Pile size limitations in seismic regions

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Acknowledgements

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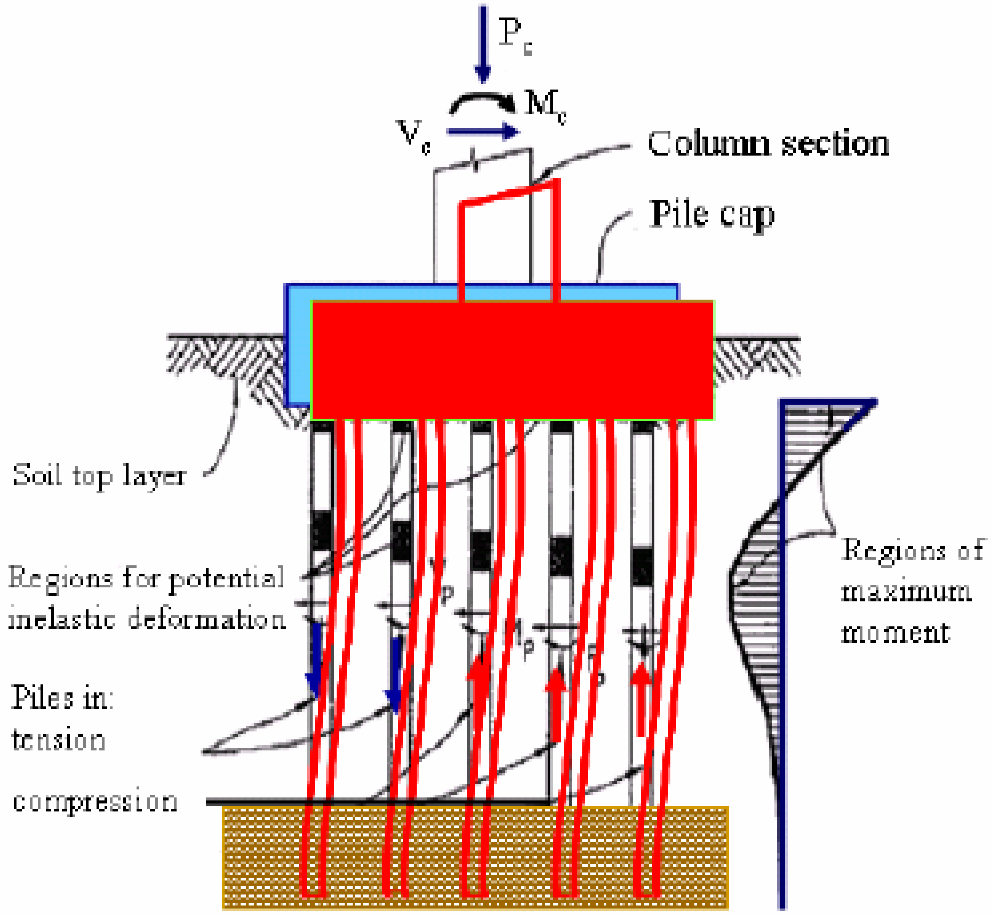
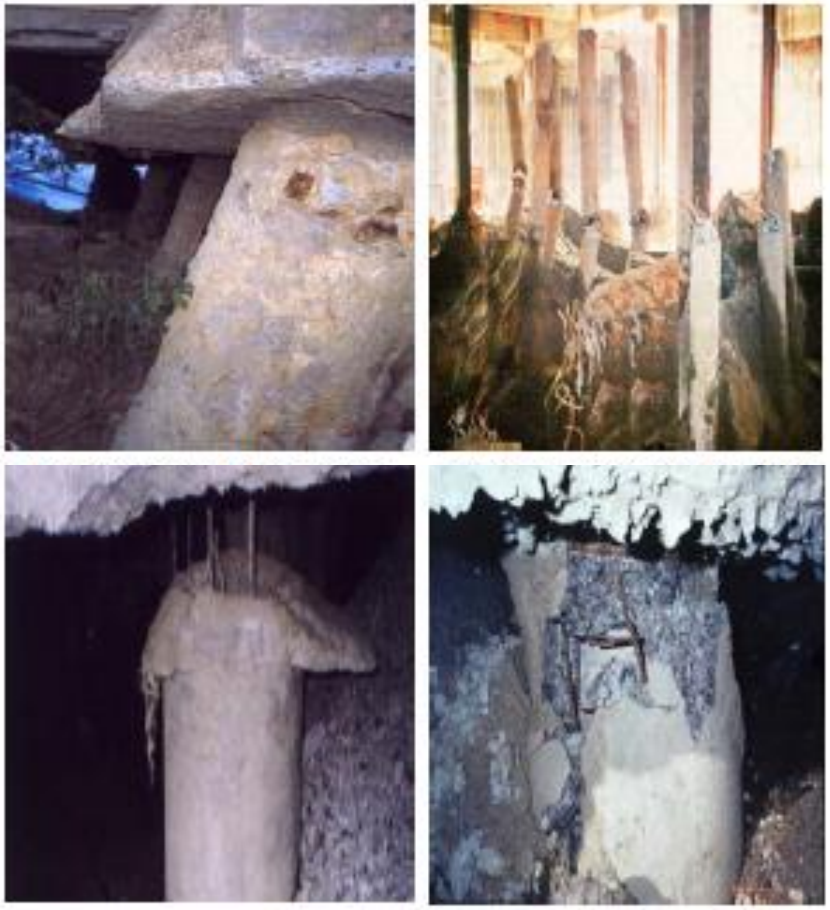
Examples of observed pile head failures

