1 Theoretical basis

1.1 Calculation methods of the liquefaction hazard.

1.1.1 Empirical methods.

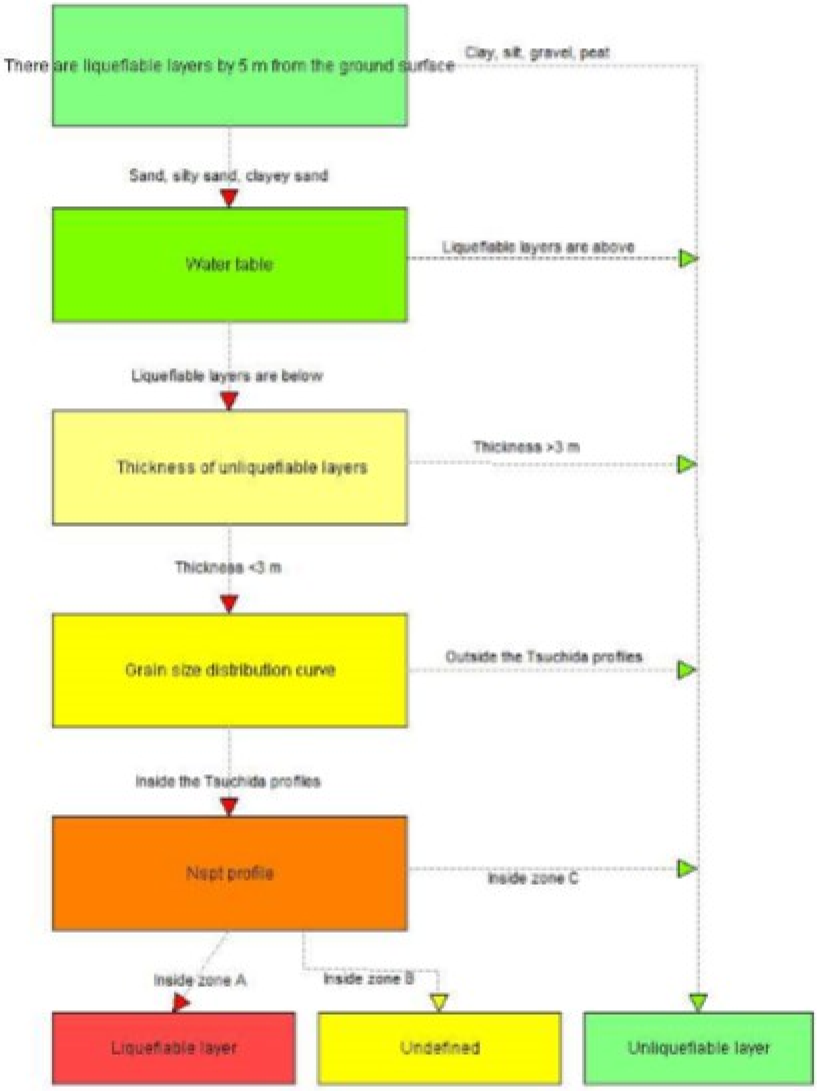
Empirical methods are generally used to get a rough estimate of the liquefaction hazard in saturated sandy layers, taking into account only the geological-geotechnical characteristics of the investigated site. They're simplified methods to use in case of surveys of seismic microzonation.

1.1.1.1 Sherif & Ishibashi (1978).

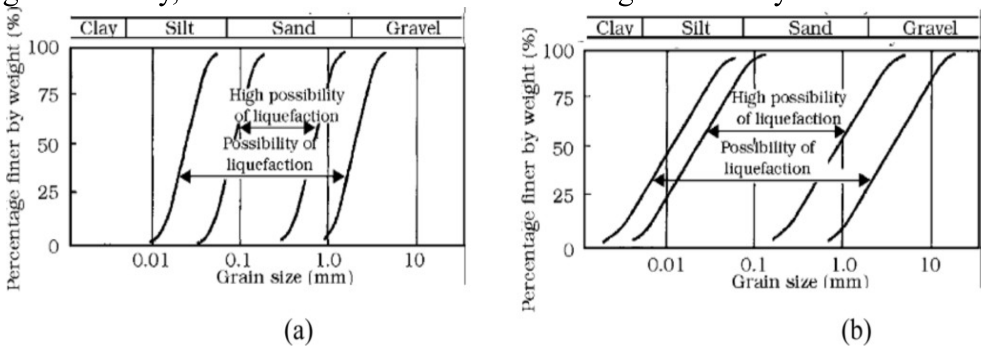
The Sherif & Ishibashi method is based on the premise that the soil layers have the following requirements:

- they are made up by sand or silty sand;
- they're below the level of the water table;
- the non-liquefiable layers, on the top, have a total thickness lower than 3 meters.

