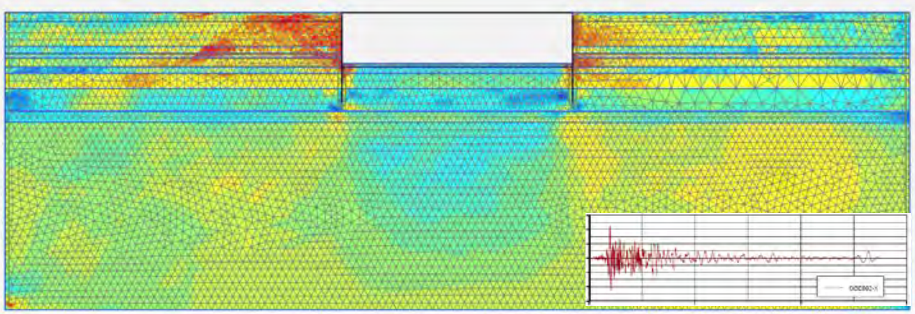
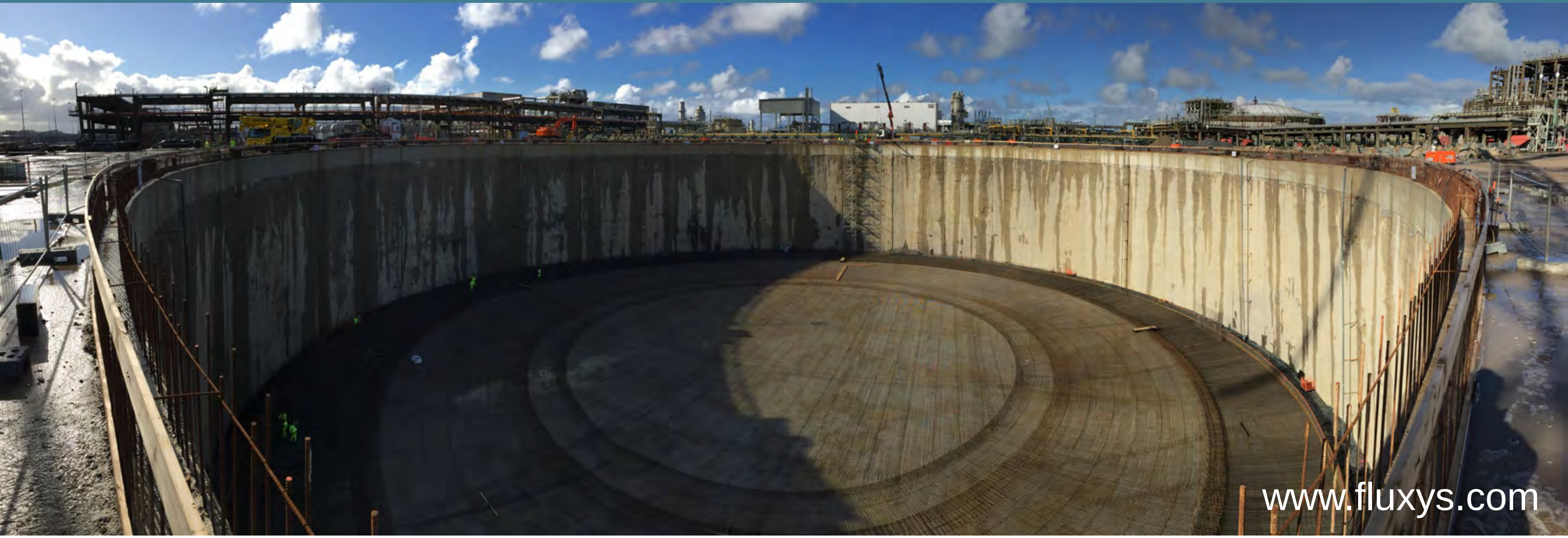


Diaphragm wall at location of the 5th Fluxys LNG Tank in Zeebrugge

Valérie Whenham

Journée d'étude Franco-Belge – Dimensionnement sismique des fondations – 15 Mars 2018

Fifth Storage Tank – Zeebrugge LNG Terminal (FLUXYS)



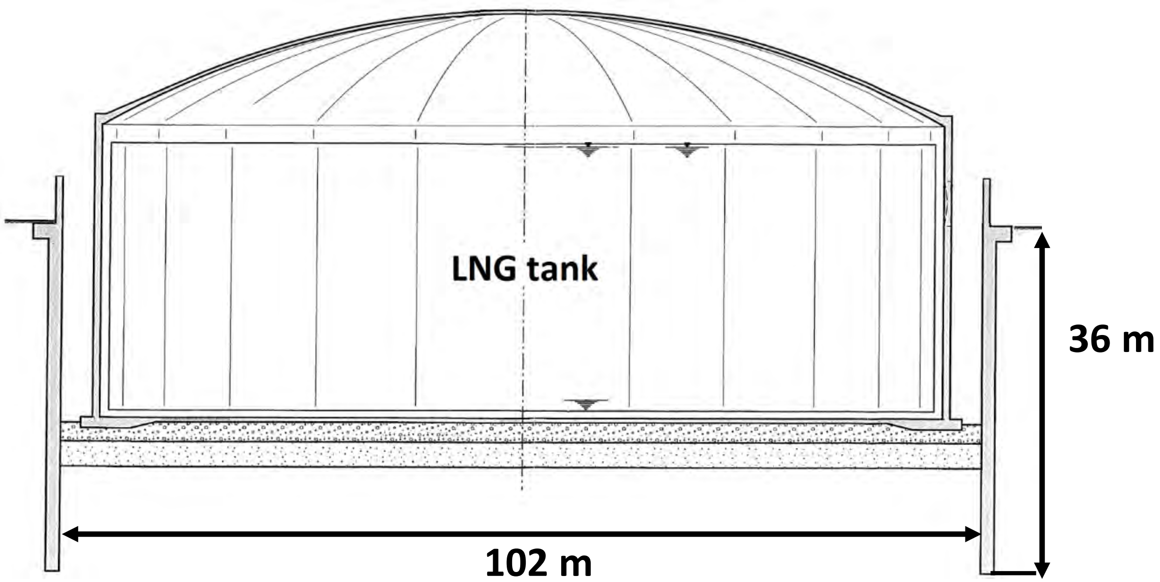
Tank diameter: 95 m
Useful capacity of the tank: 180,000 m³

Length of diaphragm wall panels: 36 m
Construction of the diaphragm wall dated November 2016

