

EN 1998-5:2021

Section 7: Evaluation of the seismic response of the construction site

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7 Evaluation of the seismic response of the construction site

Five topics are included

- Siting - Potentially active seismic faults
- Slope Stability
- Potentially liquefiable soil
- Settlements of soil under cyclic loading
- Ground response analysis (GRA)

Two associated Annexes (informative)

- Annex B - Procedure for liquefaction analyses
- Annex C - Evaluation of settlements of coarse-grained soils

7.1 Siting

- A major change in the new (2021) version is allowance for construction close to faults

from EN 1998-5:2004

4.1 Siting

4.1.1 General

(1)P An assessment of the site of construction shall be carried out to determine the nature of the supporting ground to ensure that hazards of rupture, slope instability, liquefaction, and high densification susceptibility in the event of an earthquake are minimised.

(2)P The possibility of these adverse phenomena occurring shall be investigated as specified in the following subclauses.

4.1.2 Proximity to seismically active faults

(1)P Buildings of importance classes II, III, IV defined in EN 1998-1:2004, 4.2.5, shall not be erected in the immediate vicinity of tectonic faults recognised as being seismically active in official documents issued by competent national authorities.

(2) An absence of movement in the Late Quaternary may be used to identify non active faults for most structures that are not critical for public safety.

(3)P Special geological investigations shall be carried out for urban planning purposes and for important structures to be erected near potentially active faults in areas of high seismicity, in order to determine the ensuing hazard in terms of ground rupture and the severity of ground shaking.

7.1 Siting

7.1.2 Potentially active seismic faults

- Close to potentially active faults (~ a few hundred meters), structures of Consequence Classes CC2 and CC3 may be constructed if:
 - a. a continuous stiff foundation is provided
 - b. soil cover exceeds a certain thickness H_{cov}
- Bearing piles should not be designed to cross the potential fault plane, and their tip should be located at least 10 diam. above this plane.
- It is not required to consider simultaneous effects of fault rupture and structural vibrations due to ground shaking.

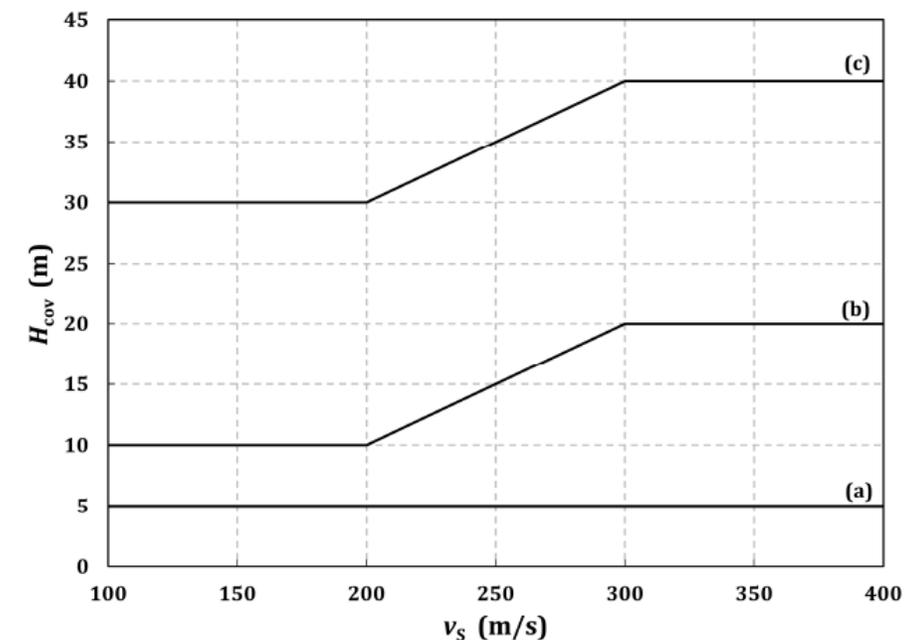


Figure 7.1 — Thickness H_{cov} of minimum allowed soil cover versus average soil shear wave velocity v_s within the depth of influence of the foundation: (a) low seismic action class; (b) moderate seismic action class; (c) high seismic action class

7.2 Slope stability

- When slope instability affects an adjacent structure, the consequence class and the limit states for the slope should be taken as those of the affected structure.
- Limit states for slopes should be associated to **acceptable permanent ground displacements**.