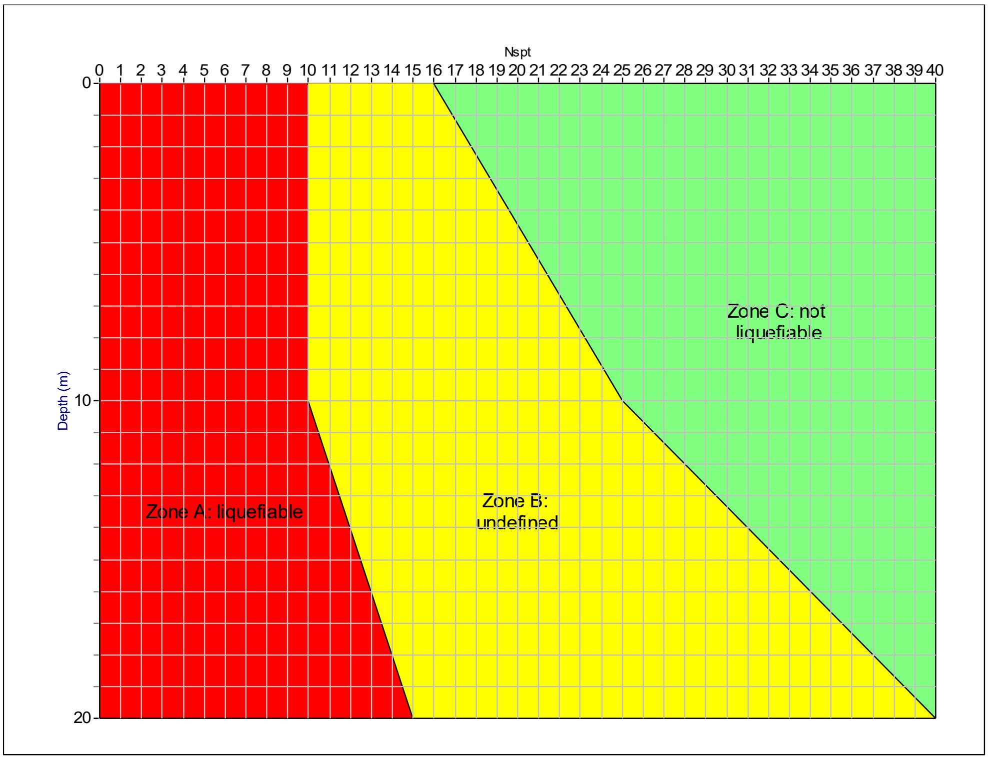
according to the following flow diagram:


There are liquefiable layers by 5 m from the ground surface
Clay, silt, gravel, peat
Sand, silty sand, clayey sand
Water table
Liquefiable layers are above
Liquefiable layers are below
Thickness of unliquefiable layers
Thickness >3 m
Thickness <3 m
Grain size distribution curve
Outside the Tsuchida profiles
Inside the Tsuchida profiles
Nspt profile
Inside zone C
Inside zone A
Inside zone B
Liquefiable layer
Undefined
Unliquefiable layer

If these requirements are present, it goes on in the procedure, taking into account the granulometry and the relative density of the soil layer. The method requires that a granulometric analysis be executed, taking a sample of the soil layer. The resulting curves has to be compared with the two critical profiles (Tsuchida, 1970), the first one related to a uniform granulometry, the second one to a non-uniform granulometry.

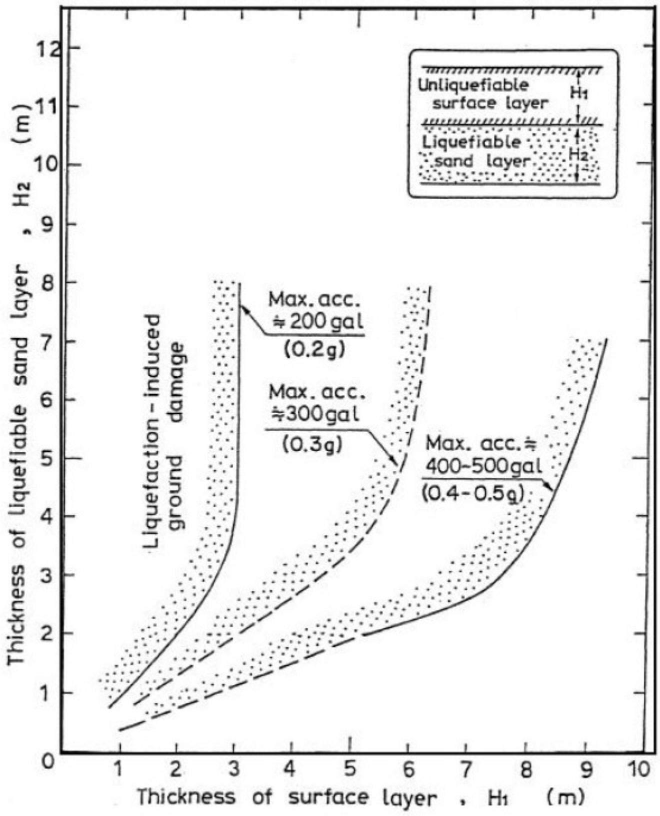


Checked that the granulometric curve of the soil layer falls within the critical limits, it's possible to proceed, taking into account the relative density of the soil layer through SPT or DP tests. If the number of blows, even if partially, falls inside the A zone, the layer is liquefiable. If it falls inside the C zone is unliquefiable. The hazard is undefined in case of profile within the B zone.



1.1.1.2 Ishihara (1985).

It's an empirical method based on a graphic.
The required data are the thickness of the unliquefiable layers (H_1), the thickness of the liquefiable layers below (H_2) and the peak ground acceleration (PGA). According the Author, saturated sandy layers with $N_{spt}<10$ are to be considered liquefiable.
The Ishihara procedure gives, in a purely qualitative manner, the magnitude of the ground deformations and consequently the damage level in case of liquefaction.



