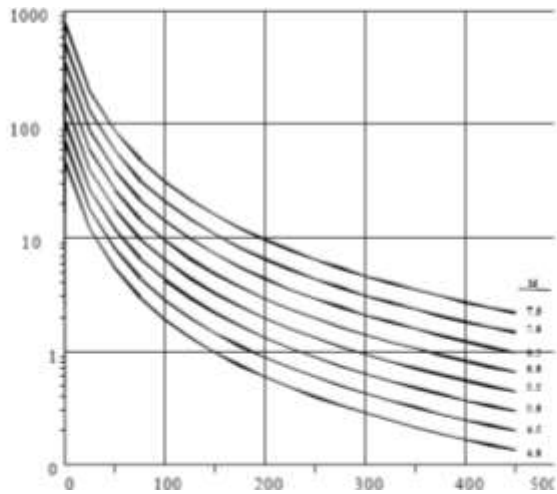
9. Calculating (Table 6) and plotting V_{S1} and CRR liquefaction resistance curves (Figure 9).

Table 6. *CRR values.*

		V_S	V_{S1}	V_{S1age}	CRR
Profile 1	1. Layer	121,24	169,736	No Correction	0,161768
	2. Layer	222,27	219,87	175,896	0,143183
	3. Layer	425,35	382,815	267,9705	0,117173
Profile 2	1. Layer	134,56	188,384	No Correction	0,347839
	2. Layer	231,08	219,1743	164,3807	0,141423
	3. Layer	464,74	418,266	292,7862	0,164634
Profile 3	1. Layer	134,04	187,656	No Correction	0,330946
	2. Layer	249,62	250,7787	188,084	0,340638
	3. Layer	347,71	312,939	234,7043	0,030219
Profile 4	1. Layer	113,8	159,32	No Correction	0,126166
	2. Layer	251,58	250,0442	187,5332	0,328283
	3. Layer	498,53	448,677	291,6401	0,162523

10. Determination of *FS*. If *FS* > 1 than there is no liquefaction, if *FS* ≤ 1 than there is liquefaction.

FS is less than 1 in first layers of profile 1 and 4, and second layer of Profile 2. So liquefaction risk is unimportant.



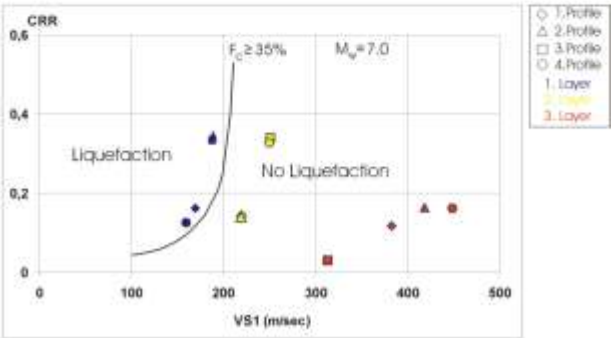


Figure 9. *Liquefaction result curve for the study field.*

7. Application of Bearing Capacity Analysis

The allowable bearing capacity, q_a, under a shallow foundation in units of kPa, may be obtained from the following empirical expressions:

$$q_a = 0.024\gamma V_s$$
$$q_a = 2.4(10^{-4})\rho V_s \qquad (3)$$

where γ is the unit weight (kN/m^3), ρ is the mass density (kg/m3), and V_S is the shear(S-) wave velocity (m/sec). There is a direct relationship between the average unit weight γ, and the P-wave velocity of a soil layer. Based on extensive case histories of laboratory testing, a convenient empirical relationship in this regard, is proposed in detail by [36] as follows;

$$\gamma_p = \gamma_0 + 0.002V_p \qquad (4)$$

P- and S- wave velocity values of the media were obtained from the refraction seismic data of 4 profiles carried out at the survey area using the 12 channel PASI (Italy) seismic refraction equipment. Seismic velocities vary as 235-311 m/s, 477-532 m/s and 893-1184 m/s for the top, middle and bottom layers respectively for the P-wave, whereas these vary as 113-134 m/s, 222-251 m/s and 347-498 m/s for the S-wave. All the layers have very close velocity values within themselves. This signifies that these layers have homogeneous lithologies. Equations 3 and 4 were used in order to calculate the unit weights and allowable bearing capacities of the layers in the survey area. Obtained results were given in Table 7. Since S-wave velocity is less than 500 m/s, equation 3 was used for the allowable bearing capacity calculations. The graphics of the obtained allowable bearing capacity, q_a – S-wave velocity, S-wave velocities were given in Figure 10. As it was observed in the q_a - S-wave velocity profile, the allowable bearing capacity q_a Show s linear variation with the shear wave velocity, S-wave velocity and this is increased with depth.

