Seismic design of sheet piles

Economic benefits of advanced design methods

Water Transport Solutions

Foreword

ArcelorMittal is the worldwide leader in sheet piling technology, and always ahead in offering most innovative foundation solutions. Our products are extensively used worldwide for the construction of quay walls, waterways, flood protection barriers, mobility infrastructure projects and containment structures.

Our values are sustainability, reliability and quality assurance, leading to highest levels of stakeholder value creation and customer satisfaction.

We offer complete package solutions, based on our comprehensive and wide range of products and services, expert technical support from the early design stages of a project to its completion, customized fabrication, just-in-time delivery and after-sales services.

ArcelorMittal, as the global leading steel producer, aims at reaching carbon neutrality by 2050 and steel sheet piles are a major contributor to the circular economy concept of "reduce-reuse-recycle".

Our innovative solutions allow to design optimised and efficient infrastructures, using our HZ®-M combined wall system, the unique AZ®-800 sheet pile range, high strength and low corrosion AMLoCor® steel grades.

Launched in 2021, our EcoSheetPile™Plus range is produced from 100% recycled and reusable steel, and with 100% renewable electricity. These steel sheet piles are produced under our ArcelorMittal's XCarb™ recycled and renewably produced label, audited and certified by an independent third-party. Based on a Life Cycle Assessment (LCA), our sheet piles are covered by an Environmental Product Declaration (EPD) which allows for an accurate assessment of their environmental impact.

To further enhance design and project efficiencies of sheet piling solutions in areas of high seismic risks, this document presents innovative design methods for extreme dynamic loading conditions in ports and waterways, and other infrastructure domains.

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1. Introduction

Steel sheet piles are widely used for the construction of a variety of structures: quay walls and breakwaters in harbours, bank reinforcements on rivers and canals, urban infrastructures such as underpasses, as well as global hazard protection schemes. In each of these applications, sheet piles have proven their ability to effectively withstand the consequences of earthquakes in seismic areas.

Commonly used seismic design methods are still considered unsatisfactory in many cases, especially for the steel-based quay wall structures where the application of these design approaches hampers a substantial potential for cost optimisation.

SENER, an international maritime engineering group based in Spain, carried out a study to highlight the main features of advanced design of sheet pile walls in high seismic areas.

This study uses the dynamic design method based on Finite Element Modelling (FEM) and considering real acceleration-time history as seismic input.

The first part of the study uses a reference case to highlight the different aspects to be considered in the dynamic design using FEM, it sheds light on the hydrodynamic loads and their impact on the design, using Computational Fluid Dynamics (CFD) models.

The second part of the study compares the dynamic design method to the traditional pseudo-static method that uses the Mononobe-Okabe formula. The comparison is carried out through a parametric study treating eleven cases. The results are presented, commented and analyzed. Conclusions are finally drawn on the best practices in terms of seismic design of sheet piles.

Port of Mejillones, Chile © Puerto Angamos

2. Dynamic design

The most suitable design approach for sheet pile walls in seismic areas is using dynamic calculations in FEM. This type of calculations provides precise information about the internal forces, the deformations, the increase of pore water pressures and the expected mode of failure to be prevented.

It also permits a correct consideration of other features like the hydrodynamic loading through added masses.

Advanced dynamic design can allow up to 50 % cost savings compared to traditional pseudo-static design approaches (see Chapter 3).

2.1. Reference case

The reference case considered to showcase the design method presents the following characteristics:

- Surface level at +3.0 m;
- Seabed level at −12.5 m;
- Water level at −1.0 m;
- Type of soil: medium dense sand;
- Characteristic live load on top of the surface: 20 kN/m^2;
- Peak Ground Acceleration (PGA): 0.40 g;