Ishihara diagram

1.1.1.3 Methods based on the seismic magnitude.

They're empirical formulas which associate the epicentral distance from the investigated site to the critical seismic magnitude, that is the moment magnitude beyond which liquefaction events could become possible in saturated sandy layers. This magnitude must be compared with the moment magnitude of the seismic event used as reference (M).

- *Ambraseys(1991)*

$$Mc = -0.31 + 2.65 \times 10^{-8} \times dist \times 100000 + 0.99 Log_{10}(dist \times 100000)$$

- *Papadopulos and Lefkopulos(1993)*

$$Mc = -0.44 + 3.00 \times 10^{-8} \times dist \times 100000 + 0.90 Log_{10}(dist \times 100000)$$

- *Galli (2000)*

$$Mc = 0.67[1.0 + 3.0 \times Log_{10}(dist)] + 2.07$$

where *dist* is the epicentral distance in km.
Thus, if the ratio M/Mc is higher than 1, liquefaction phenomenons become possible.

1.1.1.4 Method based on the Arias Intensity.

The Arias Intensity (1970) represents an index proportional to the energy released by the seismic event. Practically it's the integral of the square of the seismic acceleration extended to all the duration T of the accelerogram:

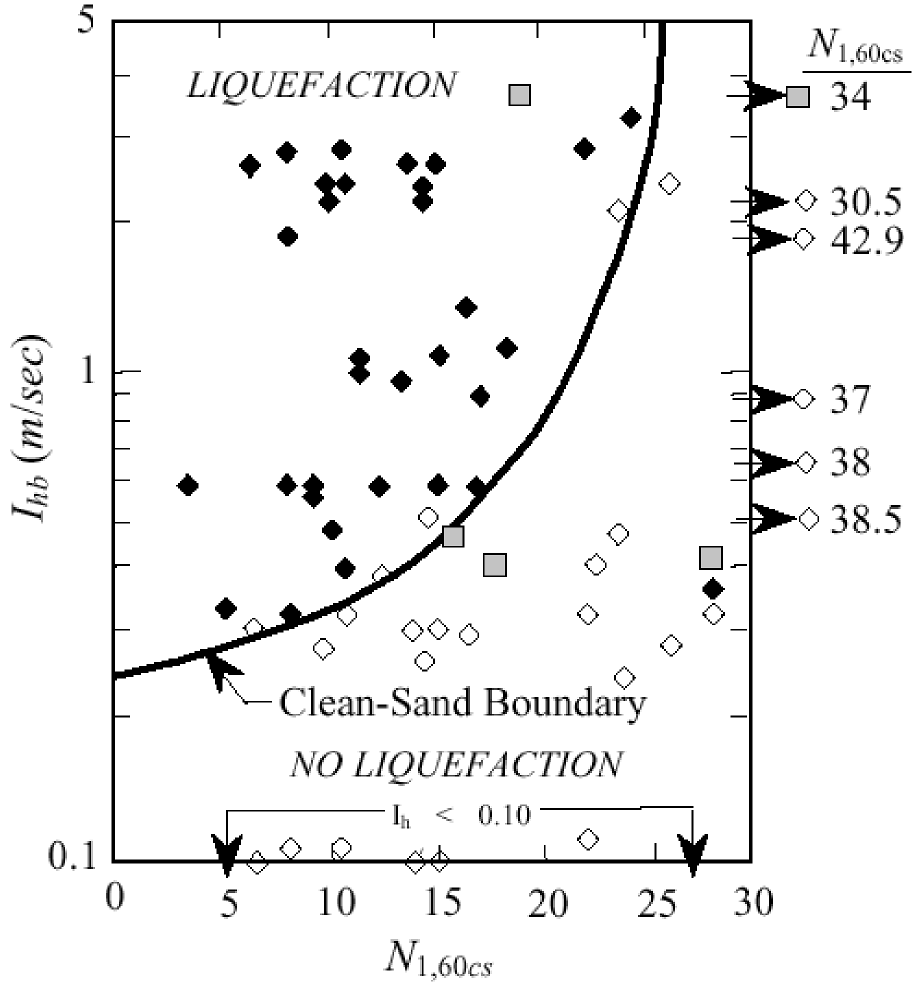
$$Ia = \frac{\pi}{2g} \int_o^T a^2(t)dt$$

The significant duration of the earthquake is defined as the time interval between 5% and 95% of the Arias intensity.
The variability of Ia as a function of the depth *h* can be gotten through the empirical formula of Kayen e Mitchell (1997):

$$Ia(h) = Ia \times r_b$$

$$r_b = \exp\left[\frac{35}{{M_w}^2} sen(-0.09h)\right]$$

where M_w is the moment magnitude of the earthquake of reference.
The Arias Intensity can be correlated to the liquefaction risk through the following scheme:



where $N_{1,60cs}$ represents the number of blows SPT normalized in case of clean sand. The $N_{1,60cs}$ parameter is given, applying a correction to Nspt, chosen between the formulas proposed by several Authors. The clean-sand boundary of the graph can be correlated to the CRR parameter through the following formula:

$$Ihb_{\lim} = 0.00255(100 \times CRR)^2$$

1.1.2 Simplified methods.

Simplified methods allow to quantify the liquefaction risk through a safety factor given by the ratio between the shear resistance of the soil layer (CRR) and the shear stress produced by the earthquake (CSR):

$$F_s = \frac{CRR}{CSR}.$$

A soil layer can be classified as a liquefiable level if the safety factor is lower than 1 (1.25 according the Eurocode 8). The CSR variable depend on the parameters of the earthquake of reference (PGA and moment magnitude).