Preliminary design: dated 2014

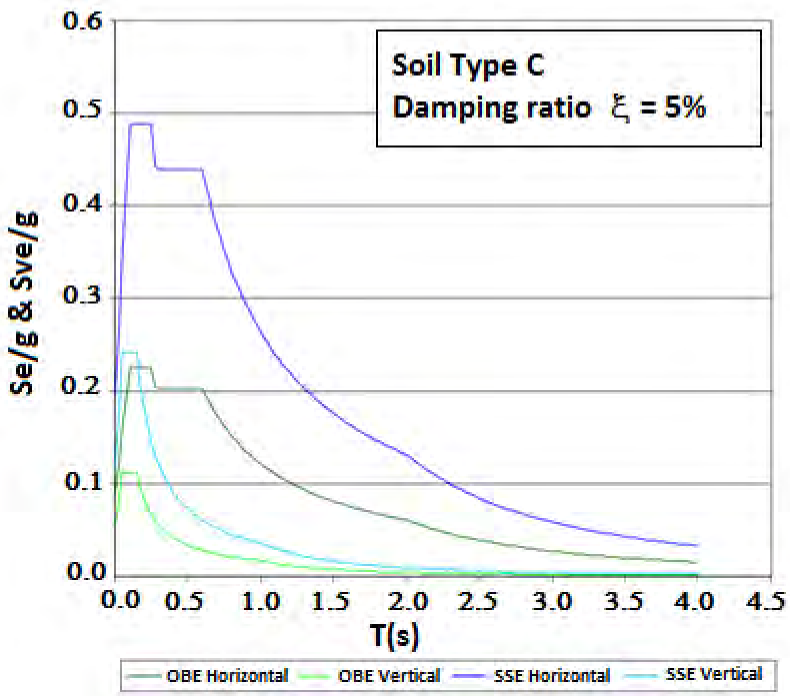
Plan of the presentation

- Ground Motion Parameters
- Stratigraphy and Key Soil Parameters
- Assessment of Liquefaction Potential
- Seismic Design
- Results and Conclusions

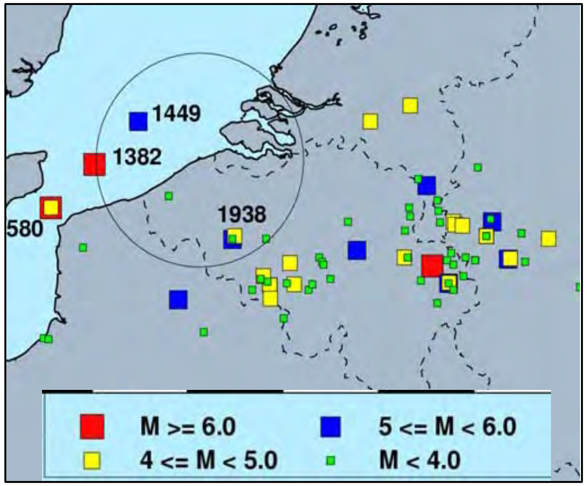


Ground Motion at the location of the project (Zeebrugge)

- Seismotectonic study by the Royal Observatory of Belgium
  - Operating Basis Earthquake (OBE): Return period = 475y
  - Safe Shutdown Earthquake (SSE): Return period = 10,000y (EN 1473-1997)



Elastic response spectra (32m BGL)

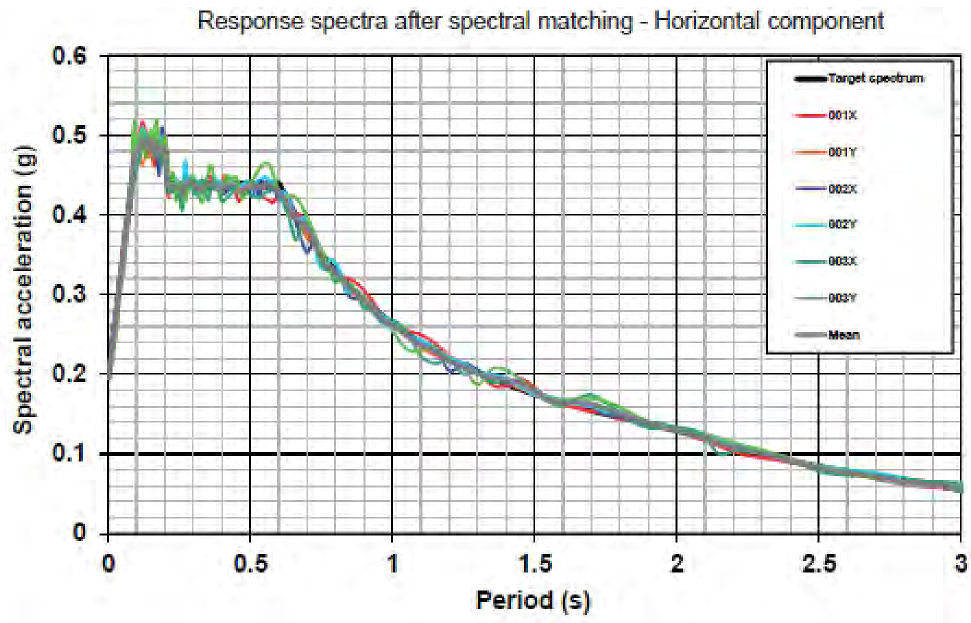


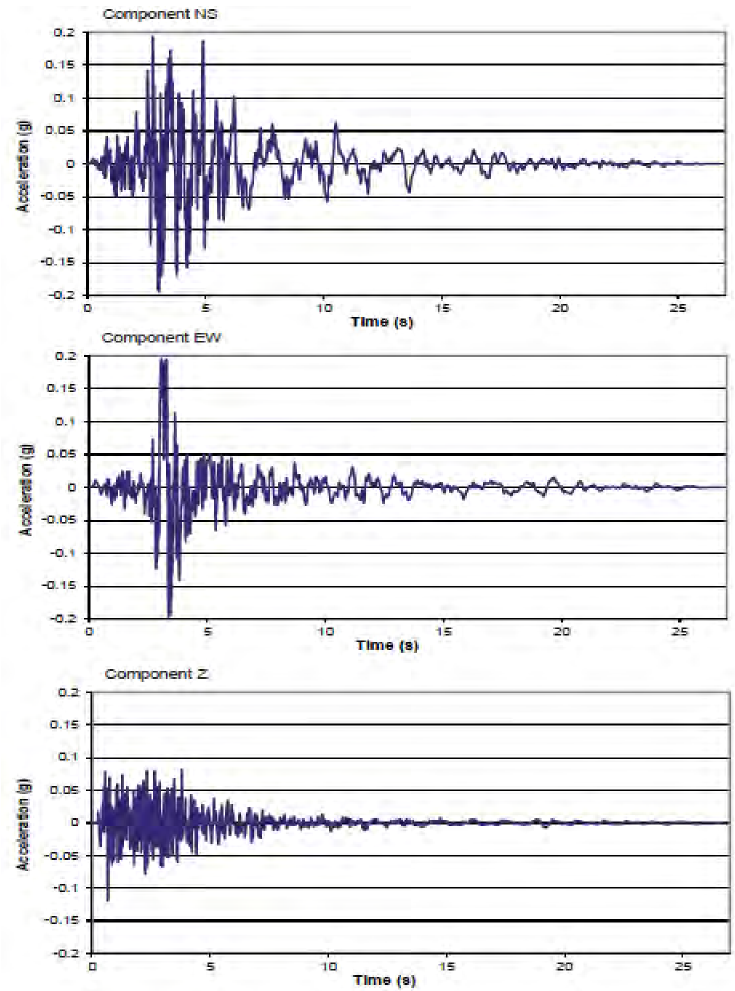
OBE	PGA (horizontal)	0.09 g
	PGA (vertical)	0.054 g
SSE	PGA (horizontal)	0.195 g
	PGA (vertical)	0.117g