- ArcelorMittal's sheet pile system:
  - Front main sheet pile wall anchored to a passive sheet pile wall with conventional tie rods;
  - Distance between the main wall and the anchor wall: 40.0 m;
  - Main wall's sheet pile section: AZ 46-700N in steel grade S 460 AP;
  - Anchor wall's sheet pile section: AZ 12-770 in steel grade S 430 GP;
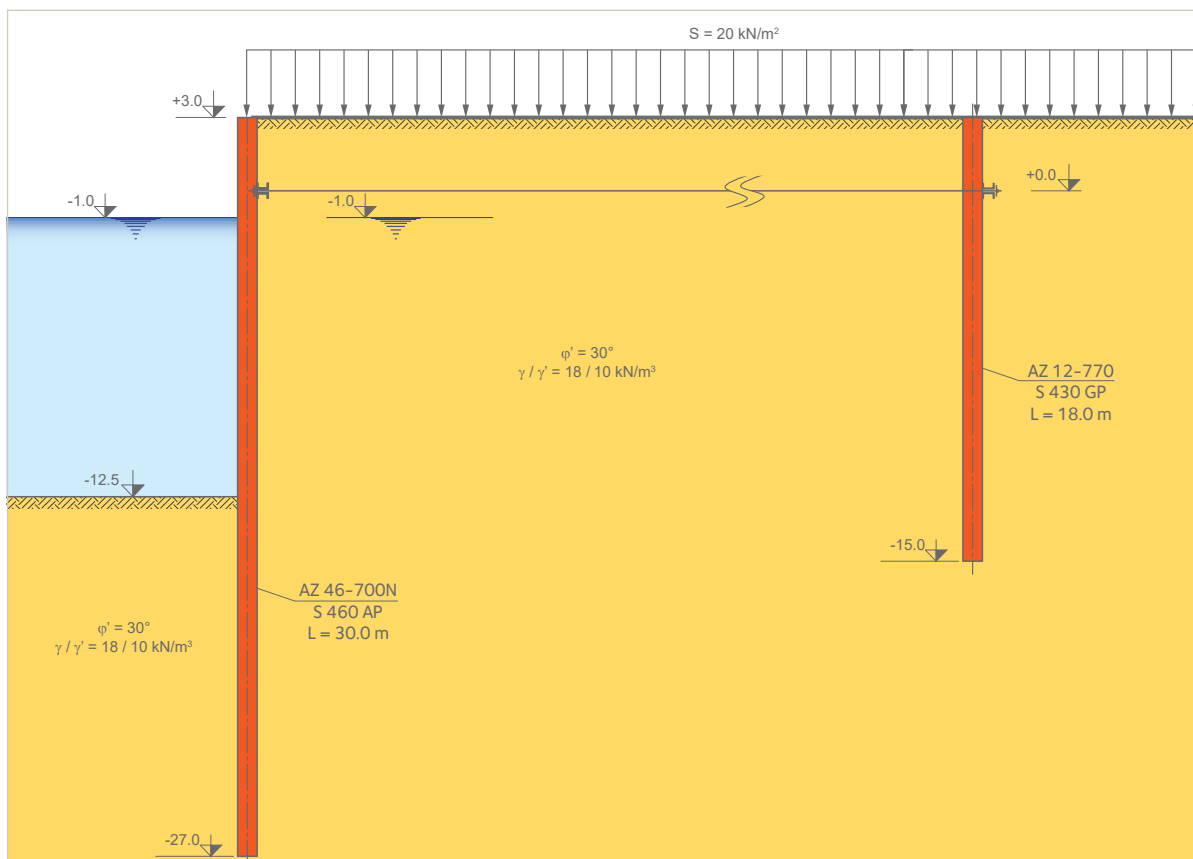 - Tie rods M64/56 every 1.4 m.

Figure 1. Cross section of the reference case.

2.2. Software

The study uses PLAXIS™ 2D FEM software for assessing the sheet piles' seismic performance. This software is widely used for solving geotechnical engineering issues.

2.3. Model geometry and boundary conditions

A general recommendation for the model size is to have a width five times greater than the height, thus ensuring that the boundaries are far enough apart and have minimum influence on the structure.

It is recommended to use a mirrored geometry in the model, this allows to examine the effect of inherent asymmetry of the accelerogram in one calculation, and to establish the "Tied degrees of freedom" boundary conditions for the lateral boundaries. These have been proven to yield a reliable response for the free-field.

A second way of examining the effect of inherent asymmetry is by considering two different directions of the seismic signal application (from left-to-right and from right-to-left). The numerical simulation is done in the same model (in two different calculation phases) and using the "Free field" boundary conditions for the lateral boundaries.