CRR is as function of the geotechnical characteristics of the soil layer, mainly by its relative density, and can be directly given through correlations with penetrometer tests (SPT and CPT) and geophysical surveys, the latter to get a S waves profile.

1.1.2.1 Evaluation of shear stress generated by an earthquake (CSR).

The CSR parameter is given by the following relation:

$$T = 0.65\frac{a_{\max}}{g}\frac{\sigma_{v0}}{\sigma_{v0}'}r_d\frac{1}{MSF};$$

where:

a_{max}	= peak ground acceleration;
g	= acceleration of gravity = 980.7 cm/s^2;
σ_{v0}	= vertical total pressure at the depth z from the ground;
σ_{v0}’	= vertical effective pressure at the depth z from the ground;
r_d	= variable as a function of the depth from the ground;
MSF	= corrective coefficient as a function of the seismic magnitude.

The variable rd can be calculated through several empirical correlations, known in literature.

- *Seed (1971)*

$rd = 1 - 0.01 h_{media}$ h_{media} (mean depth of the layer) < 10m
$rd = 1.15 - 0.025 h_{media}$ $h_{media} \geq 10$m

- *Liao & Whitman (1986)*

$rd = 1 - 0.00765 h_{media}$ $h_{media} \leq 9.15$m
$rd = 1.174 - 0.0267 h_{media}$ $9.15 < h_{media} \leq 23$m
$rd = 0.774 - 0.008 h_{media}$ $23 < h_{media} \leq 30$m
$rd = 0.5$ $h_{media} > 30$m

- *Blake (1996)*

$$rd = \frac{\alpha}{\beta}$$

$$\alpha = 1.0 - 0.4113\sqrt{h_{media}} + 0.04052 h_{media} + 0.001753 h_{media}{}^{1.5}$$

$$\beta = 1.0 - 0.4177\sqrt{h_{media}} + 0.05729 h_{media} - 0.00620 h_{media}{}^{1.5} + 0.001210 h_{media}{}^{2}$$

- *Boulanger & Idriss (2014)*

$$rd = \exp(\alpha + \beta)$$

$$\alpha = -1.012 - 1.126 sen\left(\frac{h_{media}}{11.73} + 5.133\right)$$

$$\beta = M_w\left[0.106 + 0.118 sen\left(\frac{h_{media}}{11.28}\right) + 5.142\right]$$

where M_w is the moment magnitude.

The corrective coefficient MSF can be computed, in case of SPT and CPT, through the expressions dby Boulanger & Idriss (2014):

$$MSF = 1 + (MSF_{max} - 1)\left(8.64\exp\left(\frac{-M}{4}\right) - 1.325\right)$$

where:

$$MSF_{max} = 1.09 + \left(\frac{q_{C1Ncs}}{180}\right)^3 \leq 2.2$$

$$MSF_{max} = 1.09 + \left(\frac{(N_1)_{60cs}}{31.5}\right)^2 \leq 2.2$$

In case of empirical methods based on the shear wave velocity, the formula by Idriss (1999) can be used:

$$MSF = 6.9 \cdot \exp\left(\frac{-M}{4}\right) - 0.058 \ \leq 1.8$$

As an alternative, it may use the method by Idriss (1995):

$$MSF = \left(\frac{M}{7.5}\right)^{-3.3}$$

M≤7.5

$$MSF = \frac{10^{2.24}}{M^{2.56}}$$

M>7.5.

1.1.2.2 Evaluation of the shear strength of the soil (CRR).

1.1.2.2.1 By dynamic penetration tests (SPT) – Tokimatsu & Yoshimi (1983).

According the Tokimatsu & Yoshimi method, CRR is given by the following formula:

$$CRR = 0.26\left[0.16\sqrt{Na} + \left(0.21\sqrt{Na}\right)^{14}\right]$$

where:

Na	$= N_{spt}\left(\frac{1.7}{\sigma_v + 0.7}\right) + N_1$
σ_v(kg/cmq)	= vertical effective pressure;
N_1	= 0 when percentage of fines is $p_c < 5\%$, 10 p_c+4 when $p_c \geq 5$ %

1.1.2.2.2 By dynamic penetration tests (SPT) – Seed et al. (1985).

CRR is calculated by the following relation:

$$CRR = \frac{a + cx + ex^2 + gx^3}{1 + bx + dx^2 + fx^3 + hx^4}$$

where:

x = $N_{1,60cs}$ number of blows SPT in case of a 60% energy efficiency, corrected to take into account the fine component (silt and clay).

a =0.048;
b =-0.1248;
c =-0.004721;
d =0.009578;
e =0.0006136;
f =-0.0003285;
g =-0.00001673;
h =0.000003714.