Christchurch, NZ, 2011

Niigata, Japan, 1964

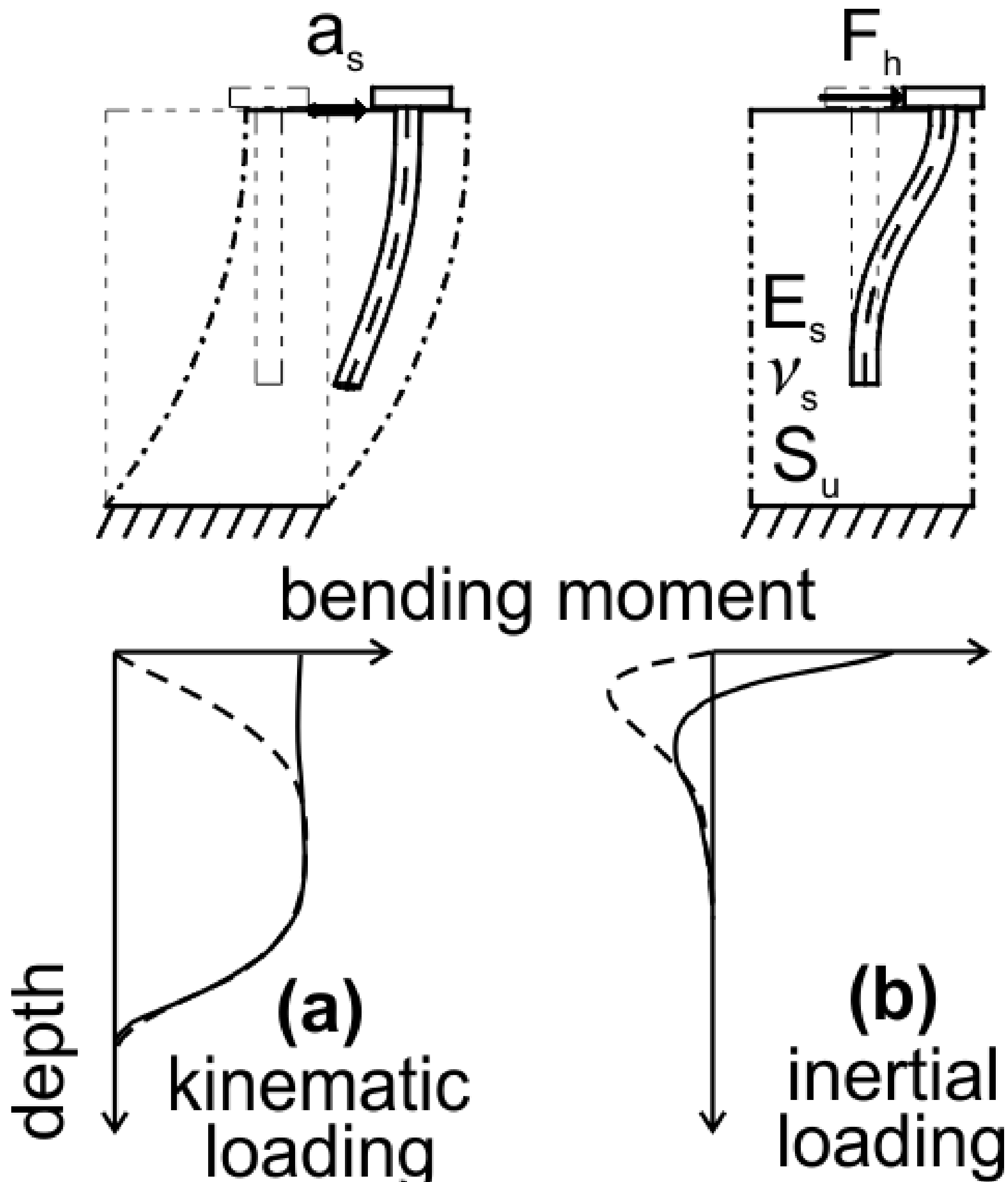
Pile failures





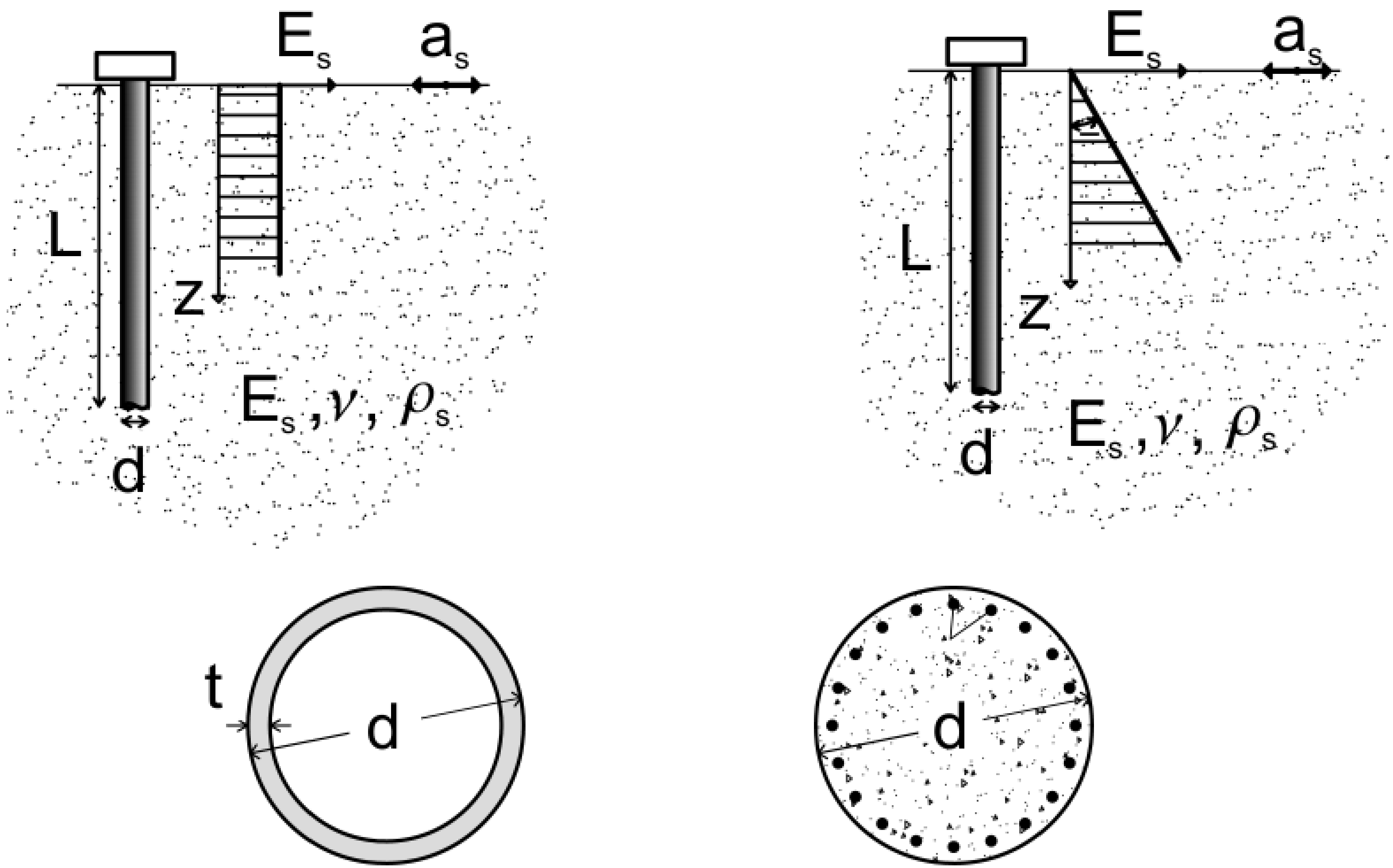
Hamada 1991 Mizuno 1987

Kinematic and Inertial Pile Loading



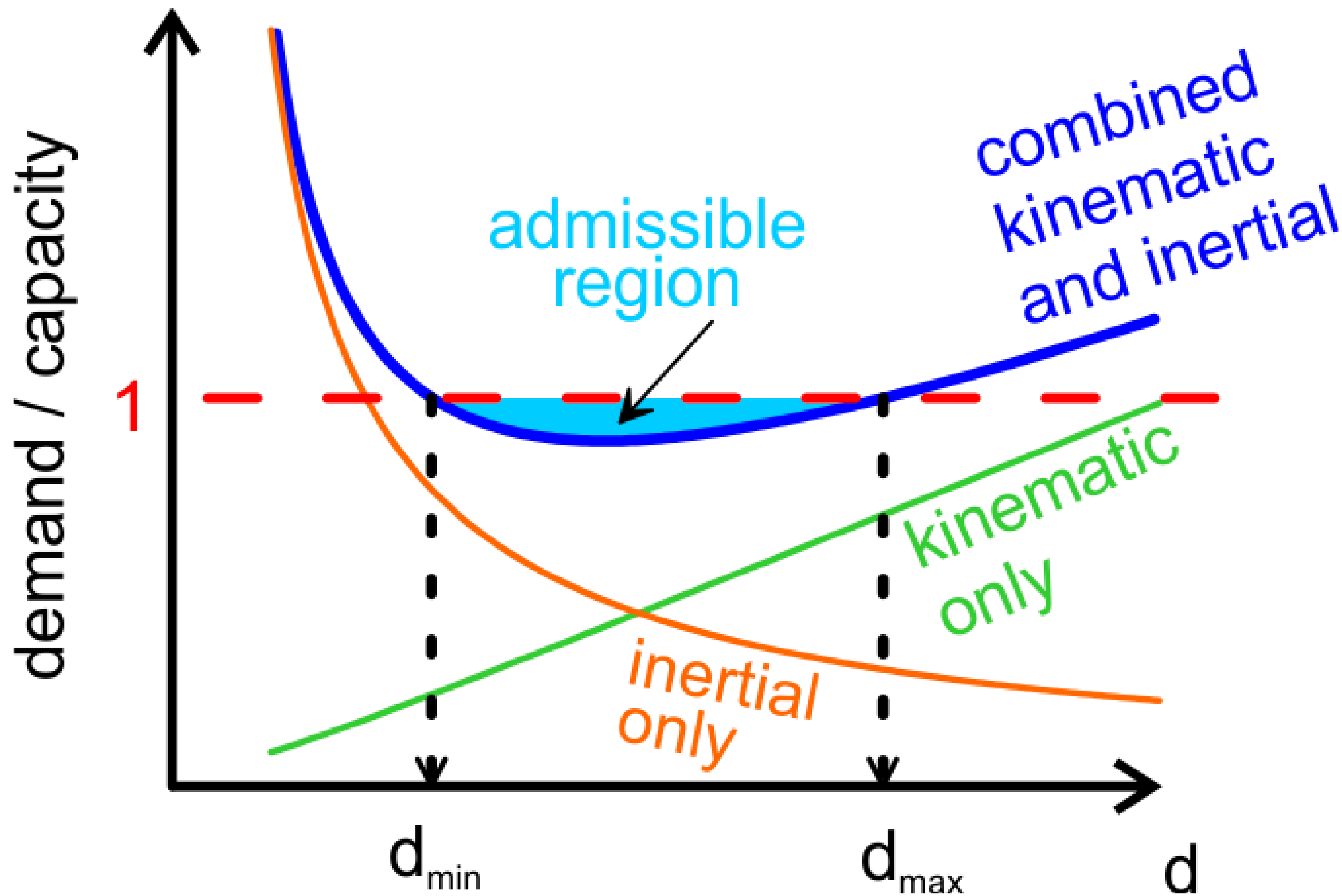
Soil Profiles & Pile Types

(a) homogeneous profile (b) linear profile



(I) steel section (II) concrete section

Effect of Pile Diameter on Pile Kinematic & Inertial Bending



Part A

Steel piles, homogeneous soil

Kinematic bending

steel piles, homogeneous soil

$$\left(1/R\right)_p = \Psi\left(1/R\right)_s$$

$$M_{head}^{kin} = E_p I_p \left(1/R\right)_p \approx E_p I_p \left(1/R\right)_s = E_p I_p \frac{a_s}{V_s^2} \quad \propto \boldsymbol{d^4}$$

Yield Moment

$$M_y = E_p I_p \varepsilon_y \frac{2}{d}\left(1 - \frac{P_p}{f_y A}\right) \propto \boldsymbol{d^3}$$

$$P_p = \frac{1}{SF}\left[\pi \alpha L d + N_c A\right] s_u$$

Limit diameter for kinematic loading

$$\frac{1}{2\varepsilon_y}\frac{a_s L}{V_s^2}\left(\frac{d}{L}\right)^2 - (1-T_1)\left(\frac{d}{L}\right) + \frac{4\alpha}{q_A\, SF}\frac{s_u}{f_y} = 0$$

$$T_1 = \frac{N_c\, s_u}{q_A\, SF\;\, f_y} \quad , \quad q_A = 1 - (1-2t/d)^2$$

$$\boxed{d_{kin} = 2\varepsilon_y \frac{V_s^2}{a_s}(1-T_1)\left[\frac{1}{2} \pm \sqrt{\frac{1}{4} - \frac{2\alpha}{\varepsilon_y\, q_A\, SF}\left(\frac{V_s^2}{a_s L}\right)^{-1}\left(\frac{s_u}{f_y}\right)(1-T_1)^{-2}}\,\right]}$$