Magnitude & Peak Ground Accelerations (32m BGL)

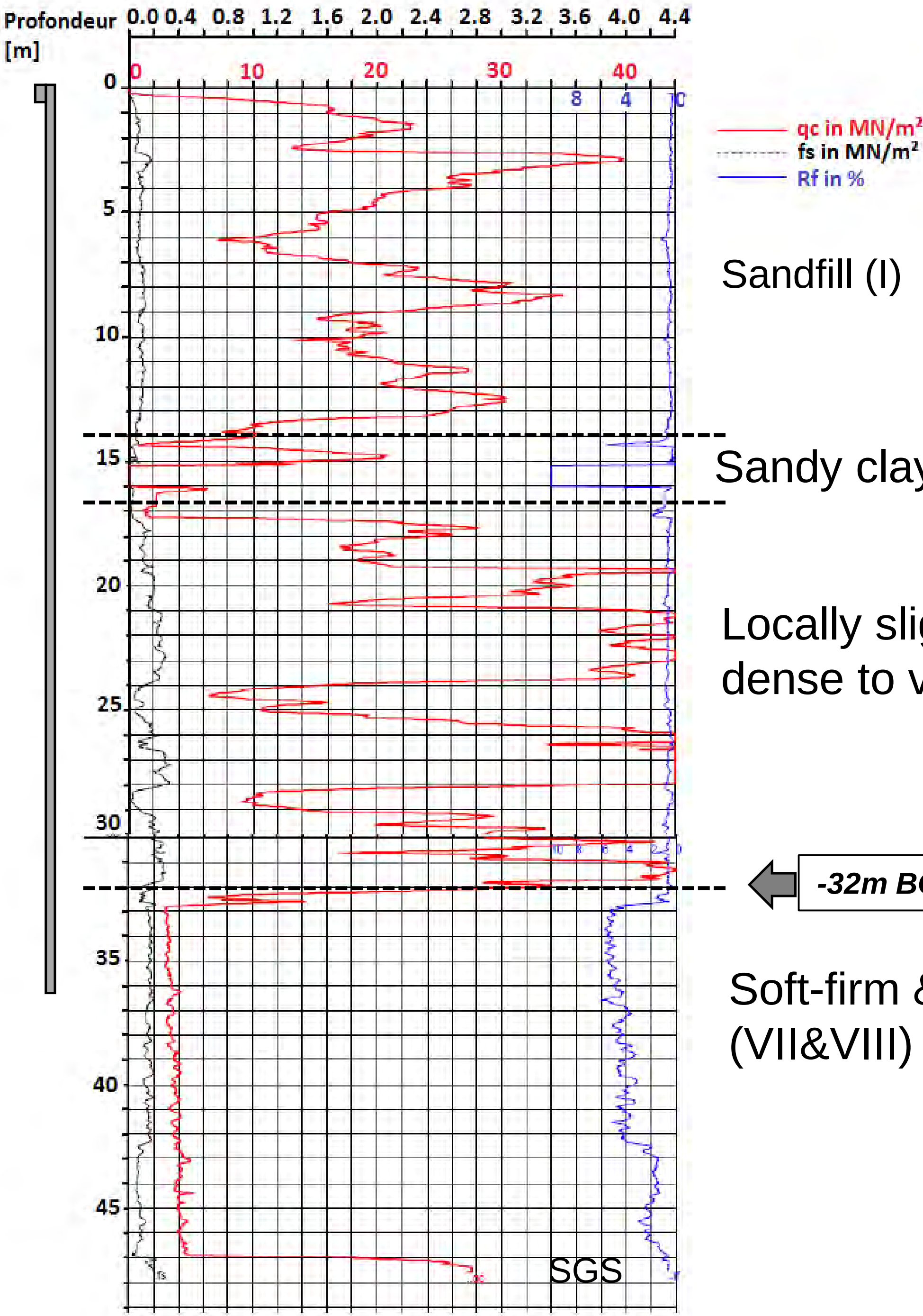
Definition of time histories (accelerograms)

Target response spectra
+
Selection of natural 3-components accelerograms
+
Spectral matching





GEOTER 2014



Water table at ~4m depth
(2 different aquifers)

Sandfill (I)

Sandy clay + peat (II)

Locally slightly clayey
dense to very dense sand (III->VI)

-32m BGL – Definition of Elastic Response Spectra

Soft-firm & sandy tertiary clay
(VII&VIII)

Key Soil parameters (preliminary design)

Stratum	Lithology	Unit weight γ_{sat} [kN/m³]	Friction Angle ϕ' [°]	cohesion c' [kPa]	Undrained shear strength s_u [kPa]	At rest Coefficient K_0 [-]	Poisson Ratio ν [-]	Max. (dyn) shear modulus G0 [MPa]	Fines Content (<60µm) [%]	Clay Content (<2µm) [%]
I	SANDFILL	20.0	35	0	-	0.43	0.3	95	1-3	-
II	SANDY CLAY+PEAT	19.0	15	15	-	0.74	0.1	130	4-27	0-7
III -> VI	SAND	19–21	25-37	0-10	-	0.40-0.58	0.3	155-240	0-8	-
VII ->VIII	TERTIARY CLAY	20-21	15-25	24-33	150	0.58-1.00	0.45-0.49	175-180	34-100	3-50
IX	SANDSTONE	22.0	25	24	-	0.58	0.3	250	12-76	2-8.5

Static analyses *Dynamic analyses* *Liquefaction assessment*

Site Investigation data includes:

- Field tests: CPT, BH, PMT
- Laboratory tests: identification, triaxial tests (incl.bender elements), oedometer tests
- Geophysical testing: Horizontal Cross-hole seismic survey
- Ground water monitoring

Assessment of Liquefaction Potential

- For granular soils (fines content < 35%, clay content < 20%)