$N_{1,60cs}$ can be calculated by the following formula:

$$N_{1,60cs} = f_a + f_b (C_E C_N C_b C_r C_s Nspt)$$

where:

C_N = correction factor as a function of the depth of the measurement = $\sqrt{\frac{1}{\sigma_v}}$ (σ_v in kg/cmq); if $C_N > 1.7$ put $C_N = 1.7$;

C_E = correction factor as a function of the energy efficiency= ER/60 with ER% is the efficiency of the used probe;

C_b = correction factor as a function of the borehole diameter;
these the recommended values:
from 65 to 115 mm C_b=1.0
from 115 to 150 mm C_b=1.05
from 150 to 200 mm C_b=1.15

C_r = correction factor as a function of the rod length;
these the recommended values:
from 3 to 4 m C_r=0.75
from 4 to 6 m C_r=0.85
from 6 to 10 m C_r=0.95
from 10 to 30 m C_r=1.0
>30 m C_r>1.0

C_s = correction factor as a function of type of sampler;
these the recommended values:
standard C_s=1.0
without liners C_s da 1.1 a 1.3

f_a = 0 when the percentage of fines (FC)≤5%;
= $\exp\left(1.76 - \frac{190}{FC^2}\right)$ when 5<FC<35 %;
=5 when FC≥35 %;

f_b = 1 for FC≤5%;
= $0.99 + \frac{FC^{1.5}}{1000}$ for 5<FC<35 %;
=1.2 for FC≥35 %.

1.1.2.2.3 By dynamic penetration tests (SPT)– Youd et al.(2001)

CRR is given by the following relation:

$$CRR = \frac{1}{34 - N_{1,60cs}} + \frac{N_{1,60cs}}{135} + \frac{50}{\sqrt{10N_{1,60cs} + 45}} - \frac{1}{200}$$

1.1.2.2.4 By dynamic penetration tests (SPT) – Boulanger & Idriss(2014)

CRR is estimated through the following formula:

$$CRR = \exp\left[\frac{N_{1,60cs}}{14.1} + \sqrt{\frac{N_{1,60cs}}{126}} - \left(\frac{N_{1,60cs}}{23.6}\right)^3 + \left(\frac{N_{1,60cs}}{25.4}\right)^4 - 2.8\right]$$

$N_{1,60cs}$ is given by this relation:

$$N_{1,60cs} = N_{1,60} + \exp\left(1.63 + \frac{9.7}{FC} - \sqrt{\frac{15.7}{FC}}\right)$$

where FC is the percentage of fines and $N_{1,60}$ is the normalized Nspt value. Its expression is the following:

$$N_{1,60} = \mathrm{Nspt} \times C_N \times C_E \times C_R \times C_S \times C_B$$

The variables C_E, C_R, C_S e C_B have the same expressions seen in the case of the Seed & Al.'s method. The parameter C_N can be estimated in a iterative way, proceeding according this scheme:

1. A initial value of $N_{1,60}$ is calculated, assuming $C_N = \sqrt{\frac{98.1}{\sigma'}}$, where σ'(kPa)= vertical effective pressure;
2. C_N is then recalculated using the following expression: $C_N = \left(\frac{98.1}{\sigma'}\right)^{\alpha}$ where $\alpha = 0.784 - 0.0768\sqrt{N_{1,60}}$
3. $N_{1,60}$ is estimated again through the new value of C_N;

4. Steps 2 and 3 are repeated till when the difference between the values of C_N calculated in two consecutive cycles is lower than a chosen limit.

If C_N is higher than 1.7, put C_N=1.7.

1.1.2.2.5 By static penetration tests (CPT) – Robertson & Wride (1997).

Robertson & Wride's method permits to correlate the soil shear resistance to the data of a static penetration test (CPT). The procedure of calculation is based on the two following equations:

$$CRR = 0.883\left[\frac{(q_{c1n})_{cs}}{1000}\right] + 0.05 \quad (q_{c1n})_{cs} < 50 \text{ and}$$

$$CRR = 93\left[\frac{(q_{c1n})_{cs}}{1000}\right]^3 + 0.08 \quad 50 \le (q_{c1n})_{cs} < 160.$$

The variable $(q_{c1n})_{cs}$ represents the normalized point resistance, corrected to take into account the percentage of fines.

The calculation of $(q_{c1n})_{cs}$ is performed through the following steps.

- The normalized cone resistance and friction ratio are evaluated through the following relations:

$$Q = \frac{q_c - \sigma_{v0}}{\sigma_{v0}'} \text{ e } F = 100\frac{f_s}{q_c - \sigma_{v0}}$$

where:

q_c (kg/cmq) = measured cone resistance;

f_s (kg/cmq) = measured sleeve friction;

σ_{v0}(kg/cmq) = vertical total pressure;

σ_{v0}'(kg/cmq) = vertical effective pressure.

- The soil behavior type index of the sandy layer is calculated:

$$Ic = \sqrt{(Log_{10}F + 1.22)^2 + (Log_{10}Q - 3.47)^2}$$

- The correction to take into account the depth of the test is estimated:

$$q_{c1n} = C_Q q_c \text{ dove } C_Q = \left(\frac{1}{\sigma_{v0}'}\right)^n$$

The exponent *n* is evaluated in the following way:

- if Ic>2.6 then n=1;
- if Ic≤2.6 an initial value of q_{c1n}, is calculated, using n=0.5; then Ic is recalculated through this relation:

$$Ic = \sqrt{\left(Log_{10}F + 1.22\right)^2 + \left(Log_{10}q_{c1n} - 3.47\right)^2}$$

if the new Ic value is lower than 2.6 again, a value n=0.5 is used, otherwise q_{c1n} is recalculated, using n=0.75;

- if q_{c1n}>2q_c put q_{c1n}=2q_c .