The boundary conditions used for the reference case are:
- "Free-field" for the lateral boundaries;
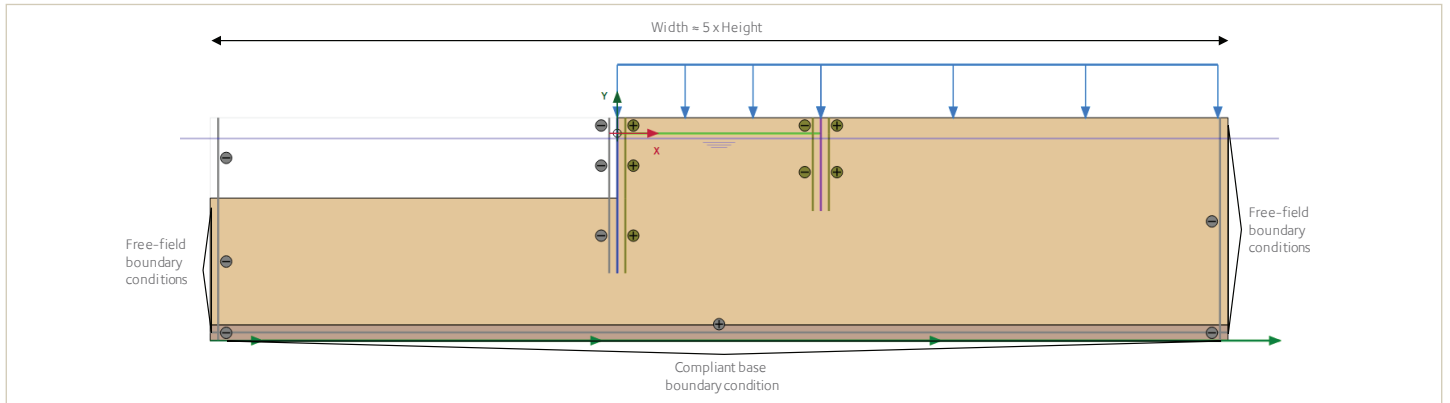- "Compliant base" for the bottom boundary.

Figure 2. Model geometry and boundary conditions.

2.4. Mesh elements

The mesh elements of the FEM model should respect two conditions:

- **Condition 1:**
  Mesh elements should be small relative to the wave length.

  Lowest expected wave length: $\lambda_{min} = \dfrac{V_{min}}{f_{max}}$

  Where:

  f_{max} Max. dominant frequency of the seismic signal
  V_{min} Min. shear wave velocity

  Element size : $L < \dfrac{\lambda_{min}}{10}$ **(6-noded elements)**

  $L < \dfrac{\lambda_{min}}{5}$ **(15-noded elements)**

- **Condition 2:**
  The fastest shear wave should not travel more than one element during one sub-step.

  Distance travelled:

  $d = V_{max} * \Delta t_{sub\text{-}step} = V_{max} * \dfrac{Dynamic\ Time\ Interval}{N_{steps} * N_{sub\text{-}steps}}$

  Where:

  V_{max} Max. shear wave velocity
  N_{steps} Number of steps
  $N_{sub\text{-}steps}$ Number of sub-steps
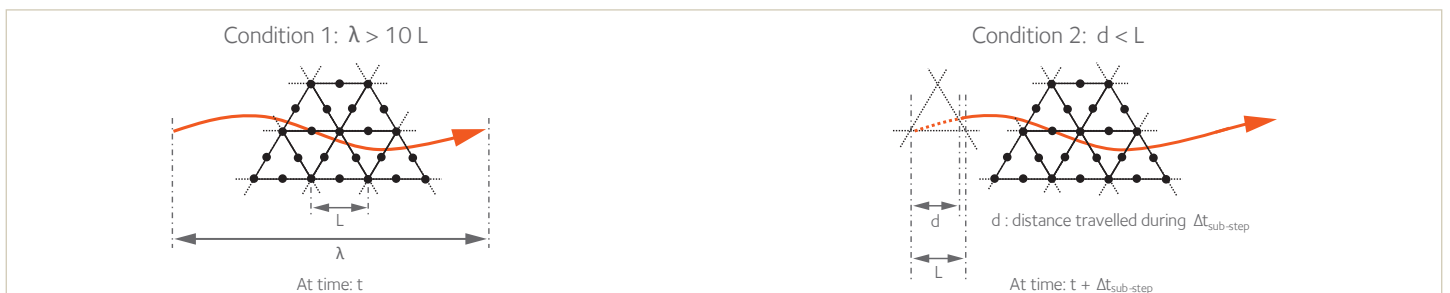
  Element size: $L > d$

Figure 3. Conditions on mesh elements size (6-noded).