- CRR = Cyclic Resistance Ratio: derived from CPT
- CSR = Cyclic Stress Ratio: derived from seismic loading specifications

$$CSR = \frac{(\tau_h)_{av}}{\sigma'_{v0}} = 0.65 \frac{a_{max}}{g} . \frac{\sigma_{v0}}{\sigma'_{v0}} . r_d$$

$$CRR = 0.833 \left(\frac{(q_{c1N})_{cs}}{1000} \right) + 0.05 \quad \text{if } (q_{c1N})_{cs} < 50$$

$$CRR = 93 \left(\frac{(q_{c1N})_{cs}}{1000} \right)^3 + 0.08 \quad \text{if } 50 \leq (q_{c1N})_{cs} < 160$$

$$MSF = \frac{CRR_M}{CRR_{M=7.5}} = 10^{2.24} / M_w^{2.56}$$

$(\tau_h)_{av}$ = average induced cyclic shear stress

σ_{v0} = total in-situ vertical stress relative to ground surface

σ'_{v0} = effective in-situ vertical stress

a_{max} = peak horizontal ground acceleration

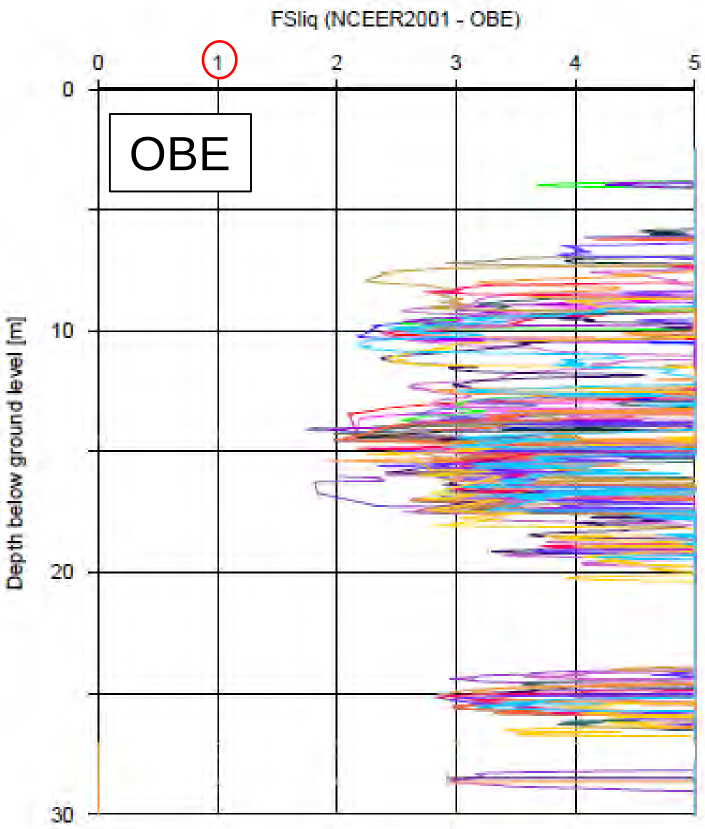
g = acceleration due to gravity

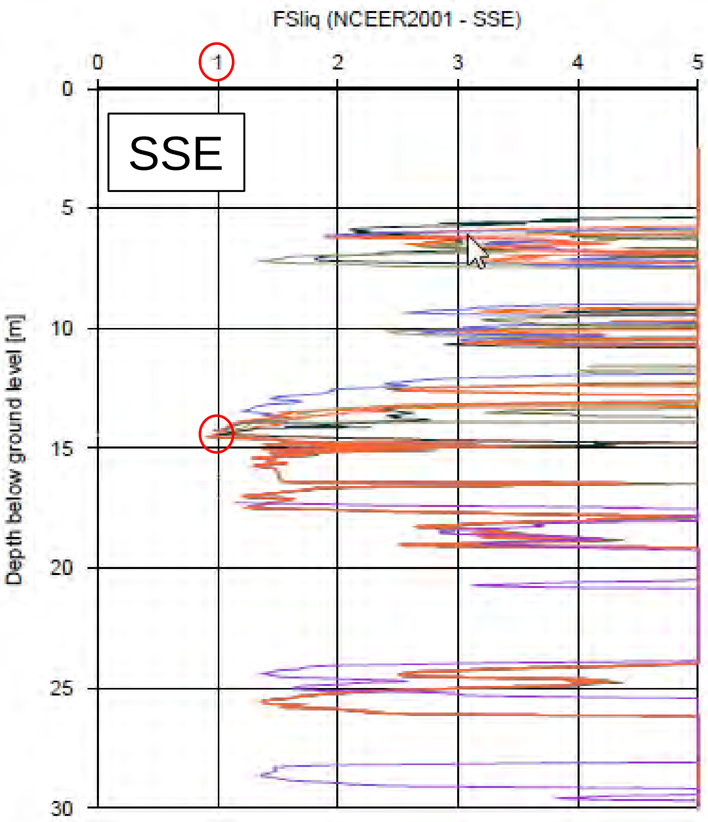
r_d = stress reduction coefficient to allow for deformability of the soil column

$(q_{c1N})_{cs}$ is the Clean Sand Equivalent Normalised Cone Resistance.

[Youd et al.2001]

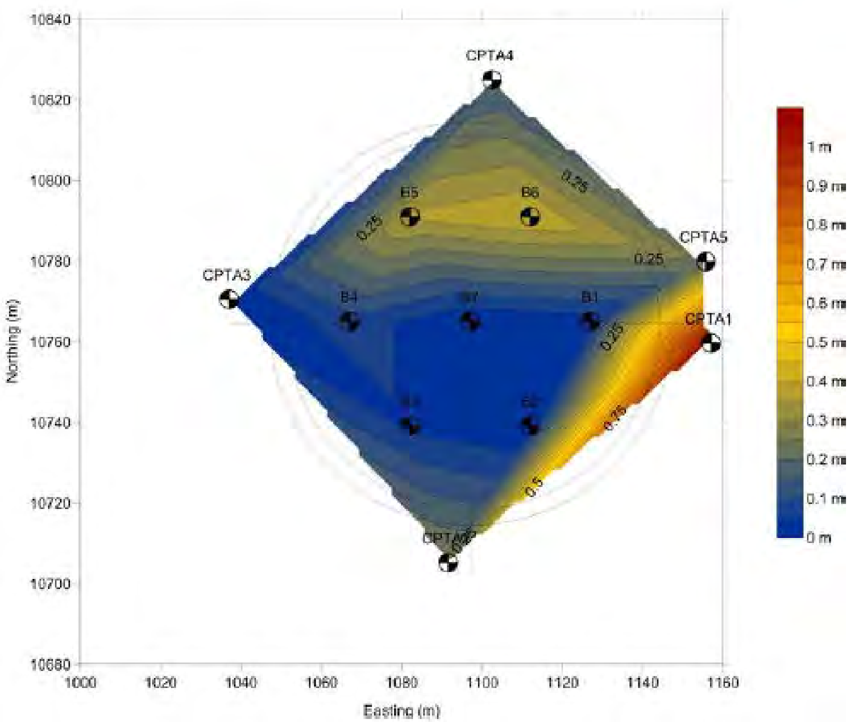
⇨ Safety Factor against liquefaction FS_{liq} = CRR/CSR



