Second Generation of Eurocode 8

EN 1998-5:2021 (EC8 Part 5)

CHAPTER 8
Soil–Structure Interaction (SSI)

George GAZETAS
Consultant & Emeritus Professor
National Technical University of Athens

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This chapter deals only with rather general aspects of Soil–Structure Interaction (SSI).

Other chapters treat SSI aspects of shallow foundations, piles, retaining walls, and underground structures.

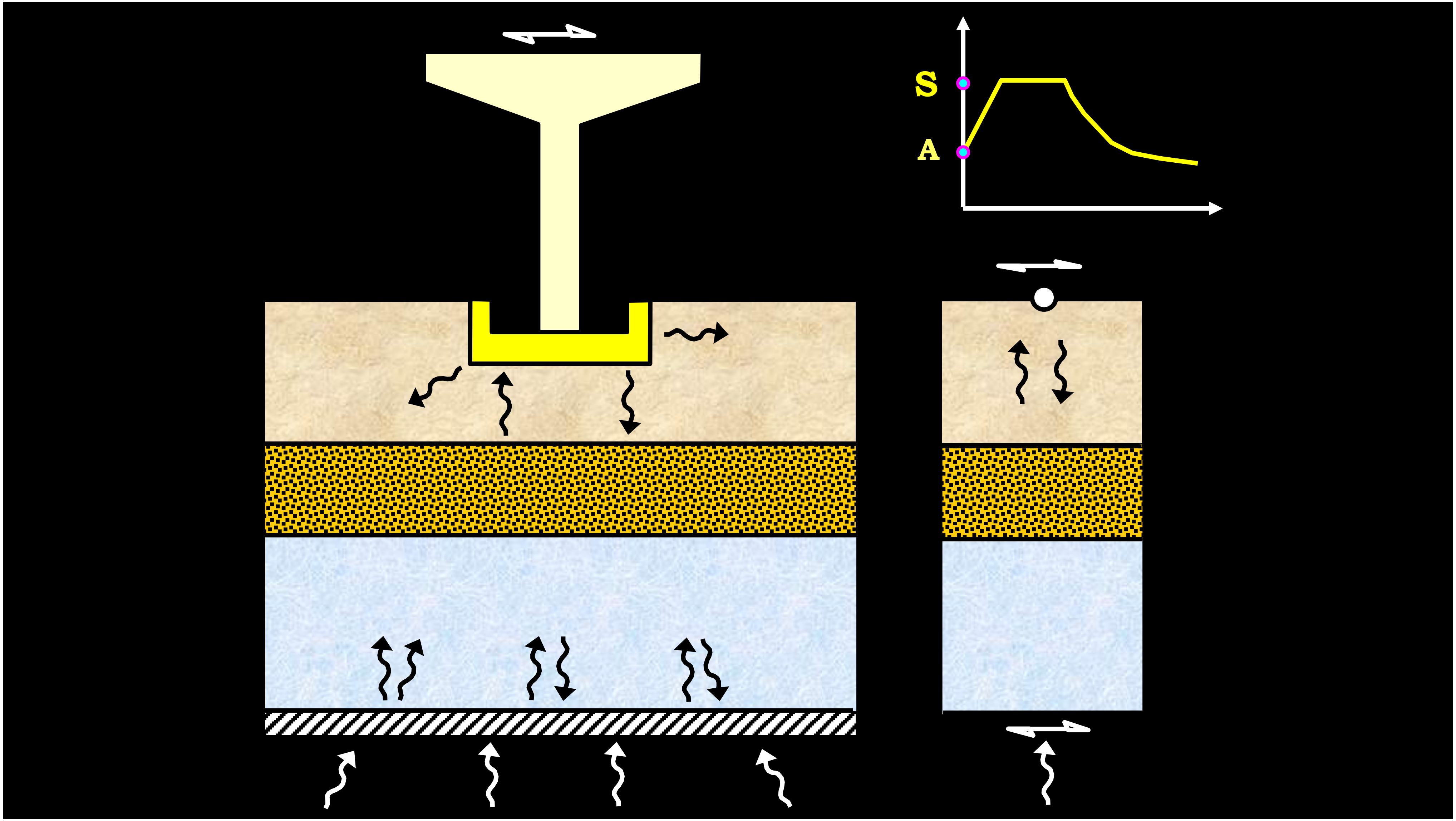
There are NO major differences from the old version of the Code. Simply more detailed description is provided for the tasks of the analysis.