Finally the correction as a function of fines in the sandy layer is evaluated:

$$(q_{c1n})_{cs} = K_c q_{c1n} ,$$

where K_c is 1, if Ic≤1.64, otherwise it's given by the following relation:

$$K_c = -0.403Ic^4 + 5.581Ic^3 - 21.63Ic^2 + 33.75Ic - 17.88$$

1.1.2.2.6 By static penetration tests (CPT) – Boulanger & Idriss(2014)

CRR is given by the following expression:

$$CRR = \exp\left[\frac{q_{c1Ncs}}{113} + \left(\frac{q_{c1Ncs}}{1000}\right)^2 - \left(\frac{q_{c1Ncs}}{140}\right)^3 + \left(\frac{q_{c1Ncs}}{137}\right)^4 - 2.80\right]$$

The variable qc_{1n} has to be estimated through an iterative way, proceeding according the following scheme:

1. An initial value of qc_{1n}, is calculated, imposing $qc_{1n} = \frac{qc}{98.1}$;

2. The corrective factor C_Q is evaluated through the following relation: $C_Q = \left(\frac{98.1}{\sigma'}\right)^{\alpha}$ where σ'(kPa)= vertical effective pressure and $\alpha = 1.338 - 0.294 \times qc_{1n}^{\ \ 0.264}$
3. qc_{1n} is recalculated by the expression $qc_{1n} = C_Q \frac{qc}{98.1}$
4. Steps 2 and 3 are repeated till when the difference between the values of C_Q calculated in two consecutive cycles is lower than a chosen limit.

If C_Q is higher then 1.7, put C_Q=1.7.

The parameter qc_{1ncs} is given by qc_{1n} , inserting a correction to take into account the percentage of fines inside the sandy layer.

$$q_{c1Ncs} = q_{c1N} + \Delta q_{c1N}$$

where:

$$\Delta q_{c1N} = \left(11.9 + \frac{q_{c1N}}{14.6}\right) \exp\left(1.63 - \frac{9.7}{FC+2} - \left(\frac{15.7}{FC+2}\right)^2\right)$$

and:

$$FC = 80\ I_c - 137$$

1.1.2.2.7 By Vs profiles – Andrus & Stokoe(1997).

The liquefaction resistance of a soil layer can be evaluated through empirical methods as a function of the S wave velocity. The Andrus & Stokoe's method is based on the following expression:

$$R = 0.03\left(\frac{V_{s1}}{100}\right)^2 + \frac{0.9}{V_{s1c} - V_{s1}} - \frac{0.9}{V_{s1}}$$

where:

V_{s1}(m/s) = corrected S wave velocity = $V_s\left(\frac{1}{\sigma_{v0}'}\right)^{0.25}$, where V_s is the measured velocity and σ_{v0}'(kg/cmq) is the vertical effective pressure in the midpoint of the layer;

V_{s1c}(m/s) = critical value of the S wave velocity in the layer, given by the following scheme:

V_{s1c}(m/s)=220 if the percentage of fines(FC)<5%;
V_{s1c}(m/s)=210 if FC=20%;
V_{s1c}(m/s)=200 if FC>35%;
interpolating in case of intermediate values of FC.

1.2.2.2.8 By Vs profile – Boulanger & Idriss(2004)

The relation is the following:

$$R = 0.022\left(\frac{V_{s1}}{100}\right)^2 + \frac{2.8}{V_{s1c} - V_{s1}} - \frac{2.8}{V_{s1}}$$

where:

V_{s1}(m/s) = corrected S wave velocity in the sandy layer = $V_s\left(\frac{1}{\sigma_{v0}'}\right)^{0.25}$, where V_s is the measured velocity and σ_{v0}'(kg/cmq) is the vertical effective pressure in the midpoint of the layer;

V_{s1c}(m/s) = critical value of the S wave velocity in the layer, given by the following scheme:

V_{s1c}(m/s)=215 if the percentage of fines (FC)<5%;
V_{s1c}(m/s)=215-0.5x(FC-5) if 5<FC<35%;
V_{s1c}(m/s)=200 if FC≥35%;

1.1.2.3 Evaluating CRR in case of external loads and/or slope of the ground surface.

The calculation methods of CRR seen in the previous paragraphs are valid in case of absence of external loads (free-field condition) and where the ground surface is horizontal. If these conditions aren't satisfied, it's possible to evaluate CRR using the following relation:

$$CRR = CRR_0 K_\sigma K_\alpha$$

where:

CRR_0 = CRR calculated in case of free-field condition and horizontal ground surface

K_σ = corrective factor for external loads or high values of the effective pressure;

K_α = corrective factor for slope ground surface (shear stress>0 in static condition).

The variable K_σ is a reducing coefficient suggested because of the measured decrease of CRR in presence of high values of σ_{v0}'. This effect is substantial in case of σ_{v0}'values higher than 200 kPa. Without external loads (*free field* condition) these values of σ_{v0}', in saturated layers, are reached at depth higher than 15 m.