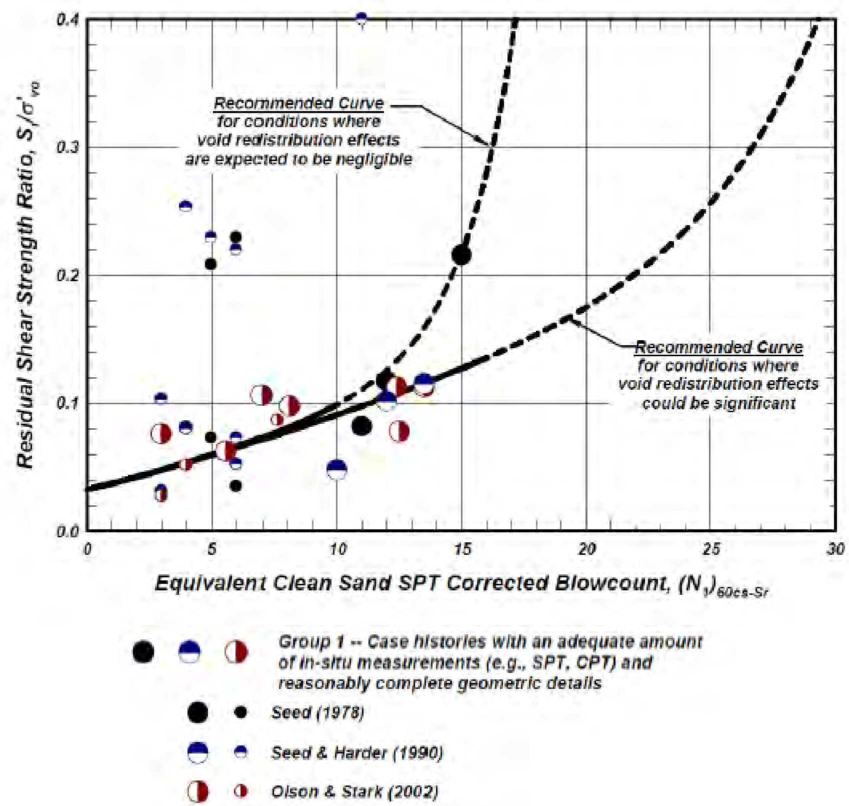


Safety factors close to 1.0 for SSE at 15m depth (« sandy clay »)
in preliminary design

Assessment of Liquefaction Potential & Residual Shear Strength



Thickness of soil layers considered close or below the SF against liquefaction



Idriss & Boulanger 2007

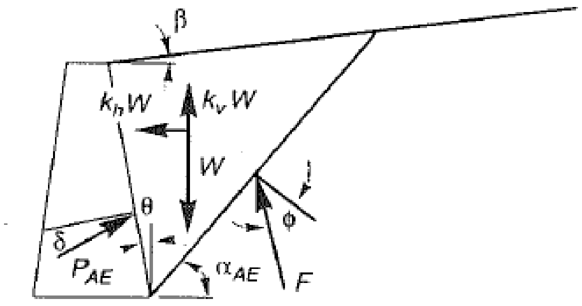
Seismic Design Approach

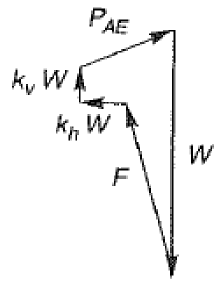
- Safety Approach:
  - OBE seismic design : Ultimate Limit State (ULS)
    - Approach DA2 of Eurocode 7
    - Calculations performed with characteristic values
    - The variable loads are multiplied by γ_Q/γ_G = 1.5/1.35 = 1.11
    - Calculation results (M,V,N) are multiplied by γ_G = 1.35 (Bauduin et al. 2003)
  - SSE seismic design : Accidental Limit State (ALS)
    - A load factor = 1 is applied to both the permanent and transient loads

- Calculation methods:
  - Pseudo-static loading approach
  - *Frequency approach*
  - Full dynamic (time history) loading approach

Dynamic wall pressures & permanent wall deflections :

- Influenced by the dynamic response of the wall and backfill
- Can increase significantly near the natural frequency of the wall-backfill system

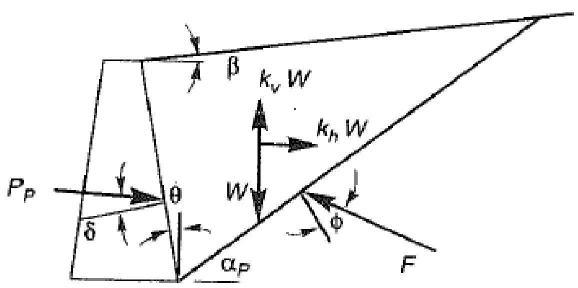


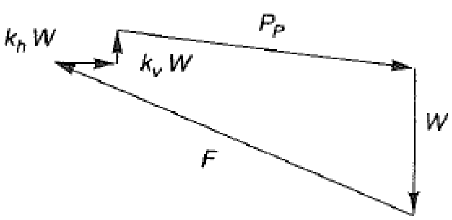


Mononobe-Okabe analysis

Forces acting on active wedge

$$P_{AE} = 0.5.\gamma.H^2.(1\text{-}k_V).K_{AE}$$





Forces acting on passive wedge

! Assumption: retained soil mass behaves as a rigid body

Seismic Design – Finite Element Modeling

- **Methodology:**

- Step 1: Selection of Soil Models
- Step 2: Static Design
- Step 3: 2D vs 3D models
- Step 4: Seismic Pseudo-Static Calculations
- Step 5: Seismic Time Domain Calculations

FEM - Selection of Soil Models

- Use of Simplified Linear Elastic Soil Model (eg. Mohr-Coulomb in PLAXIS)

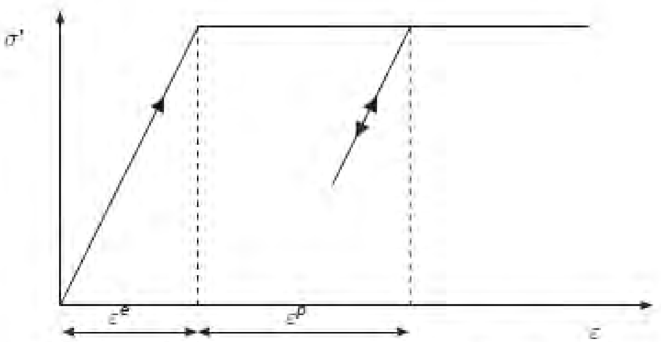
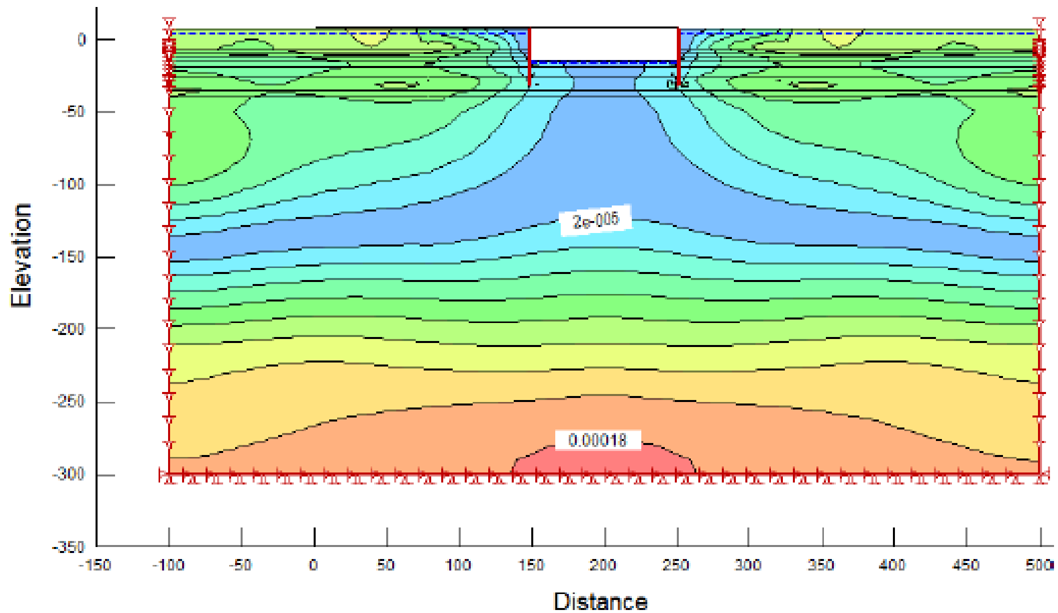