8.1 General requirements

The analysis of seismic SSI effects should consider two effects:

a) Inertial effects that modify the dynamic response of the structure by changing the fundamental period and damping of the soil-structure system.

b) Kinematic effects that modify the seismic excitation at the base of the structure with respect to the free-field, and produce loading of foundation elements.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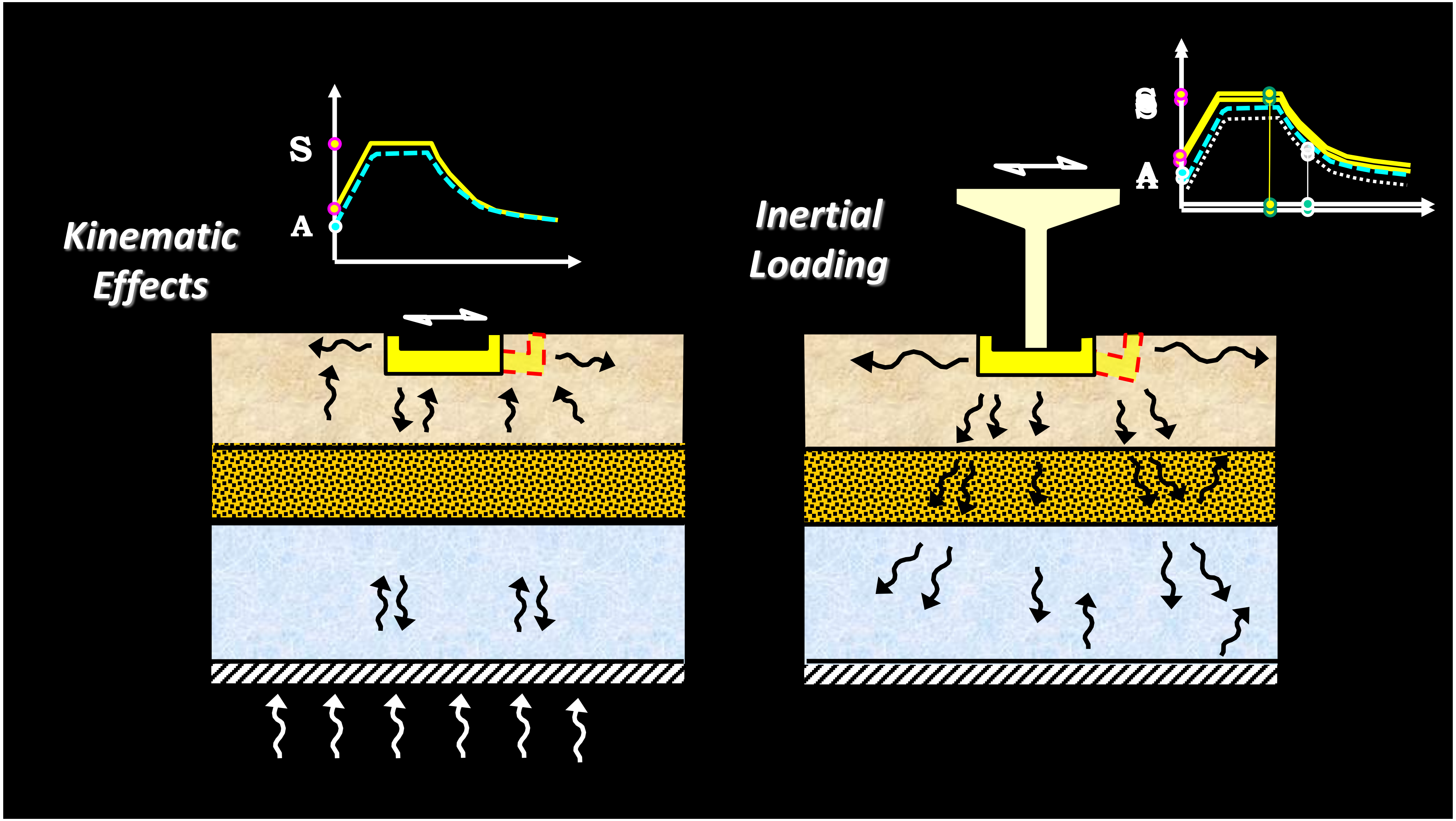
Kinematic
Effects
S
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Inertial
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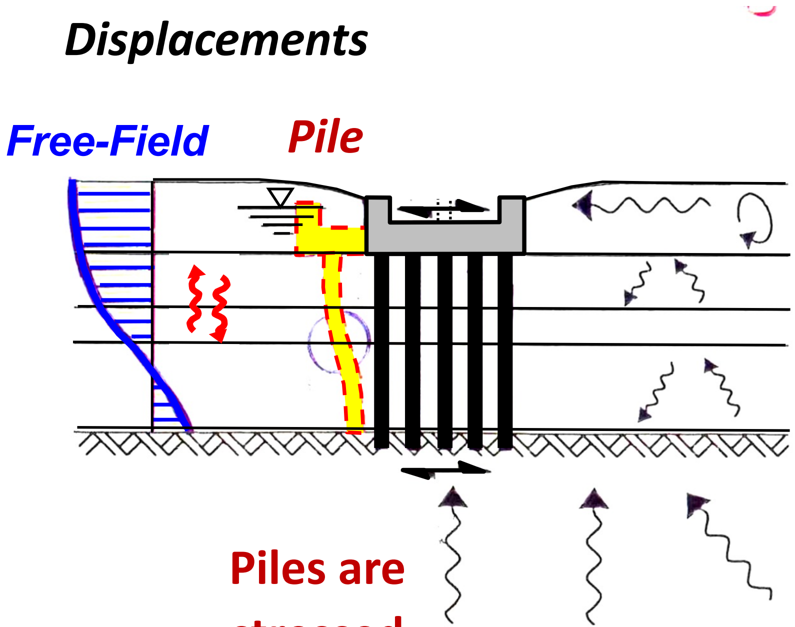
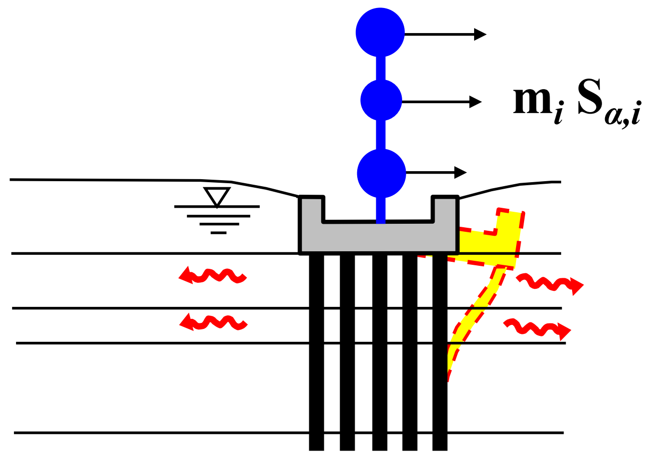
“Kinematic” Effects