Table 7. *Unit weights and allowable bearing capacities of the layers in the survey area.*

		V_S (m/sec)	V_P (m/sec)	*Unit Weight (kN/m^3)*	*Allowable Bearing Capacity, (kPA)*
Profile 1	1. Layer	121	311	16.62	48.27
	2. Layer	222	532	17.06	90.91
	3. Layer	425	893	17.78	181.41
Profile 2	1. Layer	135	276.5	16.55	53.63
	2. Layer	231	497	16.99	94.21
	3. Layer	465	1185	18.3	205.00
Profile 3	1. Layer	134	235	16.47	52.96
	2. Layer	250	478	16.95	101.73
	3. Layer	348	1088	18.17	151.80
Profile 4	1. Layer	114	276	16.55	45.28
	2. Layer	252	479	16.95	102.56
	3. Layer	499	1143	18.28	218.99

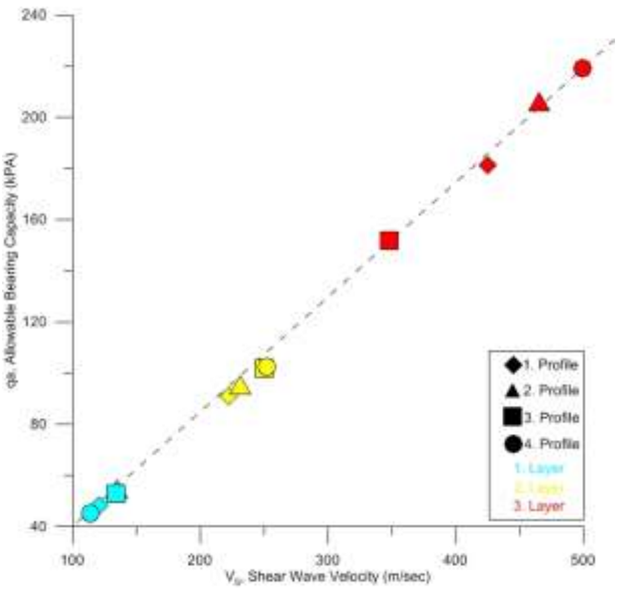


Figure 10. *Allowable bearing capacity of soils based on S-wave velocity.*

8. Conclusion

In-situ seismic refraction studies were carried out in order to determine liquefaction risk in Isparta Industrial Region Waste Treatment Facility. Seismic P-wave and S-wave velocities were defined to calculate liquefaction resistance and allowable bearing capacity parameters. According to Figure 9, it was observed that all of the first layers of the refraction models have liquefaction risk. Besides the variation of S-wave values between 114-135 m/sec can be concerned with the relationship between liquefaction risk and S-wave velocity values. Climatic changes and annual average rain amounts should also be considered while determining the liquefaction risk of these first layers. Although a measurement value is in the liquefaction zone on the Nomogram, it was determined that there is no liquefaction risk if CRR value is more than 0.2 in silty and clayed medium. As a result, there is no liquefaction risk for the data which are in the liquefaction zone and have a CRR value greater than 0.2 at the same time in Figure 9. Also first layers of profiles 1 and 4 have less allowable bearing capacity than other two profiles' first layers. So it can be said that allowable bearing capacity and liquefaction analyse result support each other. Risky areas should be considered during the construction of the buildings by choosing appropriate foundation type and possible excavation areas.

Acknowlodgements

The authors are grateful to Sumet Ltd. Co. in İzmir for permission of the seismic refraction data.