Figure 3.1 Basic idea of an elastic perfectly plastic model

E	:	Young's modulus
ν	:	Poisson's ratio
c	:	Cohesion
φ	:	Friction angle
ψ	:	Dilatancy angle
σ_t	:	Tension cut-off and tensile strength

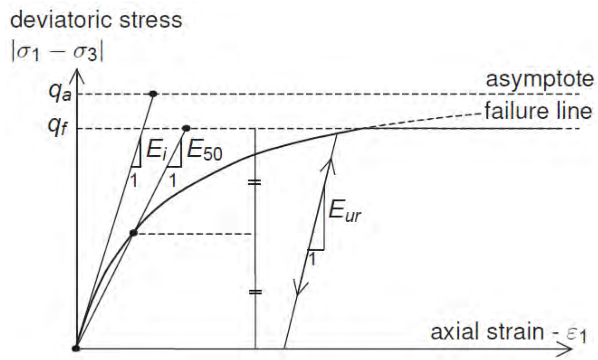


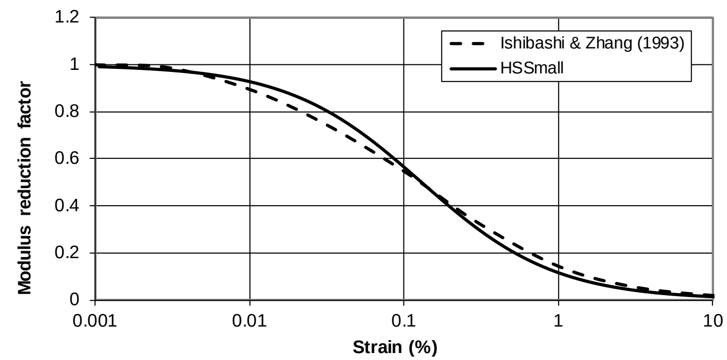
Preliminary Quake/W model to assess soil deformations under seismic loads

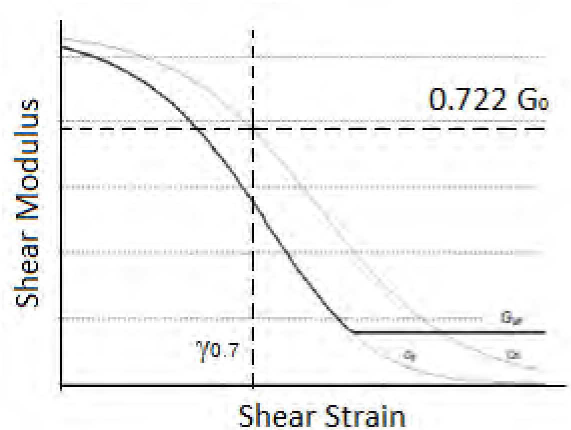
Soil parameters needs to be chosen taking into account the actual range of deformations

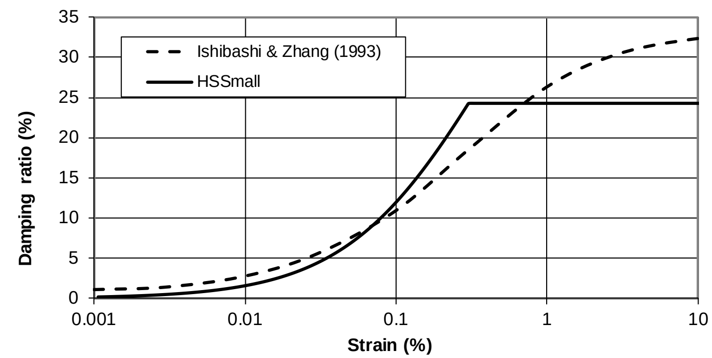
FEM - Selection of Soil Models

- Use of more “advanced” Soil Model (eg. Hardening Soil Small Strain in PLAXIS)







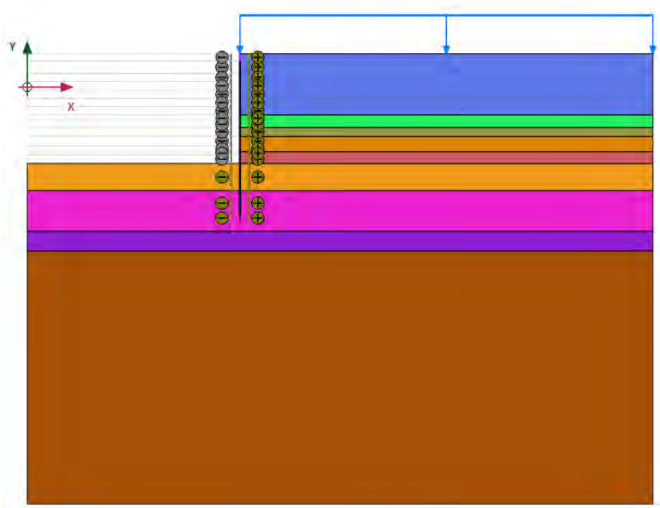


Calibration of HSS soil parameters

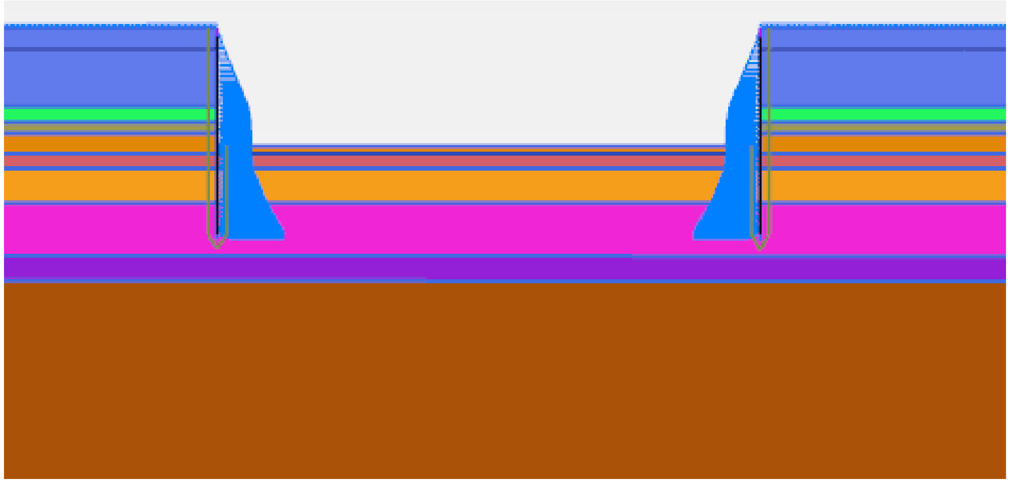
Soil parameters adjusted based on degradation curves