Piles are stressed “kinematically”

+

“Inertial” Loading



Piles are stressed from “returning” inertia forces

(5) The inertial effects of SSI <u>should</u> be considered when at least one of the following applies:

a) When increasing the fundamental period increases spectral accelerations.

b) When the displacement of the structure controls the width of joints separating nearby buildings (existing or planned), or other performance criteria.

c) For structures supported on soft soils in which v_s averaged over a depth equal to 3 times the maximum foundation width in case of footings or to the maximum width in case of a raft foundation, is less < 250 m/s.

d) Structures with geometric non-linearity (*P – Δ effect*) plays a significant role.

8.1

(6) Kinematic Modification of Foundation input motion should be considered:

a) in case of deep foundations (piles, caissons)

b) foundations embedded to a depth of at least two floors, or to a depth > *L*/4, if the foundation vertical surfaces is in full contact with the surrounding ground

c) abutments of bridges with large embankments, or integral bridges without specific provisions for minimizing SSI effects

d) very large foundations with *L or B* > 50 m consisting of a slab, or a single box foundation, or footings interconnected with tie beams.

(7) For **flexible pile** foundations, **modification of the free-field** motion, as required in 8.1(6)a), **may be neglected** and the free-field motion may be used for the foundation input motion.

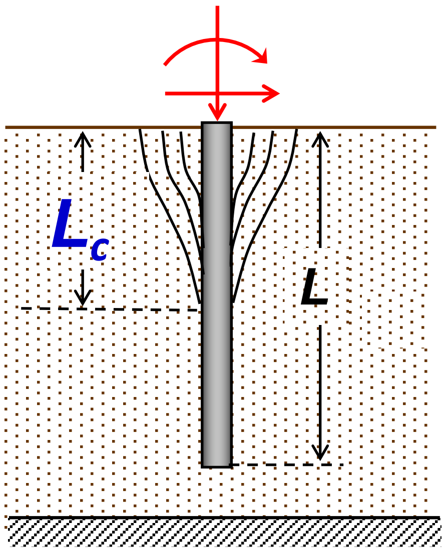
(8) A pile foundation may be considered as flexible when

$$E_P / E_S \leq (L_P / 1{,}5\, d)^4 \quad \text{from} \quad L_P \geq L_c \approx 1{,}5\, d\, \left(E_p/E_s\right)^{0.25}$$

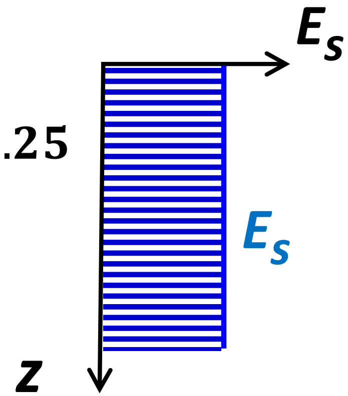
where L_P and d are the pile length and pile diameter.

(9) **Kinematic** interaction may be **neglected for the vertical** component of the seismic action.

active length L_c of flexible pile



$$L_c \approx 1.5d\left(E_p/E_s\right)^{0.25}$$



8.2 Analysis of inertial effects

(1) Seismic action effects on structure and foundations should be determined with suitable model of structure–foundation system supported on the ground.

The ground reaction may be represented by springs for all degrees of freedom.

NOTE A rigid foundation has 6 degrees of freedom, 3 translational (in x, y, z) and 3 rotational (rx, ry, rz , about the x, y and z axes).

(2) Coupling of horizontal and rotational springs should be considered for piled foundations, deeply embedded foundations, and caissons.

(3) For foundation shapes (circle, strip, rectangle), piles and ground profiles values for spring stiffnesses may be obtained from available elasticity-based solutions.

NOTE See Annex D for guidance to obtain stiffness and damping of foundations and piles.

Footing B x L on Homogeneous halfspace

$$K_{xx} = \frac{GB}{2-\nu}\left[1{,}2 + 3{,}3\left(\frac{L}{B}\right)^{0{,}65}\right]$$

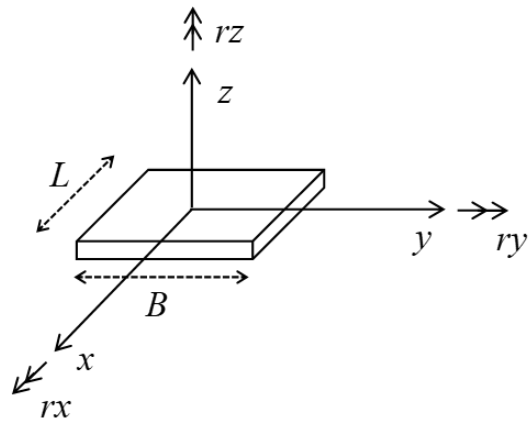
$$K_{rx} = \frac{GB^3}{8(1-\nu)}\left[0{,}4 + 3{,}2\left(\frac{L}{B}\right)\right]$$

$$K_{yy} = \frac{GL}{2-\nu}\left[2 + 2{,}5\left(\frac{B}{L}\right)^{0{,}85}\right]$$

$$K_{ry} = \frac{GB^3}{8(1-\nu)}\left[3{,}6\left(\frac{L}{B}\right)^{2{,}4}\right]$$

$$K_{zz} = \frac{GL}{1-\nu}\left[0{,}73 + 1{,}54\left(\frac{B}{L}\right)^{0{,}75}\right]$$

$$K_{rz} = \frac{GB^3}{8}\left[4{,}1 + 4{,}2\left(\frac{L}{B}\right)^{2{,}45}\right]$$