Methods of analysis

- **Forced-based approach, FBA** (allowed only if there is no danger of liquefaction or significant reduction of soil strength).
- **Displacement-based approach, DBA** (to be used when an evaluation of displacements is needed).

7.2 Slope stability

- The two analysis methods, FBA and DBA
- Seismic action in the FBA
- FBA is handled in more detail/clarity in 2021 version with consideration of soil nonlinearity

4.1.3.3 Methods of analysis

from EN 1998-5:2004

(1)P The response of ground slopes to the design earthquake shall be calculated either by means of established methods of dynamic analysis, such as finite elements or rigid block models, or by simplified pseudo-static methods subject to the limitations of (3) and (8) of this subclause.

(2)P In modelling the mechanical behaviour of the soil media, the softening of the response with increasing strain level, and the possible effects of pore pressure increase under cyclic loading shall be taken into account.

(3) The stability verification may be carried out by means of simplified pseudo-static methods where the surface topography and soil stratigraphy do not present very abrupt irregularities.

(4) The pseudo-static methods of stability analysis are similar to those indicated in EN 1997-1:2004, **11.5**, except for the inclusion of horizontal and vertical inertia forces applied to every portion of the soil mass and to any gravity loads acting on top of the slope.

(5)P The design seismic inertia forces F_H and F_V acting on the ground mass, for the horizontal and vertical directions respectively, in pseudo-static analyses shall be taken as:

$$F_H = 0,5\alpha \cdot S \cdot W \quad (4.1)$$

$$F_V = \pm 0,5F_H \text{ if the ratio } a_{vg}/a_g \text{ is greater than } 0,6 \quad (4.2)$$

7.2.2.2 Forced-based approach

- **Seismic demand** for the slope is expressed by a **horizontal seismic coefficient** α_H

$$\alpha_H = \frac{a_H}{g} \quad \text{where} \quad a_H = \frac{\beta_H S_\alpha}{\chi_H F_A} = \frac{\beta_H}{\chi_H} PGA_e$$

- $\chi_H > 1$ is a coefficient reflecting the soil nonlinearity and the **amplitude of accepted permanent** ground with different values depending on the considered limit state (DL, SD or NC)

- **Vertical component** of seismic action may be neglected except for high seismic action where it should be taken as half of horizontal.

- **Seismic resistance** of the slope should be expressed by its critical seismic coefficient α_C (minimum value of horizontal seismic coefficient leading to pseudo-static failure).

Table 7.1 – Values of χ_H for slope stability analyses

χ_H	1,5	2,0	2,5
Range of permanent displacements (mm)	30-50	60-100	120-200
NOTE Values of χ_H in Table 7.1 are calibrated for the recommended values of material factors and global resistance factors. Values of χ_H for other values of the material factors or global resistance factors are not provided in this standard.			

7.2.2.3 Displacement-based approach

- Performance of a slope should be evaluated based on the **acceptable permanent displacements** (depending on, for example, adjacent structure)
- Permanent displacements may be calculated using either a non-linear dynamic analysis or a rigid block model – NB: rigid-block model cannot be used where there is significant reduction in soil strength unless the residual soil shear strength is used.
- The seismic demand of the slope is expressed as the **permanent displacement** produced by the seismic action and the **seismic capacity** is expressed as the maximum acceptable permanent displacement

7.3 Potentially liquefiable soils

- Liquefaction assessment should be performed for **free-field** site conditions (ground surface elevation, ground water level) prevailing during the design service life of the structure
 - Note: The water level (Clause 6.2) should be equal to its **quasi-permanent** value (per EN 1990:2020), – a simple definition is the value averaged over a chosen time period.
- Susceptibility to liquefaction (more specific in Annex B)
 1. Sands, gravelly sands, silts, mine tailings, and fine-grained soils with plasticity index not greater than 15 should be evaluated for liquefaction susceptibility.
 2. Soils with clay fraction greater than 15% are not susceptible to liquefaction.
- Liquefaction assessment may be neglected for magnitudes smaller than $M_{wT} = 5$ (**NDP value**)
- For structures on foundations other than piles, in **low seismic action** classes, the consequences of liquefaction may be ignored if liquefaction is found at **depths greater than 15 m** below the foundation base.