References

[1] Seed H.B., I.M.Idriss "Simplified procedure for evaluating soil liquefaction potential" J. Soil Mech. and Found. Div. 97(9) (1971) 1249-1273
[2] Andrus R.D., , P.Paramananthan, S.E. Brian, Z. Jianfeng, J.C.Hsein "Comparing liquefaction evaluation methods using penetration-V_s relationships" Soil Dynamics and Earthquake Engineering. 24 (2004) 713-721
[3] Andrus R.D., K.H.Stokoe "Liquefaction resistance of soils from shear-wave velocity" Journal of Geotechnical and Geoenvironmental Engineering. ASCE 126(11) (2000) 1015-1025
[4] Seed H.B. "Soil liquefaction and cyclic mobility evaluation for level ground during earthquakes" Journal of the Geotechnical Engineering Division. ASCE 105(GT2) (1979) 201-255
[5] Seed H.B., I.M.Idriss, Ground motions and soil liquefaction during earthquakes. Earthquake Engineering Research Institute Monograph, Oakland, California (1982)
[6] Seed H.B., K.Tokimatsu, L.F.Harder, R.M.Chung "The influence of SPT procedures in soil liquefaction resistance evaluations" J. Geotech. Eng. ASCE 111(12) (1985) 1425-1445
[7] Youd T.L., R.E.Kayen, J.K.Mitchell "Liquefaction criteria based on energy content of seismograms", Proc. NCEER Workshop on Evaluation of Liquefaction resistance of Soils, Nat. Ctr. for Earthquake Eng. Res., State Univ. of New York at Buffalo, pp. 217-224 (1997)
[8] Robertson P.K., R.G.Campanella "Liquefaction potential in sands using the CPT" Journal of the Geotechnical Engineering Division. ASCE 111(3) 384-403 (1985)
[9] Olsen R.S. "Cyclic liquefaction based on the cone penetration test", Proc. NCEER Workshop on Evaluation of Liquefaction Resistance of Soils. Nat. Ctr. for Earthquake Eng. Res., State University of New York at Buffalo, pp. 225-276 (1997)
[10] Robertson P.K., C.E.Wride "Evaluating cyclic liquefaction potential using cone penetration test" Can. Geotech. J. 35(3) (1998) 442-459
[11] Seed H.B., , P.De Alba "Use of SPT and CPT tests for evaluating the liquefaction resistance of sands" SP Clemence Proceedings, in situ'86 Virginia Tech, Blacksburg, pp. 281-302 (1986)
[12] Stark T.D., S.M.Olson "Liquefaction resistance using CPT and field case histories" J. Geotech. Eng. ASCE 121(12) (1995) 856-869
[13] Youd T.L. "Liquefaction Resistance of Soils: Summary Report from The 1996 NCEER and 1998 NCEER/NSF Workshop on Evaluation of Liquefaction Resistance of Soils" J. Geotech. Geonviron. Eng., ASCE 127(10) (2001) 817-833
[14] Dobry R., K.H.Stokoe II, R.S.Ladd, T.L.Youd "Liquefaction susceptibility from S-wave velocity. In situ testing to evaluate liquefaction susceptibility" ASCE, Geotechnical Engineering Division, New York, (1981) 1-15
[15] Seed H.B., I.M.Idriss, I.Arango "Evaluation of liquefaction potential using field performance data" Journal of the Geotechnical Engineering Division. 109(3) (1983) 458-482
[16] Stokoe K.H., S.Nazarian, G.J.Rix, I.Sanchez-Salinaro, J.C.Sheu, et al. "In situ seismic testing of hard to sample soils by surface wave method" In: Von Thun, J.L. (ed.) Earthquake Engineering and Soil Dynamics II-Recent advances in ground motion evaluation. Geotechnical Special Publication 20 Park City, Utah, pp. 264-278 (1988)
[17] Tokimatsu K., A.Uchida "Correlation between liquefaction resistance and shear wave velocity" Soils and Found. 30(2) (1990) 33-42
[18] Jamiolkowski M., D.C.F.Lo Presti "Correlation between liquefaction resistance and shear wave velocity" Soils and Found. 32(2) (1990) 145-148
[19] Roy D., R.G.Campanella, P.M.Byrne, J.M.O. Hughes "Strain level and uncertainty of liquifaction related index tests" In: Shackelford, C.D., Nelson, P.P., Roth, M.J.S. (eds.) Uncertainty in the geologic environment: From theory to practice. Geotech. Spec. Publ. No: 58(2), ASCE New York, pp. 1149-1162 (1996)
[20] Teachavorasinskun S., F.Tasuoka, D.C.F.Lo Presti "Effects of the cyclic prestaining on dilatancy characteristics and liquefaction strength of sand" In: Shibuya, S., Mitachi, T., Miura, S. (eds.) Pre-failure deformation of geomaterials, Balkema, Rotterdam, The Nederlands, pp. 75-80 (1994)
[21] Prandtl L. Über die Eindringungsfestigkeit (Härte) plastischer Baustoffe und die Festigkeit von Schneiden. (On the penetrating strengths (hardness) of plastic construction materials and the strength of cutting edges). Zeit. Angew. Math. Mech. 1(1) 15-20 (1921)