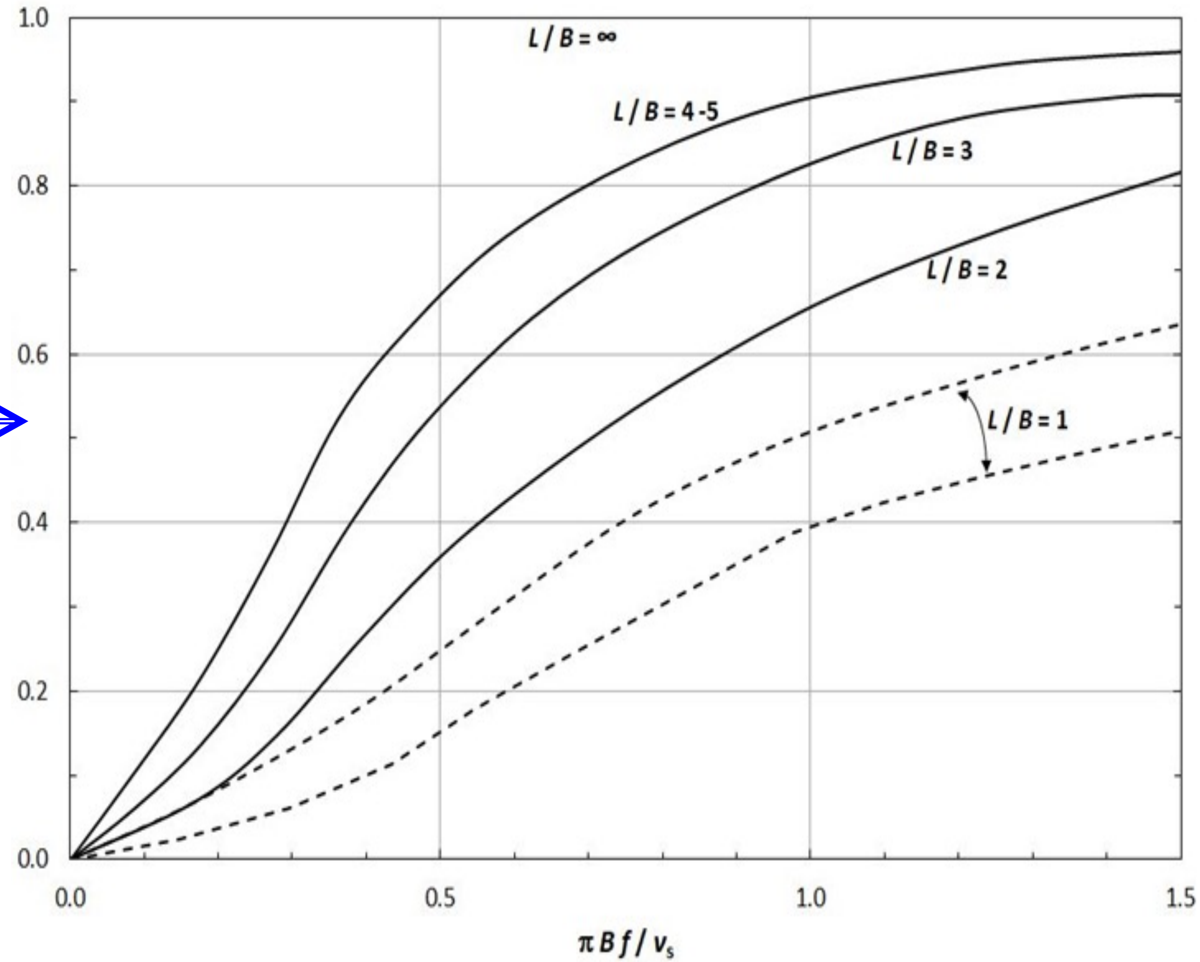
Translational modes

$$C_{xx} = C_{yy} \approx \rho v_s A_b$$

Rotational modes

$$C_{ry} = \frac{\rho v_s J_{by}}{1-\nu} c'_{ry}$$

On HOMOGENEOUS Halfspace



(4) Frequency-independent stiffness may be assigned to each spring, corresponding to the period of the fundamental mode, accounting for SSI in the considered direction. If this period is difficult to determine reliably, the static stiffnesses may be used instead.