7.3.5 Liquefaction assessment

More specific and more informative compared with **2004 version**

- Liquefaction assessment follows the conventional procedure using the resistance factor approach (MFA):

$$(CRR/\chi_{\tau_{cy,u}})/CSR \leq 1,0$$

$$CRR = \frac{\tau_{cy,u}}{\sigma_v} \quad CSR=0,65 \frac{\tau_{max}}{\sigma_v} \quad \tau_{max} = \alpha_H r_d \sigma_v \Rightarrow \alpha_H \text{ computed with } \beta_H = \chi_H = 1,0$$

- For strongly **heterogeneous** soil profiles, τ_{max} should be determined from a GRA.
- CRR should be evaluated using accepted SPT or CPT based methods, and **conventional correction factors** may/should be applied (**Informative Annex B**)
 - a) SPT hammer impact energy (for SPT-based methods); b) overburden pressure; c) fines content, d) thin layer correction; e) ageing effects; f) shaking history; g) earthquake magnitude correction; h) effective overburden pressure; i) initial static shear stress correction

- For **finned-grained soils** (described in Annex B) and high seismic action classes laboratory tests should be used.

Plasticity Index	Water content / Liquid Limit	Susceptibility
< 10	> 0,85	susceptible
10 to 15	> 0,80	moderately susceptible
0 to 10	0,80 to 0,85	moderately susceptible
< 15	< 0,80	not susceptible
> 15		not susceptible

7.3.5 Liquefaction assessment

If the soil is considered liquefiable:

- For **low seismic action** classes, consequences of liquefaction may be assessed using a simplified liquefaction index (Annex B)
- For **moderate seismic action** classes, consequences of liquefaction may be assessed using a combination of a simplified liquefaction index and an evaluation of the free-field settlements (Annex C)
- For **high seismic action** classes, potential consequences of liquefaction should be evaluated (numerical or empirical methods):
 - exceedance of load bearing capacity (using residual strength)
 - instability of foundations (using residual strength)
 - settlement and differential settlement of the structure (Annex C)
 - lateral spreading (Annex C)
- **Liquefaction remediation** (ground improvement, use of piles), Sec. 7.3.6