[22] Reissner H. “Zum Erddruckproblem” (Concerning the earth-pressure problem). Proc. 1st Int. Congress of Applied Mechanics. Delft, pp. 295-311 (1924)
[23] Meyerhof G.G. “Penetration tests and bearing capacity of cohesionless soils” Proceedings ASCE, 82(SM1) pp. 1-19 (1956)
[24] Hansen J.B. “A revised extended formula for bearing capacity” Danish Geotechnical Institute Bulletin. 28 (1968)
[25] DeBeer E.E. “Experimental determination of the shape factors and the bearing capacity factors of sand” Geotechnique. 20 (1970) 387-411
[26] Sieffert J.G., Ch Bay-Gress. “Comparison of the European bearing capacity calculation methods for shallow foundations” Geotechnical Engineering Institution of Civil Engineers. 143 (2000) 65-74
[27] Stokoe K.H., R.D.Woods “Insitu shear wave velocity by cross-hole method” Journal of the Soil Mechanics and Foundation Divison, ASCE. 98(SM5) (1972) 443-460
[28] Tezcan S.S., S.M.Erden, H.T.Durgunoğlu “Insitu measurement of shear wave velocity at Bosphorus (Boğaziçi) University Campus” Proceedings of the International Conference on Soil Mechanics and Foundation Engineering, Istanbul Technical University, Istanbul, 2 pp. 157-164 (1975)
[29] Belloti R., J.Jamiolkowski, D.C.F.Lo Presti, , D.A.O'Neill “Anisotropy of small strain stiffness of Ticino sand” Geotechnique. 46(1) (1996) 115-131
[30] Roesler S.K. “Anisotropic shear modulus due to stress anisotropy” Journal of the Geotechnical Engineering Division. ASCE 105(7) (1979) 871-880
[31] Stokoe K.H., S.H.H.Lee, D.P.Knox “Shear moduli measurements under true triaxial stresses” Proc. Adv. in the Art of Testing Soil Under Cyclic Conditions. ASCE, New York, pp. 166-185 (1985)
[32] Dobry R. “Some Basic aspects of soil liquefaction during earthquakes” Earthquake hazards and the design of constructed facilities in the eastern United States. In: Jacob, K.H., Turkstra, C.J. (eds.) Ann. of the New York Acad. of Sci., New York, 558 172-182 (1989)
[33] Idriss I.M. “Presentation notes: An update of Seed-Idriss Simplified Procedure for Evaluating Liquefaction Potential” Proc. TRB Workshop on New Approaches to Liquefaction Anal. Publ. No. FHWA-RD-99-165. Washington DC Federal Highway Administration (1999)
[34] Tezcan S.S., Z.Özdemir, A.Keçeli “Allowable Bearing Capacity of Shallow Foundations Based on Shear Wave Velocity”. Geotechnical and Geological Engineering. CV-338 24 (2006) 203-218
[35] Skempton A.W., D.H.MacDonald “Allowable settlement of buildings” Proceedings ICE, 5(3) pp. 727-768 (1956)
[36] Tezcan S.S., Z.Özdemir, A.Keçeli, A.Erkal “A rapid technique to determine allowable bearing pressure” International Eartquake Symposium, Kocaeli pp. 234-241 (2007)
[37] Jakosky J.J. “Exploration Geophysics” Time-Mirrors Press, Los Angeles (1940)
[38] Gardner G.H.F., L.W.Garner, A.R.Gregory “Formation velocity and density-the diagnostic basics for stratigraphic traps” Geophysics. 39(6) (1974) 770-780
[39] Yılmaz H.R. “Ground Mechanics-1 Lecture Notes” Aegean University Engineering Faculty Department of Civil Engineering. Unpublished (2001)
[40] Uyanık O., Ph.D. Thesis, The Graduate School of Natural and Applied Sciences, Dokuz Eylül University, Izmir, Kayma Dalga Hızına Bağlı Potansiyel Sıvılaşma Analiz Yöntemi. 2002 (in Turkish).