$$d_{kin} = 2\varepsilon_y \frac{V_s^2}{a_s}$$

Maximum diameter!

Limit diameter for inertial loading

$$M_{in} = \frac{1}{4}\left(\frac{\pi\, q_I}{\delta}\right)^{\frac{1}{4}}\left(\frac{a_s}{g}\right)\left(\frac{E_p}{E_s}\right)^{\frac{1}{4}} S_a\, P_p\, d \propto \boldsymbol{d^2}$$

$$\boxed{d_{in} = \frac{8\alpha}{SF\left(1-T_2\right)} L \left[\frac{S_a}{\varepsilon_y}\left(\frac{\pi}{\delta}\right)^{\frac{1}{4}}\left(\frac{a_s}{g}\right)\left(q_I\,\frac{E_p}{E_s}\right)^{-\frac{3}{4}}\left(\frac{s_u}{E_s}\right) + \frac{1}{2q_A}\left(\frac{s_u}{f_y}\right)\right]}$$

$$T_2 = T_1\left[1 + 8\left(\frac{\pi q_I}{\delta}\right)^{\frac{1}{4}}\left(\frac{a_s}{g}\right)\left(\frac{E_p}{E_s}\right)^{\frac{1}{4}}\frac{S_a q_A}{q_I}\right]$$

Minimum diameter!

Combined kinematic and inertial loading

$$M_{tot} = e_{kin} M_{kin} + e_{in} M_{in}$$

$$\frac{1}{2}\frac{a_s L}{V_s^2}\left(\frac{d}{L}\right)^2 - \left(1 - T_3\right)\varepsilon_y\left(\frac{d}{L}\right) + \frac{4\alpha}{q_A SF}\left(\frac{s_u}{E_p}\right)\left[1 + 2\frac{q_A}{q_I}\left(\frac{\pi q_I}{\delta}\right)^{\frac{1}{4}}\left(\frac{a_s}{g}\right)\left(\frac{E_p}{E_s}\right)^{\frac{1}{4}} S_a\right] = 0$$