7.4 Settlements of soils under cyclic loading (moderate/high seismicity classes)

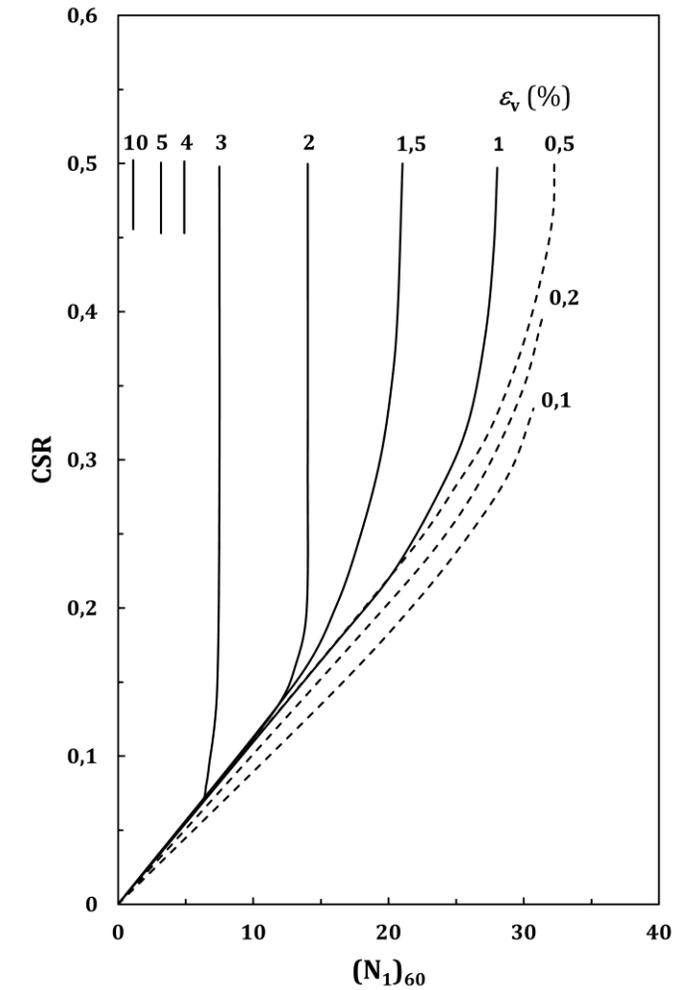
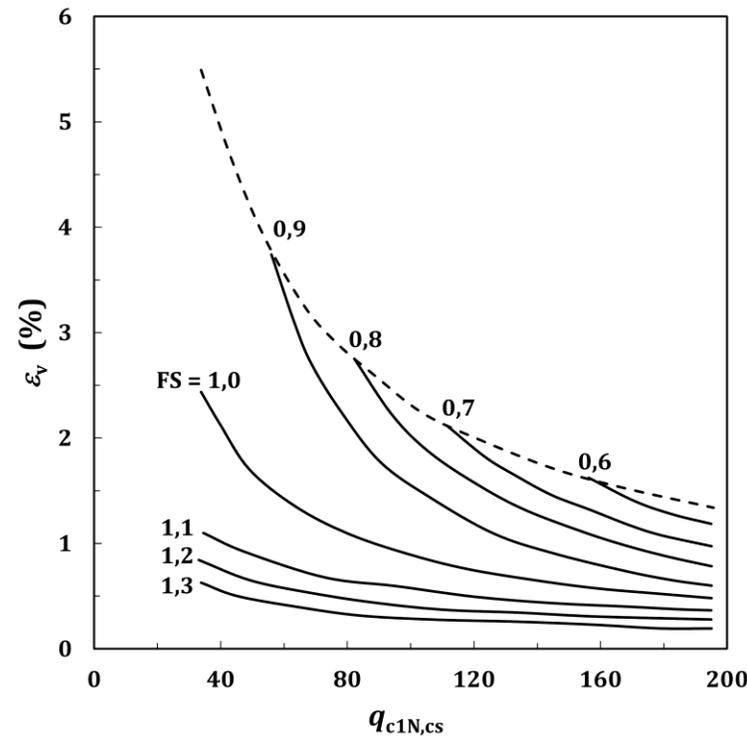
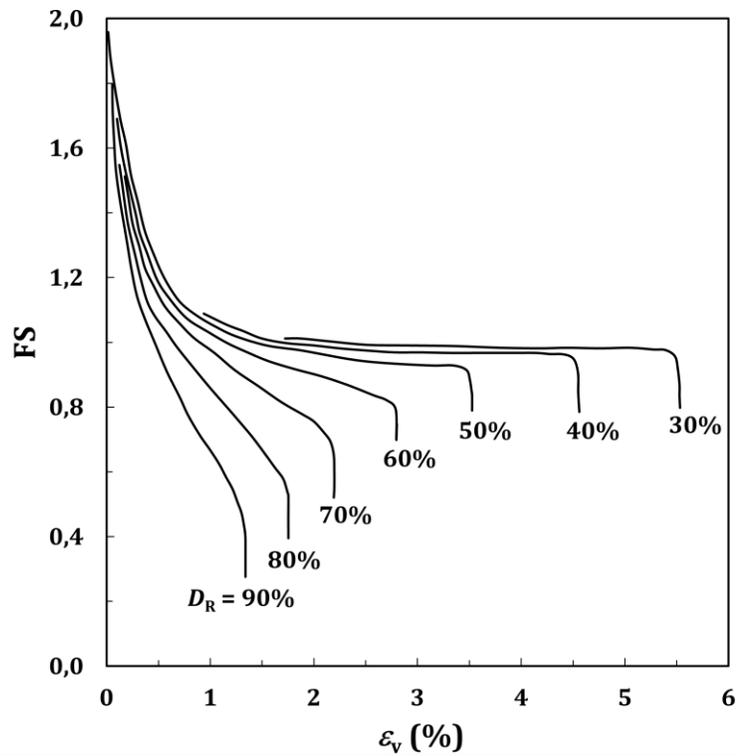
More specific and more informative compared with **2004 version**

- Susceptibility of **unsaturated loose, coarse-grained soils** to densification and settlements caused by cyclic stresses should be evaluated. Settlements and densification may be estimated using empirical relationships (Annex C).
- Settlements in saturated coarse-grained soils due to dissipation of excess pore water pressures due to earthquake should be considered (Annex C).
- Settlements in **soft fine-grained soils** due to cyclic degradation under ground shaking and dissipation of induced excess pore water pressures should be addressed.
- Densification and settlement potential of soils may also be evaluated with appropriate **cyclic laboratory tests**.