FEM – Static Design & 2D/3D equivalence

2D Axisymmetric Model



2D Plane Strain Model



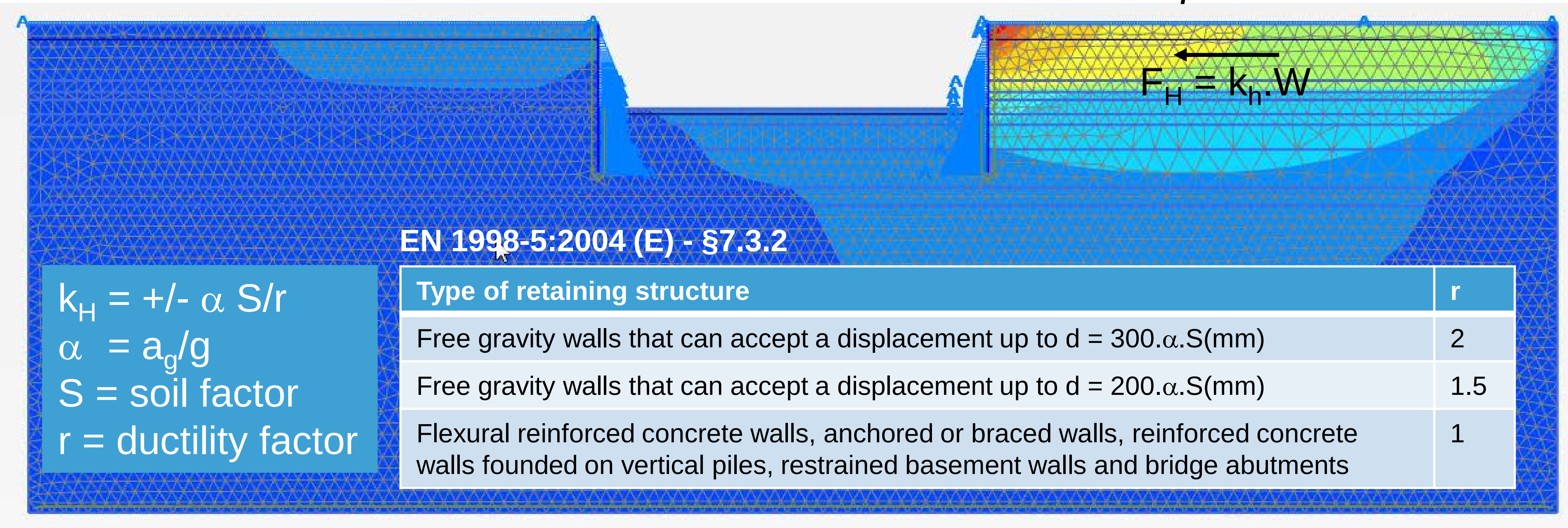
Ground loads taken by hoop forces

Earth pressures applied as external (stabilising) loads

- Bending Moment profiles
- Shear Forces
- Hoop Forces
- Convergence displacements

FEM – Pseudo-static analyses

Max. displacement = 2m



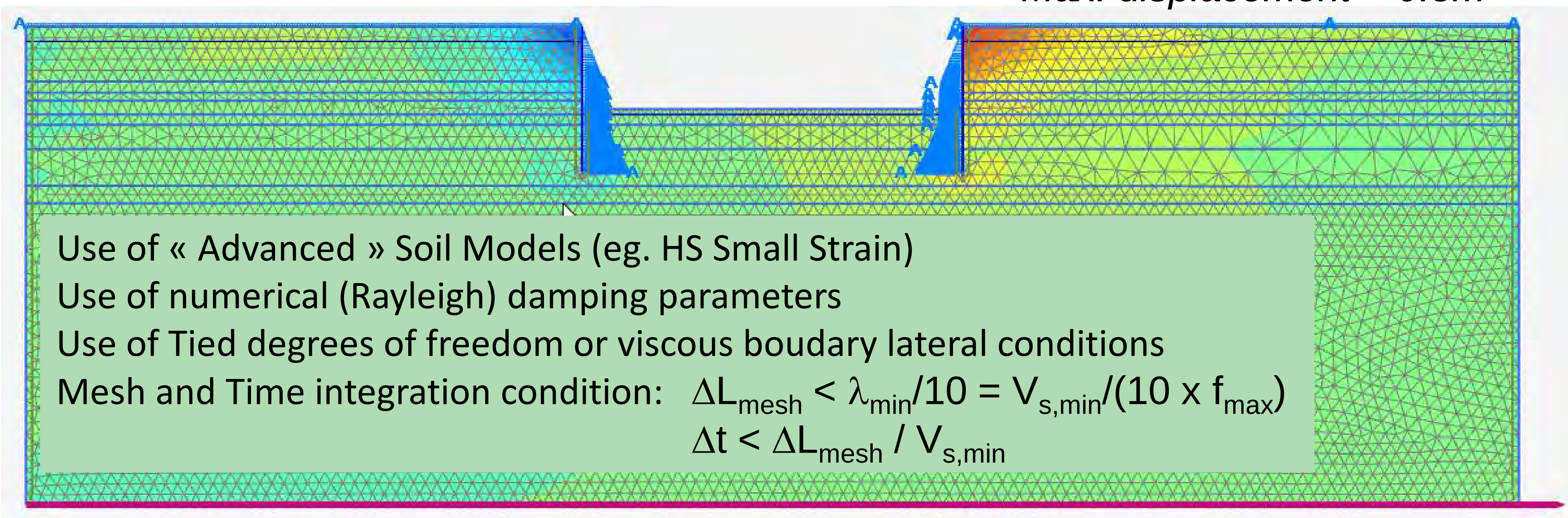
k_H = +/- α S/r
α = a_g/g
S = soil factor
r = ductility factor

EN 1998-5:2004 (E) - §7.3.2

Type of retaining structure	r
Free gravity walls that can accept a displacement up to d = 300.α.S(mm)	2
Free gravity walls that can accept a displacement up to d = 200.α.S(mm)	1.5
Flexural reinforced concrete walls, anchored or braced walls, reinforced concrete walls founded on vertical piles, restrained basement walls and bridge abutments	1

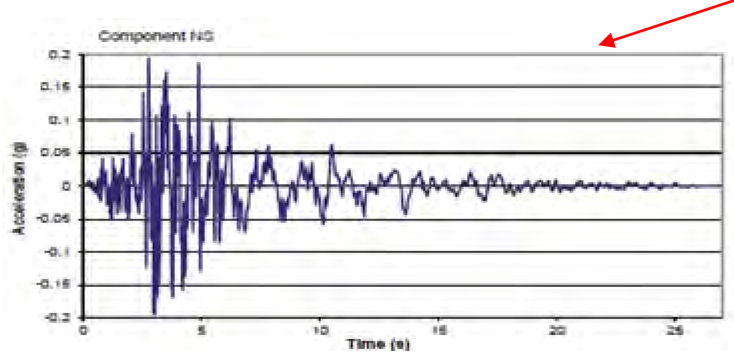
No specific requirements regarding the:
- meshing
- time integration
- Boundary conditions