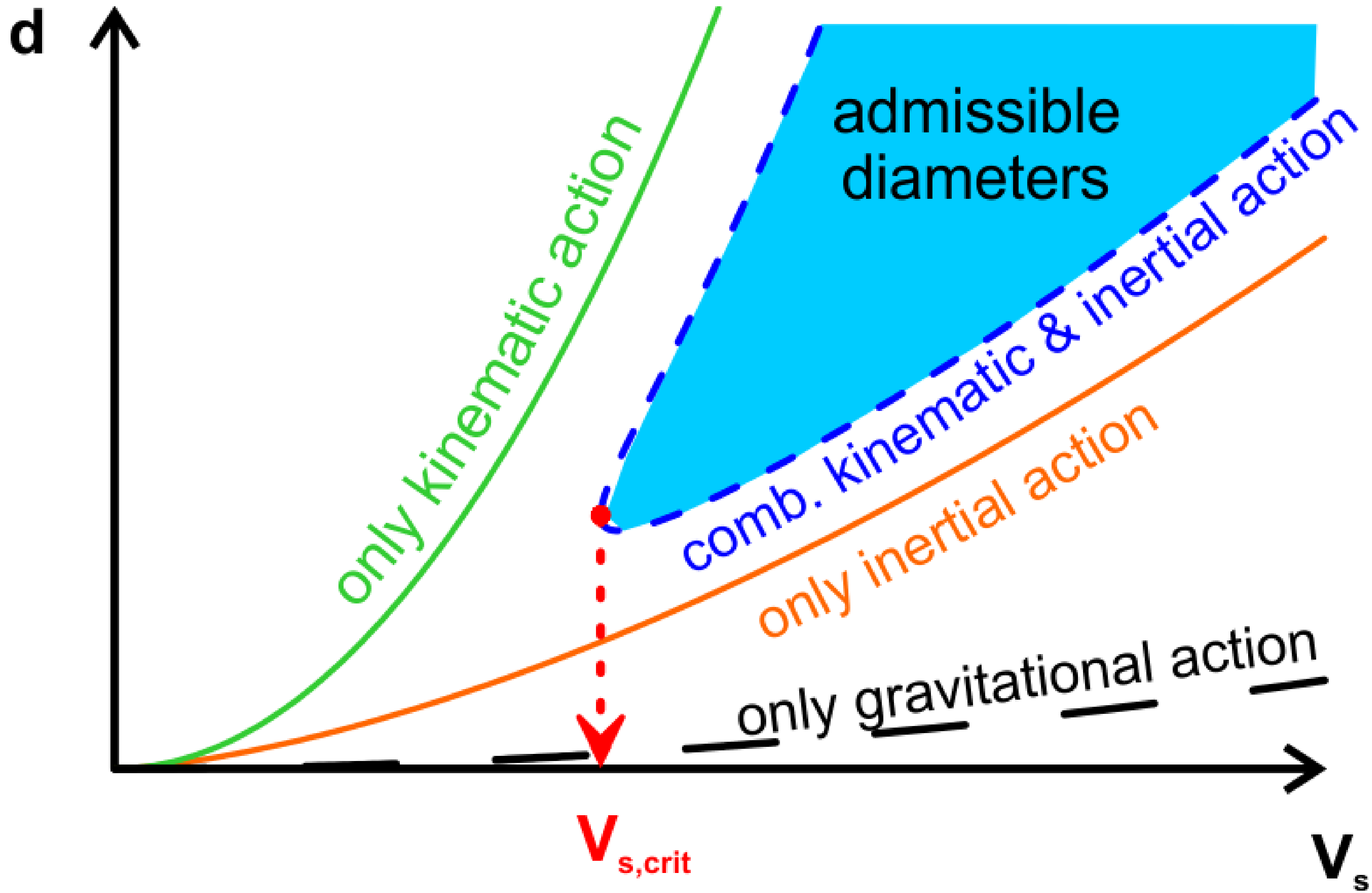
$$\boxed{d_{1,2} = \frac{\varepsilon_y V_s^2}{a_s}\left(1 - T_3\right)\left\{1 \mp \sqrt{1 - \frac{24\alpha\,\rho_s\, a_s L}{\left(1 - T_3\right)^2 q_A\, f_y\, \varepsilon_y\, SF}\left(\frac{s_u}{E_s}\right)\left[1 + 2\frac{q_A}{q_I}\left(\frac{\pi q_I}{\delta}\right)^{\frac{1}{4}}\left(\frac{a_s}{g}\right)\left(\frac{E_p}{E_s}\right)^{\frac{1}{4}} S_a\right]}\right\}}$$

$$T_3 = \left[\frac{1}{q_A\, SF\;\, f_y} + \frac{2}{\varepsilon_y}\left(\frac{\pi}{\delta}\right)^{\frac{1}{4}}\left(\frac{q_I E_p}{E_s}\right)^{-\frac{3}{4}}\left(\frac{s_u}{E_s}\right)\right] N_c$$

Minimum & maximum diameters!

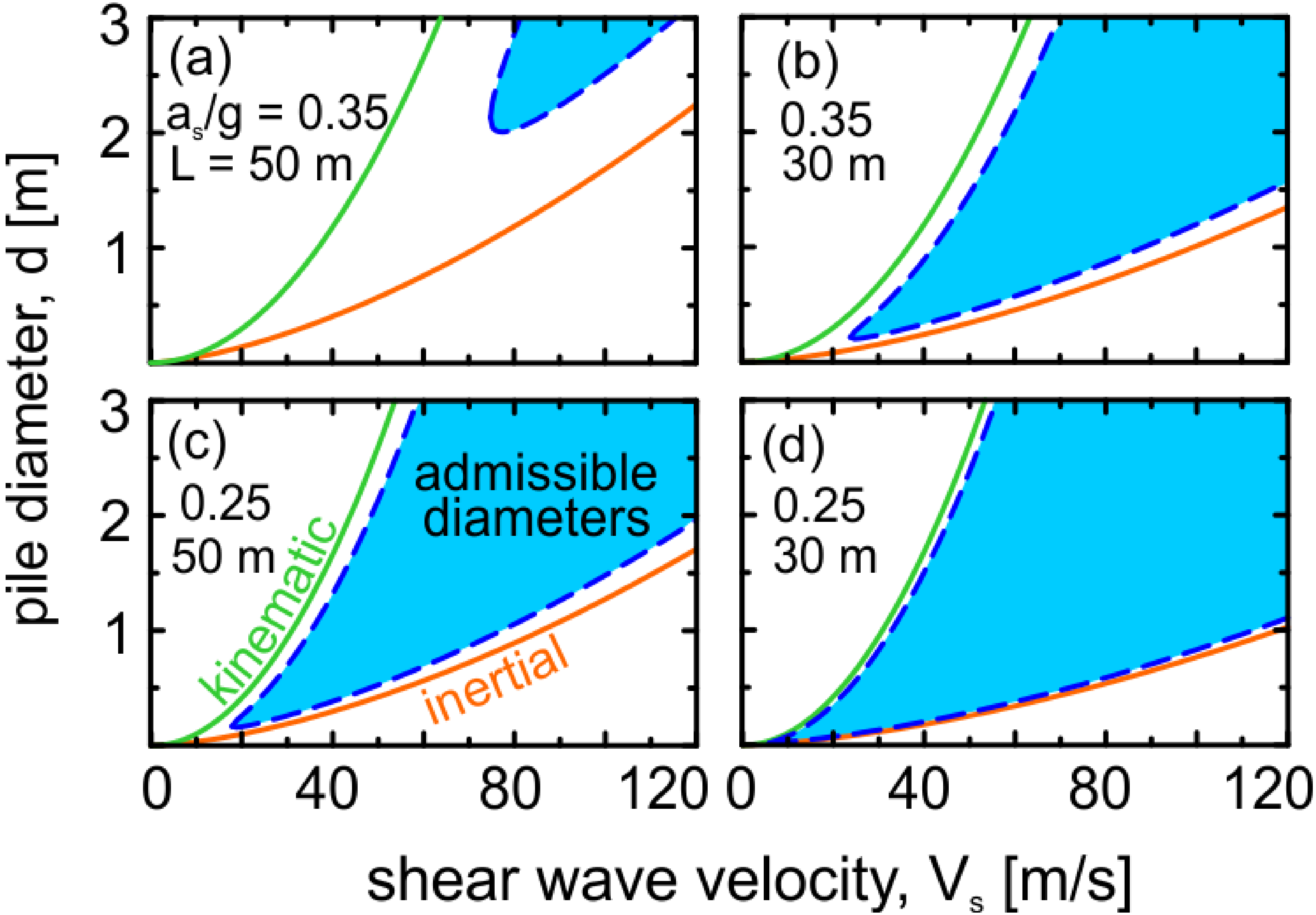
Admissible and inadmissible diameters

for different types of loading



Admissible pile diameters against Vs

E_s/S_u = 500, $f_{yk,s}$ = 275 MPa, E_p = 210 GPa, ν_s = 0.5, ρ_s = 1.7 Mg/m^3, S_a = 2.5, FS = 3, t/d = 0.015, α = 0.7, δ = 1.2, T1 = 0