(5) For design limit states SD and NC, the equivalent-linear stiffnesses for nonlinear springs to be used should be compatible with the amplitude of horizontal displacements and rotations of the foundation.

(6) To apply (5), the equivalent–linear stiffnesses of each spring may be calculated with the soil moduli compatible with the strain amplitude developed in the free-field.

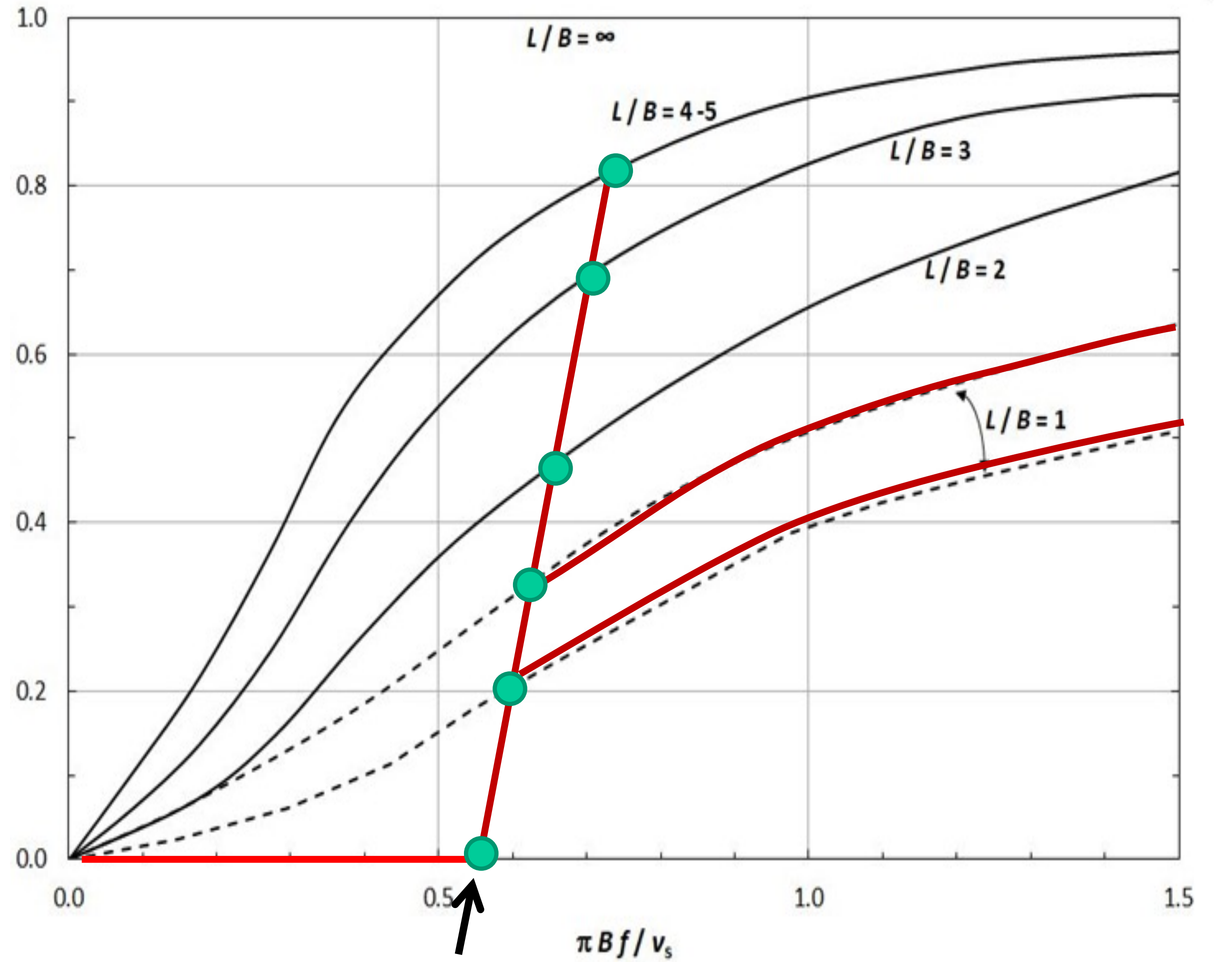
8.2.1 Force–based approach

(1) **Radiation damping <u>may</u> be used only for periods $T < T_o$ (the fundamental period of the soil deposit).**

Unless supported by numerical calculations which model the layers properties down to a depth where $v_s > 600$ m/s, radiation damping should be limited to 20 %.