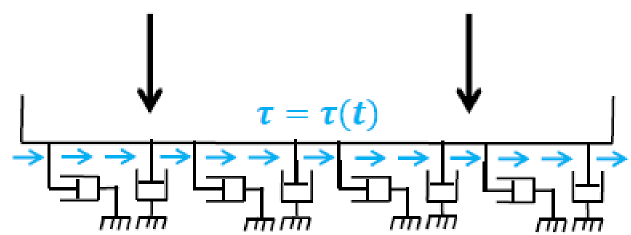
Max. displacement < 0.5m



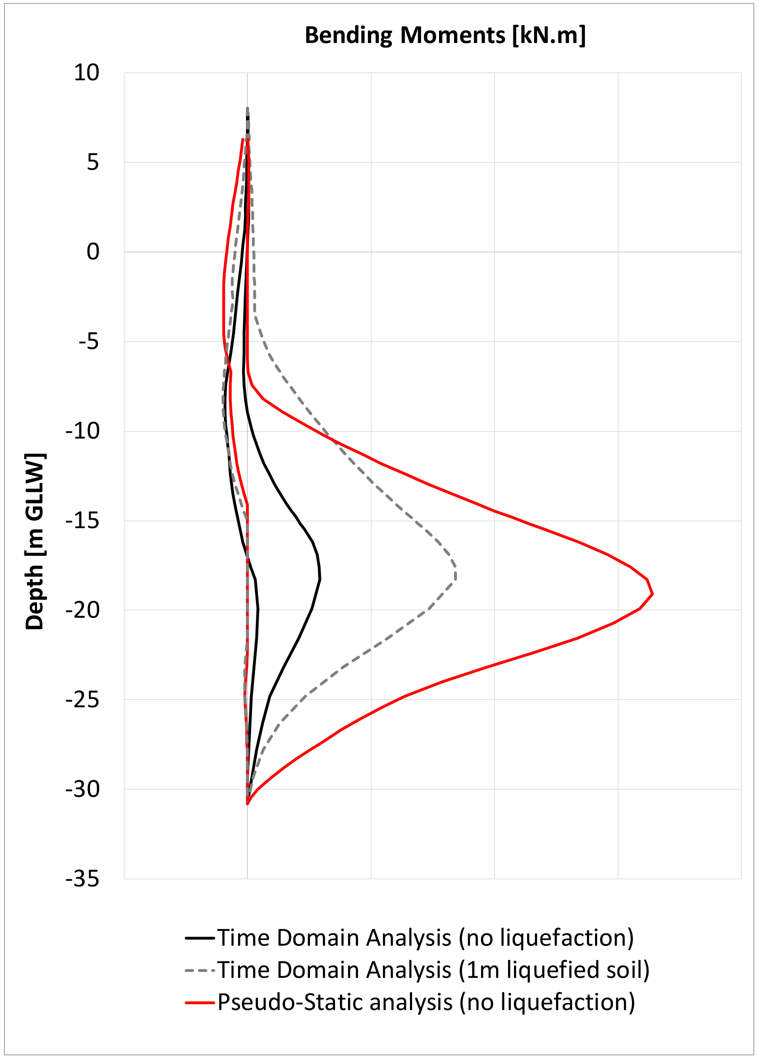
Use of « Advanced » Soil Models (eg. HS Small Strain)
Use of numerical (Rayleigh) damping parameters
Use of Tied degrees of freedom or viscous boudary lateral conditions
Mesh and Time integration condition: $\Delta L_{mesh} < \lambda_{min}/10 = V_{s,min}/(10 \times f_{max})$
$\Delta t < \Delta L_{mesh} / V_{s,min}$

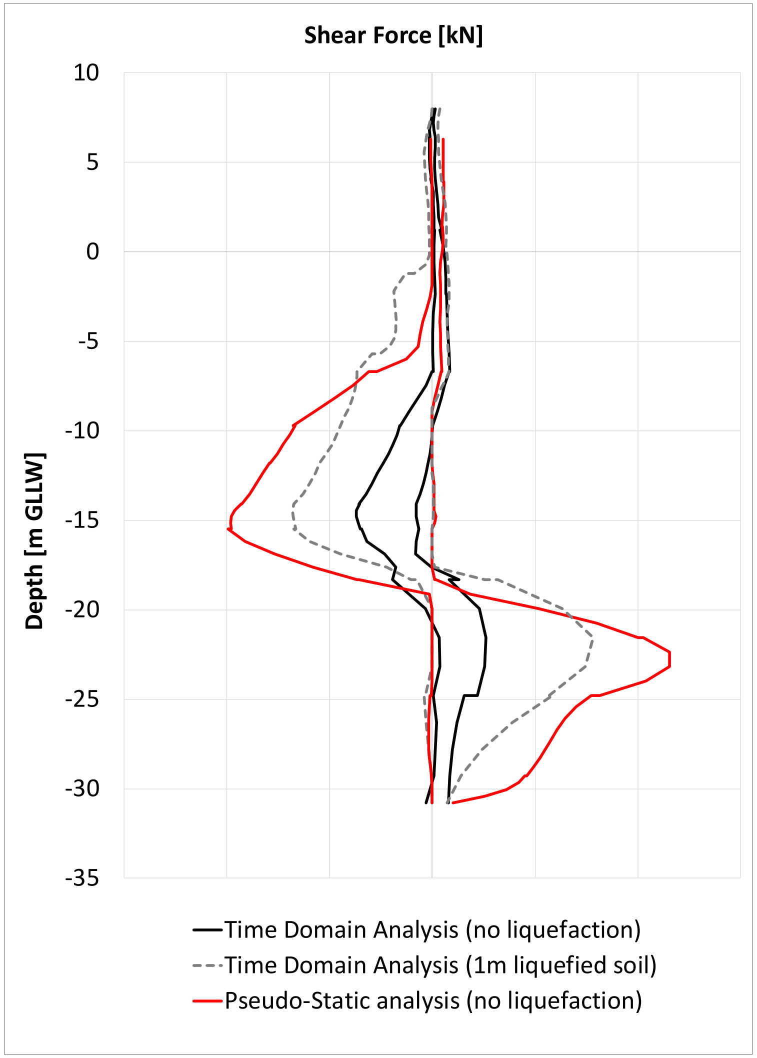
Seismic input (after deconvolution) + Compliant base motion





FEM – Example of Comparative Results





Conclusions

- Importance of the depth / definition of the target response spectra
- Importance of considering potential liquefaction assessment early in the design process (soil investigation !)
- Simplifying the problem to 2D equivalent analyses allows saving valuable time, especially for preliminary design
- Pseudo-static analyses may be very overconservative as compared to time-domain analyses

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Thank you