Limit shear wave velocity

$$V_{s,crit} = \left(\frac{E_p}{\rho_s}\right)^{\frac{1}{2}} \left[\frac{2\dfrac{q_A}{q_I}\left(\dfrac{\pi q_I}{3\delta}\right)^{\frac{1}{4}} \dfrac{a_s}{g} S_a}{\dfrac{q_A \varepsilon_y^2 SF}{24\alpha}\left(\dfrac{E_s}{s_u}\right)\left(\dfrac{E_p}{a_s \rho_s L}\right) - 1} \right]^2$$

$$d_1 = d_2 = \frac{\varepsilon_y V_s^2}{a_s}\left(1 - T_3\right)$$

for V_s<50m/sec, maximum pile diameter d_2<1m

Part B

Steel piles, soil with stiffness varying proportional with depth

Kinematic bending

steel piles, soil with stiffness varying proportional with depth

$$E(z) = \bar{E}_s \cdot z$$

$$M_{kin} = 0.185 a_s \rho_s \left(\frac{q_I E_p}{\bar{E}_s} \right)^{\frac{4}{5}} d^{\frac{16}{5}} \propto \boldsymbol{d^{3.2}}$$

$$M_{in} = 1.6 \frac{S_a L \alpha S_u}{SF} \left(\frac{a_s}{g} \right) \left(\frac{q_I E_p}{\delta \bar{E}_s} \right)^{\frac{1}{5}} d^{\frac{9}{5}} \propto \boldsymbol{d^{1.8}}$$

Limit diameters for combined loading

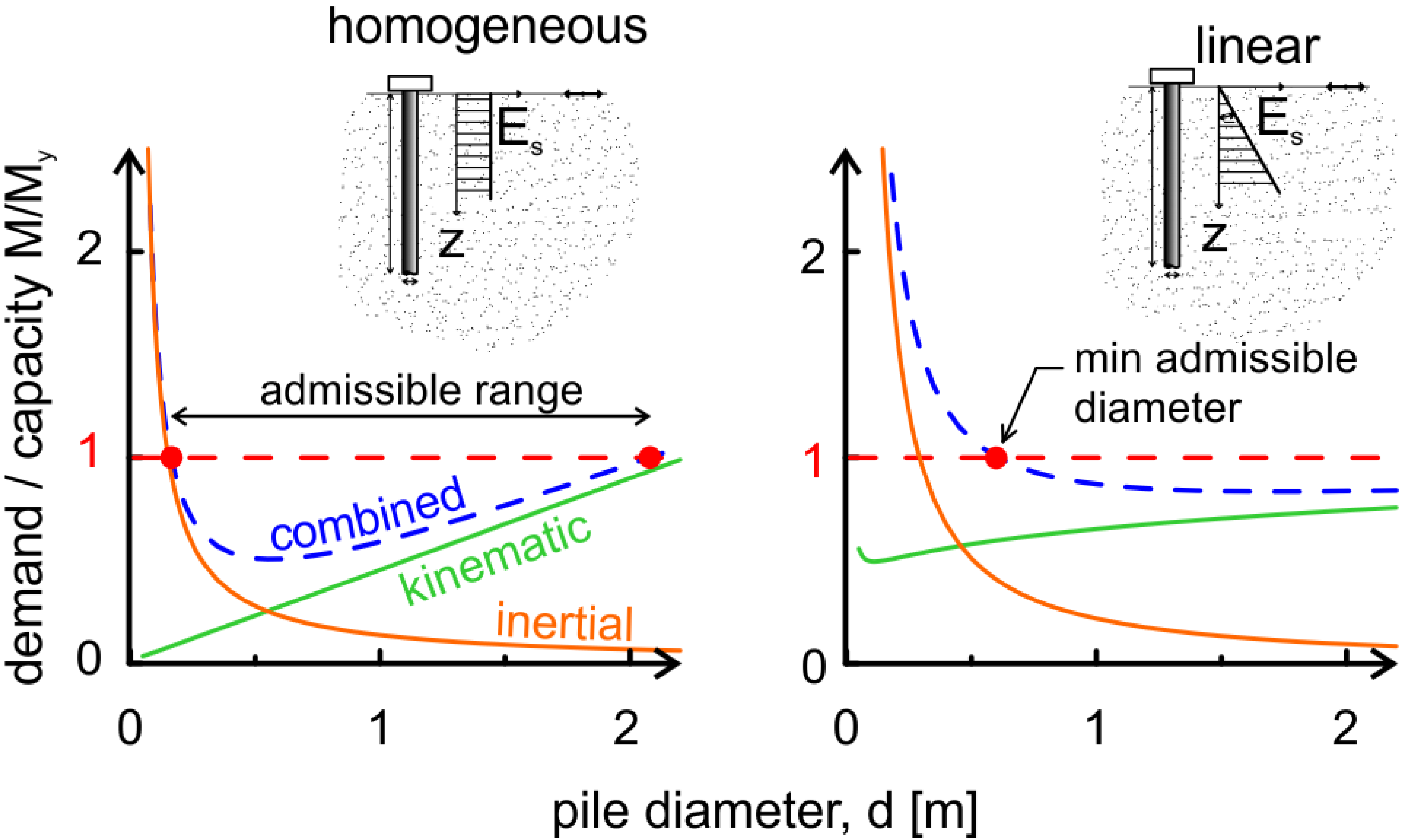
$$0.185\left(\frac{q_I E_p}{\bar{E}_s L}\right)^{\frac{4}{5}}\left(\frac{d}{L}\right)^{\frac{16}{5}} - \frac{\pi}{64}\left(\frac{q_I E_p \varepsilon_y}{a_s \rho_s L}\right)\left(\frac{d}{L}\right)^{3} + \frac{\pi}{16}\frac{q_I \alpha S_u}{q_A SF\, a_s \rho_s L}\left(\frac{d}{L}\right)^{2} + 1.6\frac{S_a \alpha S_u}{SF\,\gamma L}\left(\frac{q_I E_p}{\delta \bar{E}_s L}\right)^{\frac{1}{5}}\left(\frac{d}{L}\right)^{\frac{9}{5}} = 0$$

$$\boxed{d \approx \frac{\alpha S_u}{SF\,\gamma}\,\frac{1.6 S_a\left(\dfrac{q_I E_p}{\delta \bar{E}_s L}\right)^{\frac{1}{5}} + \dfrac{\pi}{16}\dfrac{q_I}{q_A a_s / g}}{0.185\left(\dfrac{q_I E_p}{\bar{E}_s L}\right)^{\frac{4}{5}} - \dfrac{\pi}{64}\left(\dfrac{q_I E_p \varepsilon_y}{a_s \rho_s L}\right)}}$$

Minimum diameter!