(2) **Numerical analyses should comply with 8.5.**

$$C_{ry} = \frac{\rho v_s J_{by}}{1-\nu} c'_{ry}$$



$$f_o = 1 / T_o \approx v_s / 4H$$

8.2.2 Displacement–based approach

8.2.2.1 Nonlinear static analysis

(1) In non-linear static analysis of surface or shallow foundations, translational and rotational **inelastic springs** may be used.

(2) When springs are not used, the **lateral force–displacement relation** of the foundation-soil system under large deformations may be calculated from a suitable **non-linear static analysis** in which the inelastic ground is modelled by FE / FD.

The possibility of **uplift on the tension side** of the foundation, as well as **of** **slippage at the ground-foundation** contact surface, may be included in the model.

8.2.2.2 Time history analyses

(1) The effect of inertial SSI in time history analyses may be taken into account by modelling the foundation/ground system with springs and dashpots.

(2) A **frequency-independent stiffness** value may be assigned to each spring, corresponding to the **period of the fundamental mode**, accounting for SSI in the considered direction.

NOTE The frequency dependence of the springs and dashpots can be modelled in time history analyses with lumped models of constant springs, dashpots and masses.

(3) Radiation damping (C_α) may be added to material damping (ξ): $C_{\alpha t} = C_\alpha + \xi \frac{K_\alpha T}{\pi}$

NOTE 1 **Annex D** provides guidance for stiffness and damping.

NOTE 2 Radiation damping is strongly affected by ground layering. Solutions for a homogeneous elastic half-space result in unrealistically large values of damping.