K_σ can be estimated through the following empirical formula (Boulanger & Idriss, 2004):

$$K_\sigma = 1 - C_\sigma \ln\left(\frac{\sigma_{v0}'}{p_a}\right)$$

where:

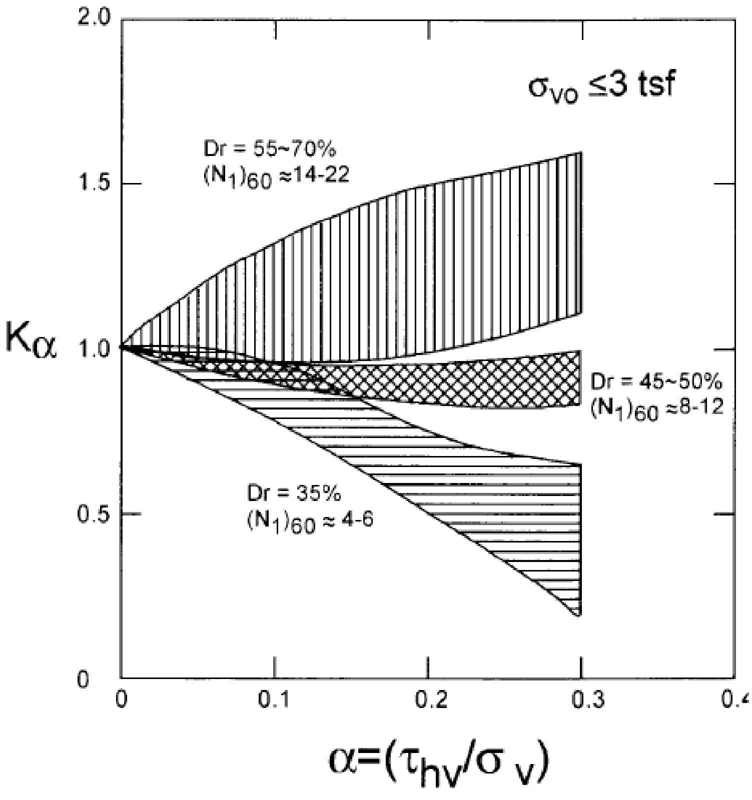
$$C_\sigma = \frac{1}{18.9 - 2.55\sqrt{(N_1)_{60}}} \leq 0,3 \text{ by SPT;}$$

$$C_\sigma = \frac{1}{37.3 - 8.27(q_{c1N})^{0.264}} \leq 0,3 \text{ by CPT;}$$

$$C_\sigma = \frac{1}{18.9 - 3.1(V_{s1}/100)^{1.976}} \leq 0,3 \text{ by Vs profile.}$$

The corrective factor K_α quantifies the effect of a static shear stress inside the soil before the seismic event. A rough value of K_α can be gotten by the following chart, as a function of the relative density (D_r). The parameter α is given by the ratio between the static shear stress and the vertical effective pressure.

The applicability of K_α is restricted at cases where σ_{v0}'is lower or equal than about 300 kPa. Beyond this limit the effect of the effective pressure cancels the one due to the static shear stress. In these cases only K_σ has to be applied.



1.1.2.4 Calculation of the liquefaction potential index.

An estimate of the liquefaction hazard along a specific vertical of calculation is given by the parameter Index of Liquefaction (IL). Such an index is estimable through the following relation:

$$IL = \sum_{i=1}^{n} FW(z)\Delta z$$

where:
n = calculation steps of SF along the vertical;
F = 0 if Fs≥Fs of reference;
$F=2000000e^{-18.427Fs}$ if Fs of reference>Fs≥0.95
F = 1-Fs if Fs<0.95
Δz = thickness of the calculation step;
W(z) = 10 – 0,5z, where z = depth of calculation (maximum value=20 m).
Based on the calculated IL value it's possible gets a measure of the liquefaction hazard through the following table:

IL	Rischio di liquefazione
IL=0	Very low
0<IL≤2	Low
2<IL≤5	Moderate
5<IL≤15	High
15<IL	Very high

1.2 Improvement techniques of the liquefiable soils.

In the following paragraphs the elements for a rough sizing of two common improvement techniques are introduced:

- vertical drains;
- dynamic methods (compaction and heavy tamping).

1.3.1 Vertical drains.

Vertical drains are columns of gravel driven inside the liquefiable layers, in order to improve the dissipation of the pore pressure during the seismic events. A rough sizing can be executed, by trial, starting from a specific value of the diameter of the drain (usually larger than 0.8 m). The calculation of the spacing is based on the hypothesis that the drainage of the interstitial water can be permitted only along the radial (horizontal) direction and that the relative density of the soil layer doesn't change. An estimate of the drain spacing can be performed, using the Kjelmann's simplified formula (1995):

$$(2)\, t = \frac{d_e}{8c_{vh}}\left[\ln\left(\frac{d_e}{d}\right) - \frac{3}{4}\right]\ln\frac{1}{1-U_h}$$

where:

d_e(m) = diameter of the cylinder of drained soil ;

d(m) = drain diameter;

c_{vh}(m^2/s) = horizontal coefficient of consolidation of the drained soil, given by

$$c_{vh} = \frac{k_h E_d}{\gamma_w}$$

where k_h if the horizontal coefficient of permeability.

U_h = design value of the degree of dissipation of the pore pressure.