Annex C

- Free-field settlement in saturated sand

$$S_{ff} = \sum_{i=1}^n \varepsilon_{vi} \Delta z_i$$



Annex C

- Settlement under a building

$$\ln(D_S) = c_1 + 4,59 \ln Q_L - 0,42 (\ln Q_L)^2 + c_2 LBS + 0,58 \ln \left[\tanh \left(\frac{H_L}{6} \right) \right]$$

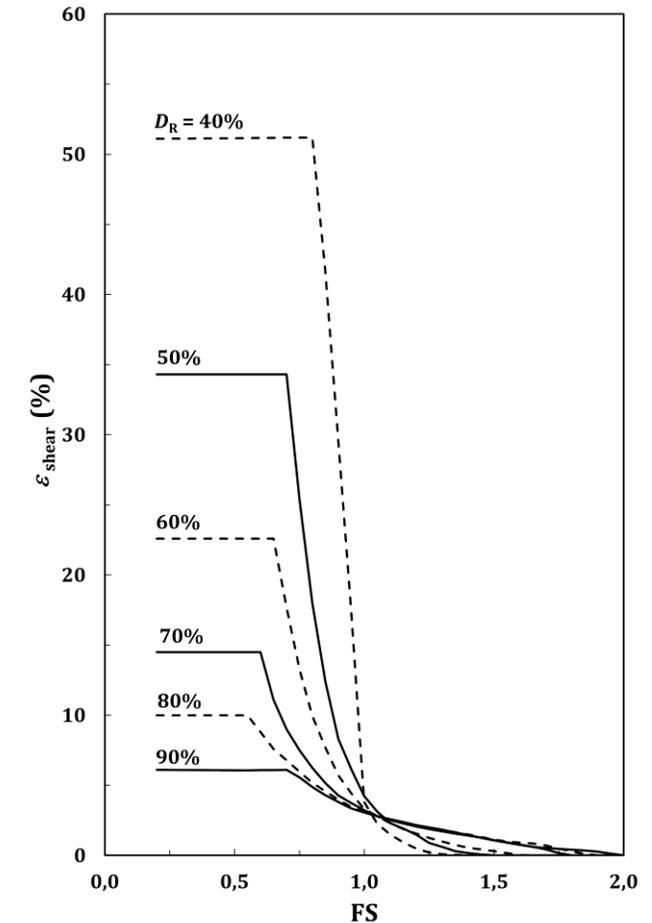
$$-0,02 B_b + 0,84 \ln \left(\frac{CAV_{dp}}{g} \right) + 0,41 \ln(S_1/g)$$

$$CAV_{dp} = \sum_{i=1}^n \left[H_{heav} (PGA_i - 0,25) \int_{t_{i-1}}^i |a(t)| dt \right]$$

- Lateral spreading due to liquefaction

$$\lg D_H = -16,71 + 1,532 M_w - 1,406 \lg R^* - 0,012 R + 0,592 \lg a_1 + 0,540 \lg a_2$$

$$+ 3,413 \lg(100 - a_3) - 0,795 \lg(a_4 + 0,1)$$



7.5 Site-specific response analyses

- When the relevant conditions in EN 1998-1-1 apply (namely, clauses 5.1.2(2) and 5.2.2.1 (4) related to special ground conditions or type of seismic analysis), the seismic actions required for the analyses in this chapter and those for foundations, retaining walls and underground structures (Chapters 8-11) should be derived from site-specific GRAs. For this purpose, one could use conventional **total stress methods** (per EN 1998-1-1 Annex B).
- If the ground response analysis is carried out in terms of **effective stresses**, a non-linear constitutive model (accounting for, for example, the volumetric and deviatoric behaviour of the soil and drainage conditions) should be considered.

Thank you for your attention