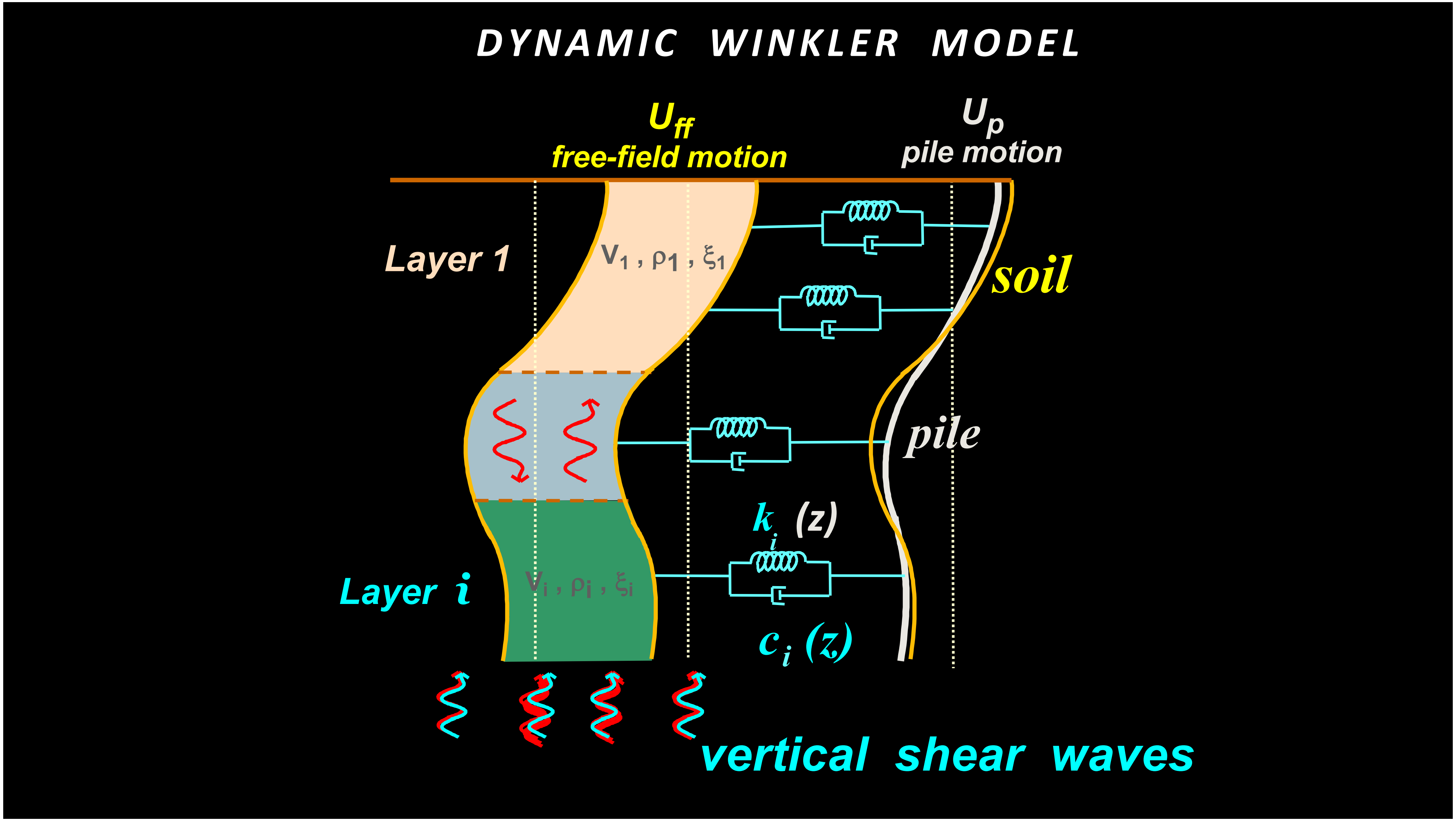
8.3 Modelling of kinematic effects

(1) Kinematic interaction effects may be calculated in accordance with 8.5 as part of the whole structure-foundation-soil system, or with a separate analysis in which only the foundation, without mass, and the soil are included.

(2) The second type of analysis in (1) may be performed either through FE/FD.

For **piles a suitable Winkler type model** may be used with lateral soil springs and dashpots representing the action of the soil in contact with the foundation elements.

(3) In FE/FD of pile–soil system, the seismic excitation should be imposed at the base of soil stratum and lateral boundaries should be capable of deforming as the free-field.


DYNAMIC WINKLER MODEL
U_ff
free-field motion
U_p
pile motion
Layer 1
V_1 , ρ_1 , ξ_1
soil
pile
k_i (z)
Layer i
V_i , ρ_i , ξ_i
c_i (z)
vertical shear waves

(4) With Winkler modelling, ground should be discretised into horizontal layers. One-dimensional ground response analysis should be conducted to obtain the time-histories of displacement at each layer. These displacements should be imposed at the supports of the lateral springs-and-dashpots.

(5) With Winkler modelling, an alternative to (4) may be used to impose the ground displacements by representing the action of the surrounding ground with a shear beam connected to the free ends of the springs and dashpots.

(6) In (5), the shear beam should have masses an order of magnitude larger than the pile masses.

(7) To obtain the induced **bending moments** in a pile, the analysis in (4) may replace the time histories of displacements with the respective **peak values to be imposed statically** at the supports of the springs, with the dashpots neglected.

8.4 Combination of inertial and kinematic effects for internal forces

(1) If inertial and kinematic effects are evaluated separately, the forces in the foundation elements from the two analyses **may be combined according** to either a) or b):

a) when the frequency of the mode contributing most to the SSI response differs by more than 15% from the fundamental frequency of the soil deposit, the action effects are **combined with SRSS rule** (square root of the sum of the squares)

b) when the condition in a) is not satisfied, the **absolute values** of the action effects of the two analyses are **summed up**.

8.5 Simultaneous modelling of kinematic and inertial effects

(1) Dynamic time-history analysis of the **whole structure-foundation-soil** system (FE/FD)

(2) The analysis model for (1) should allow for the **transmission of seismic waves** across the lateral and bottom boundaries of the system.

NOTE 1 Improper modelling of the boundary conditions creates wave spurious reflections
NOTE 2 Accurate modelling of the relevant frequencies of the structure requires FE sizes smaller than 1/6 λ (frequency domain solutions) to 1/10 λ (time domain solution).
λ is the smallest wavelength of interest.

(3) Base excitation acceleration time histories should be compatible with the elastic response spectrum.

THANK YOU

FOR YOUR ATTENTION