Practically, chosen a dissipation time *t* according to a specific value of U_h (for example 0.92), the parameter c_{vh} is calculated and then, by trial, also d_e and *d* are estimated. Calculated d_e, the spacing *S* of the drains is gotten by the following relations:

S(m)=d_e/1.05 (triangular pattern)

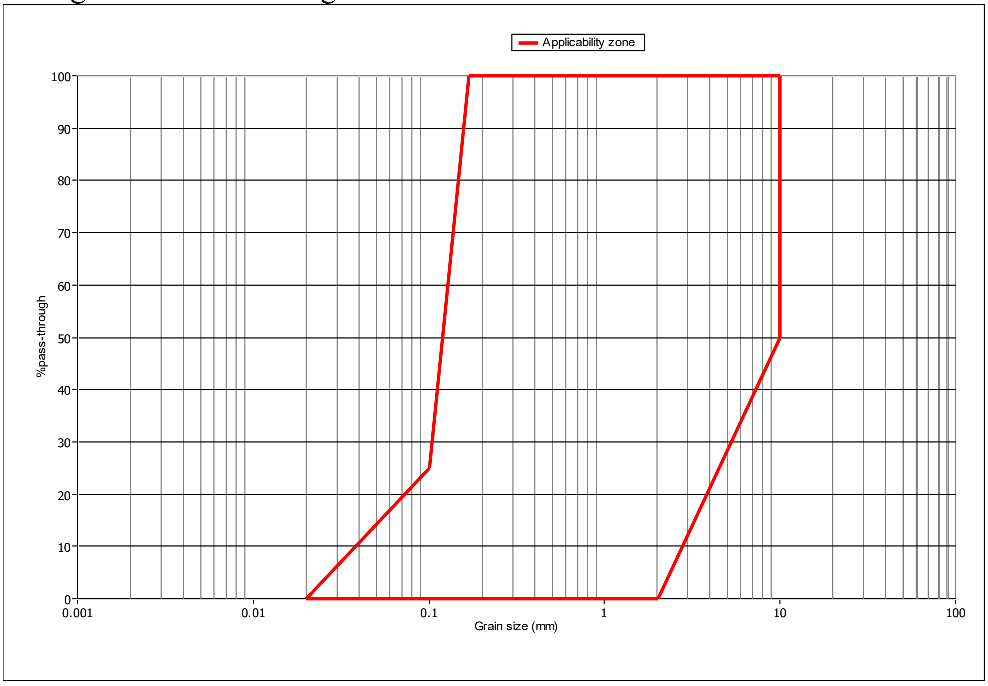
S(m)=d_e/1.128 (square pattern)

1.2.2 Compaction techniques.

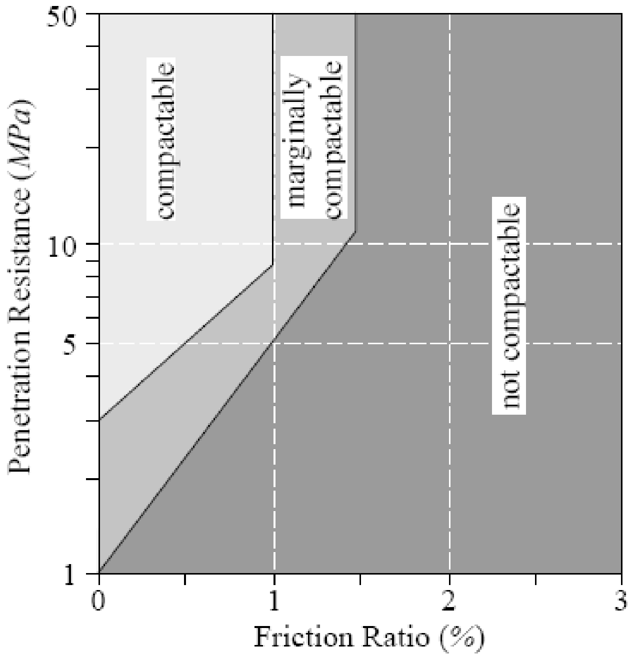
Dynamic methods have the purpose to improve the relative density of the liquefiable layers by means of vibrations generated through specific devices. The most common methods are vibrocompaction and heavy tamping.

1.2.2.1 Applicability of the dynamic methods.

The effectiveness of the compaction methods mainly depends by the granulometry of the liquefiable layer. In case of layers having a high percentage of fines, the dynamic methods are scarcely useful. Thorburn (1975) suggested a reference scheme to evaluate the applicability of the compaction methods. Only the soil layers, whose granulometric curves completely fall inside the suggested applicability ranges, are improvable through these methodologies.



Alternatively, it can use the scheme proposed by Massarsch and Heppel (1991), where the effectiveness of the compaction methods is correlated to the soil strength measured through CPT tests.



Generally these procedures are useful in case of sandy soils having a percentage of silt and clay lower than 20%.

1.2.2.3 Heavy tamping

The purpose of this method is to improve the relative density of the liquefiable soil layers through the vibrations generated by the impact of a mass, falling repeatedly on the ground surface. Concrete blocks of several tons of weight are usually used, with a drop height up to 20-30 m.
The procedure normally involves 2-3 blows per mq. Considering the difficulty to envisage the effectiveness of the methodology, it suggests to do a check after, through, for example, CPT tests in order to estimate the reached degree of density.
The effective depth reached by the intervention can be roughly evaluated through the following formula:

$$D = n\sqrt{WH}$$

where
H (m) = drop height of the mass;
P (t) = weight of the mass;
n= coefficient varying from 0.65 to 0.80
D(m) = maximum depth of improvement.

This methodology is useful in case of sandy soils having a percentage of silt and clay lower than 10%.