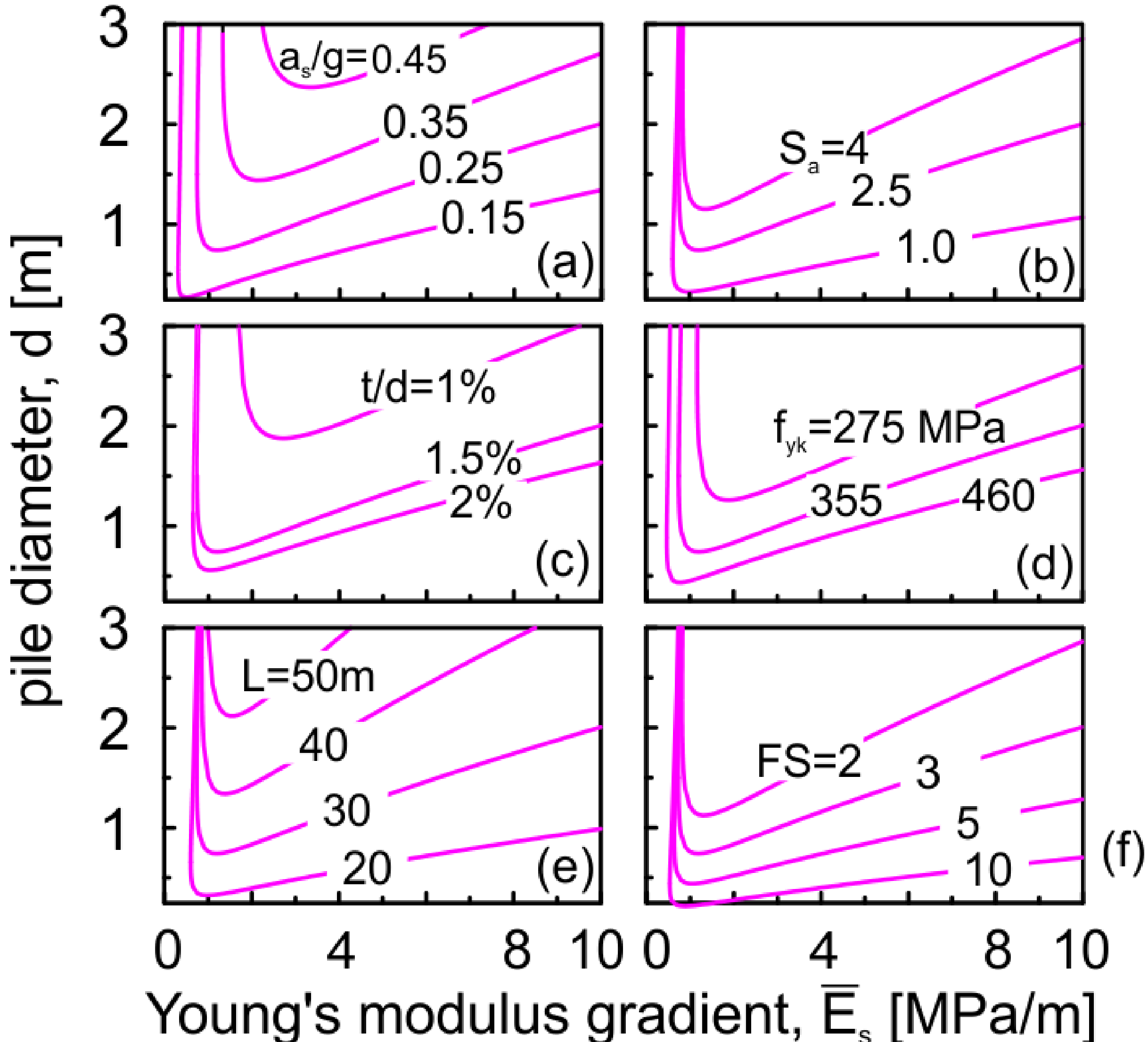
Homogeneous vs Linear Soil Profile

a_s/g = 0.35, E_s/S_u = 500, $f_{yk,s}$ = 275 MPa, E_p = 210 GPa, ν_s = 0.5, ρ_s = 1.7 Mg/m^3, S_a = 2.5, FS = 3, t/d = 0.015, α = 0.5, L = 15 m, E'_s= 2 MPa/m, E_s =E'_s ,L/2 = 15 MPa



Admissible diameters: steel pile with depth proportional stiffness

$a_s/g = 0.25$, $E_s/S_u = 500$, $f_{yk,s} = 355$ MPa, $E_p = 210$ GPa, $\nu_s = 0.5$, $\rho_s = 1.7$ Mg/m^3, $S_a = 2.5$, FS = 3, t/d = 0.015, $\alpha = 0.5$, L =30 m



Part C

Concrete piles

Moment Capacity

$$\boxed{M_u = M_{u,c} + M_{u,s} = \frac{2}{3}\left(\frac{d}{2}\right)^3 \sin^3\theta\, f'_{ck} + \frac{2}{\pi}\left(\frac{d}{2} - c\right) A_s \sin\theta\, f_{yk}}$$

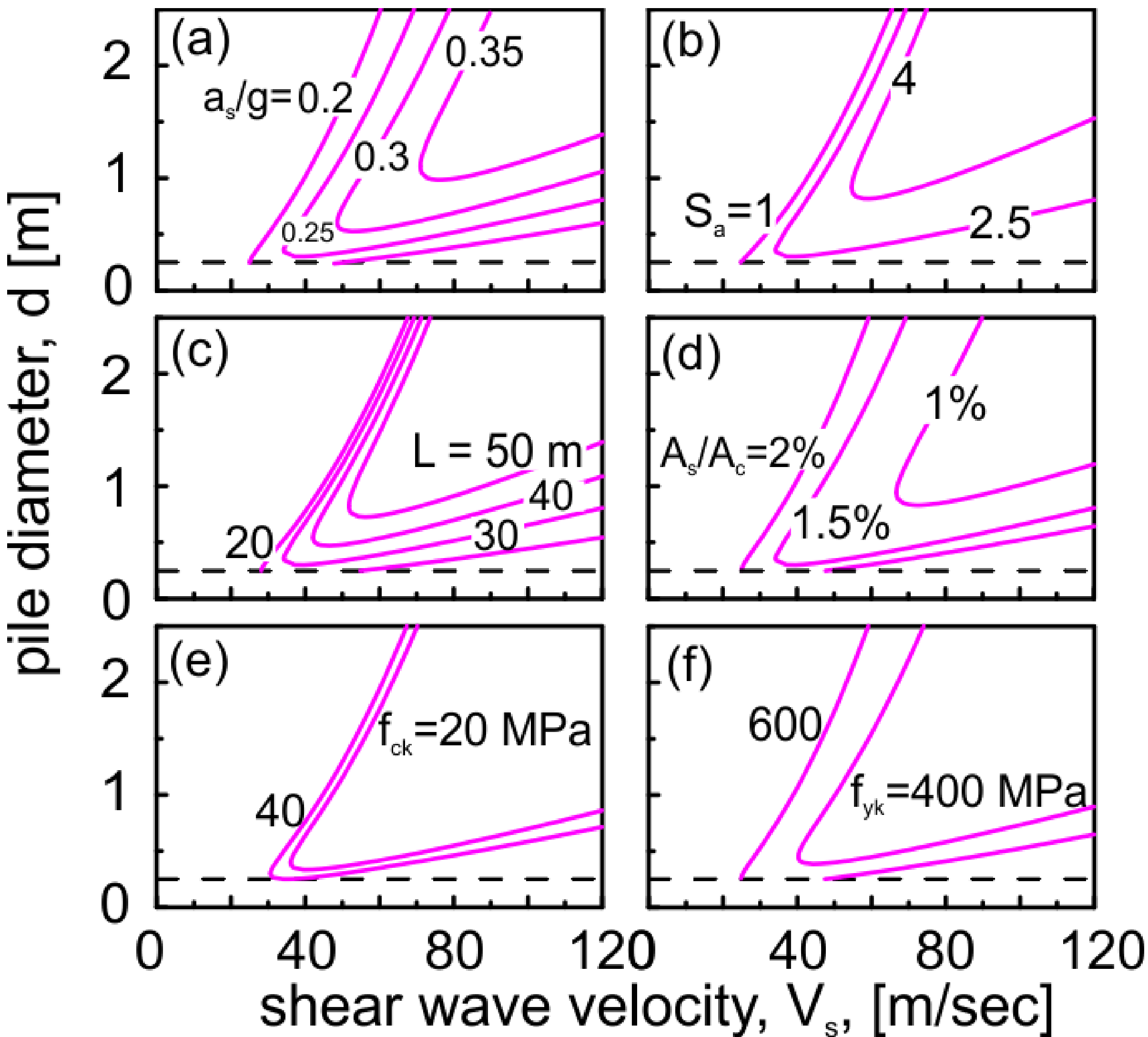
$$2\theta\left(1 + 2\omega\right) - \sin 2\theta - 2\pi\left(\omega + \nu_k\right) = 0$$

$$\omega = A_s\, f_{yk} / (A_c f'_{ck}) \quad , \quad \nu_k = W_p / (A_c f'_{ck})$$

$$\theta = \left(\frac{\pi}{4}\right)^2 \left(1 + 2\omega - \frac{4}{\pi}\right)\left[-1 + \sqrt{1 + \frac{32}{\pi}\frac{\omega + \nu_k}{\left(1 + 2\omega - 4/\pi\right)^2}}\right]$$

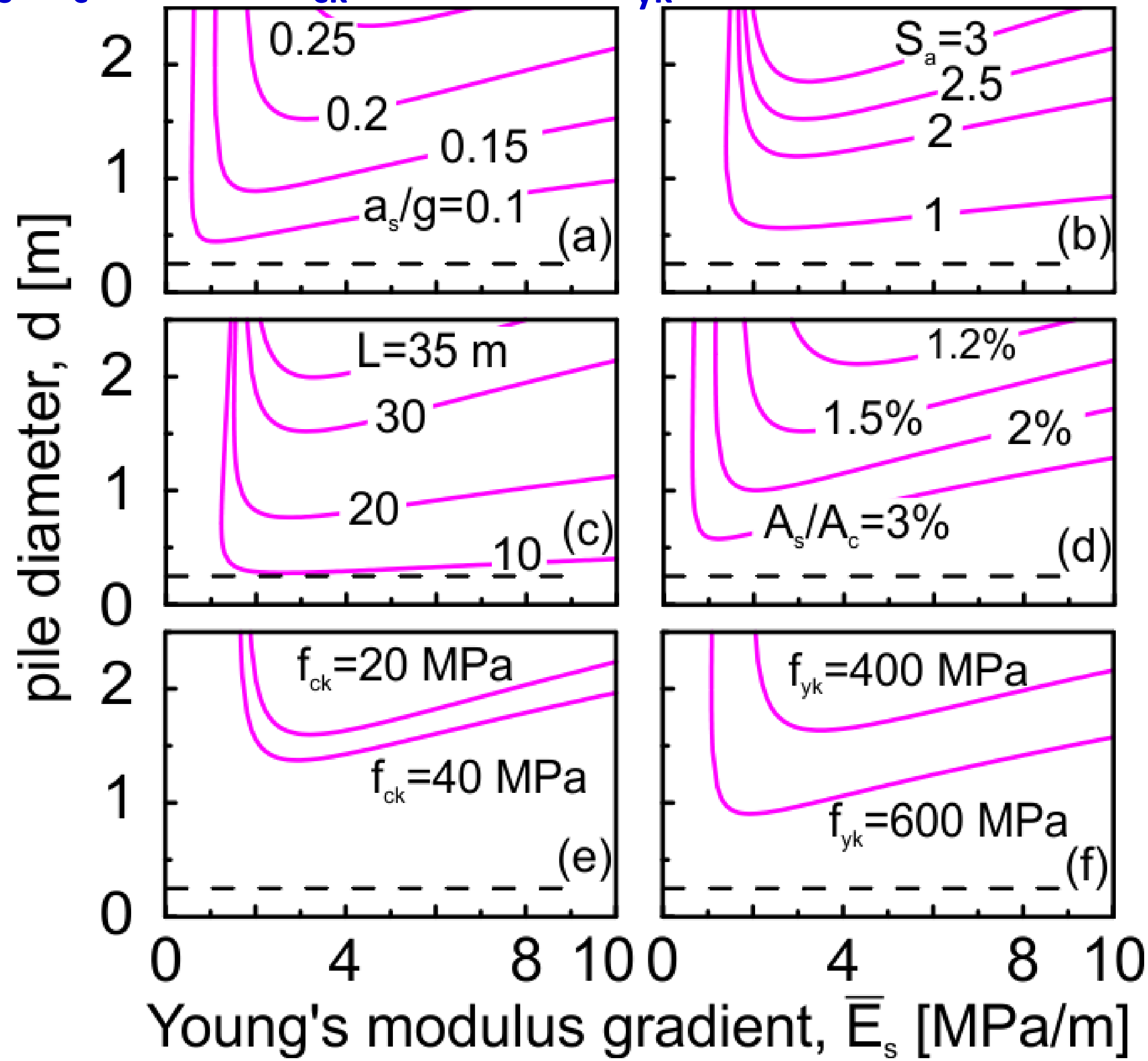
Admissible diameters: concrete pile in homogeneous soil

$a_s/g = 0.25$, $E_s/S_u = 500$, $E_p = 30$ GPa, $\nu_s = 0.5$, $\rho_s = 1.7$ Mg/m$_3$, $S_a = 2.5$, FS = 3, $A_s/A_c = 0.015$, $f_{ck} = 25$ MPa, $f_{yk} = 450$ MPa, $\alpha = 0.5$, L = 30 m



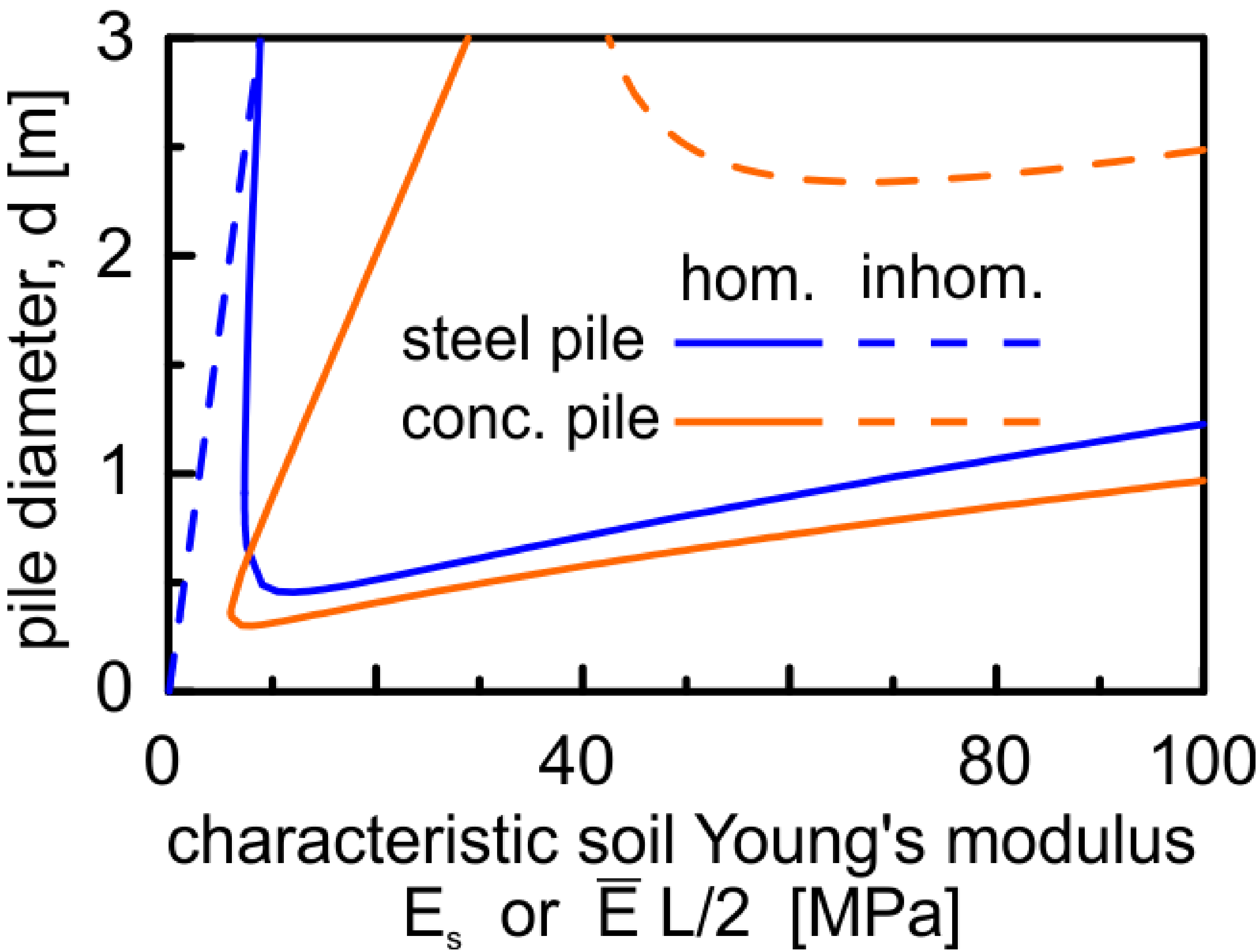
Admissible diameters: concrete pile with depth proportional stiffness

a_s/g = 0.2, E_s/S_u = 500, E_p = 30 GPa, ν_s = 0.5, ρ_s = 1.7 Mg/m^3, Sa = 2.5, FS = 3, A_s/A_c= 0.015, f_{ck} = 25 MPa, f_{yk} = 450 MPa, α = 0.5, L = 30 m



Steel vs Concrete Piles in Homogeneous and Linear Soil Profile

a_s/g = 0.25, E_s/S_u = 500, $f_{yk,s}$ (steel) = f_{yk} (concrete reinforcement) = 450 MPa, f_{ck} = 25 MPa, E_p = 30 GPa or 210 GPa (for concrete and steel, respectively), ν_s = 0.5, ρ_s = 1.7 Mg/m^3, S_a = 2.5, FS = 3, t/d = A_s/A_c = 0.015, α = 0.5, L = 30 m



Conclusions

- Concrete piles possess a narrower range of admissible diameters to withstand seismic action over hollow steel piles. This can be attributed to the higher bending stiffness of the concrete pile cross-section (which attracts higher kinematic moments), as well as the inability of the concrete material to carry tension.

- For soft soils of constant stiffness with depth, kinematic interaction dominates seismic demand. As a result, admissible pile sizes are essentially over-bounded by a critical diameter which, in some cases, may be quite small (~1 m) and, hence, may affect design. Under these circumstances, adding more piles or increasing pile length will not improve safety, as such remedial solutions do not affect kinematic demand.

- In stiffer soils, inertial interaction is prominent due to the heavier load carried by the pile under a constant *FS*. This yields a minimum admissible pile diameter which, in regions of moderate to high seismicity, may be quite large (~ 1 m).

Conclusions (cont'd)

- Soils with stiffness increasing proportionally with depth essentially enforce only a lower bound on pile diameter, which may be rather large (> 2 m), especially for strong stiffness gradients. Note that the absence of an upper limit is not due to weak kinematic demand. On the contrary, in such soils the ratio of kinematic over inertial moment may be larger than unity, yet the kinematic moment does not strongly depend on diameter.

- The range of admissible diameters decreases with increasing design ground acceleration, spectral amplification, soil strength and pile length, whereas it increases with increasing soil stiffness, pile safety factor and amount of reinforcement (or wall thickness). On the other hand, pile material strength plays a minor role in controlling pile size.

Conclusions (cont'd)

- There is always a critical soil shear wave velocity or stiffness gradient below which no pile diameter is admissible for a given design ground acceleration. Below this threshold, a fixed-head pile cannot stay elastic regardless of diameter or material strength. In the extreme case where $V_s = 0$ (e.g., a pile in water), no diameter is apparently admissible. This behavior should not be viewed as paradoxical, since then a_s would also be zero. Exploring the interplay between V_s and a_s lies beyond the scope of this study.

- Pile-soil contact stresses due to kinematic interaction are not expected to be important at low frequencies and do not induce major nonlinearities into the soil. The Authors also recognize the lack of documented case histories demonstrating the effects discussed herein. This may be attributed to an insufficient number of observations involving multiple pile diameters under restraining caps in soils with sufficiently low shear wave propagation velocities (<100m/s) to trigger this effect.

References

- Di Laora R, Mylonakis G, Mandolini A. (2013) Pile-head kinematic bending in layered soil. *Earthquake Engineering & Structural Dynamics*, 42: 319-337.

- Mylonakis G, Di Laora R, Mandolini A. (2014) The role of pile diameter on earthquake induced bending. 15th European Conference on Earthquake Engineering, Istanbul, August 24-29. In "Percpectives on European Earthquake Engineering and Seismology" Springer 533-556.

- Di Laora R, Mylonakis G, Mandolini A. (2016) Size limitations for piles in seismic regions. Earthquake Spectra (under review).

Thank you!!!

Ευχαριστώ!!!

Grazie!!!