2.5. Soil constitutive model

There are several soil constitutive models in the literature that can be used in a dynamic FEM calculation. The models that best describe the soil behaviour usually require complex numerical parameters that are not always available to the designer. The Hardening Soil Small Strain (HSSmall) constitutive model is however a good compromise between the complexity of parameters and the accuracy of results.

The HSSmall constitutive model presents the following features:

- Densification;
- Stress–dependent stiffness;
- Soil–Stress history;
- Plastic yielding;
- Dilatancy;
- Strong stiffness variation in the domain of small strains;
- Hysteretic, nonlinear elastic stress-strain relationship (applicable in the range of small strains).

Compared to the traditional Hardening Soil model, the HSSmall model requires two additional parameters: G_{max} and $\gamma_{0.7}$ (dynamic soil parameters).

The study uses the HSSmall constitutive model with undrained conditions (Undrained A) in order to characterize the dynamic properties of the soil.

2.6. Seismic motion

The seismic motion in the dynamic analysis is introduced by means of a prescribed displacement, with a reference initial value combined with an acceleration-time history defined as a dynamic multiplier.

Three seismic signals were considered for the dynamic design:

- The LGPC signal from 1989 Loma Prieta earthquake (USA);
- The L'Aquila – V. Aterno – F. Aterno signal from 2009 L'Aquila earthquake (Italy);
- The Jensen Filter Plant Generator Building signal from the 1994 Northridge earthquake (USA).

The three signals are further named: "LGPC", "L'Aquila" and "Jensen" respectively.

The signals originate from outcrop motions, that have been fitted to a Type 1 spectrum of Soil Type B according to EN 1998-5 [4] (see Figure 4) with a peak ground acceleration equal to 0.40 g.

Furthermore, the FE model considers only the upward waves at its base, corresponding to 50 % of the intensity of the outcrop signal (which contains both the upward and the downward waves). That is why a factor of 0.5 is applied to the signals (in Plaxis, $u_{x,start,ref}$ is set to 0.5).
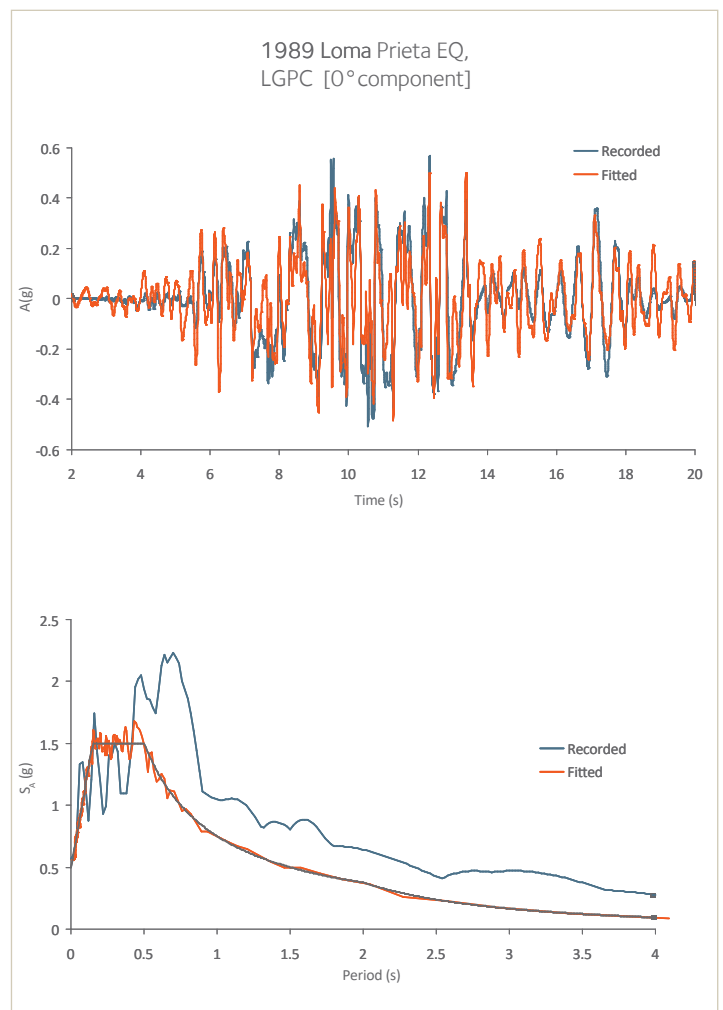

Figure 4. The recorded (blue) and fitted (orange) Los Gatos LGPC ground motions with their spectra plotted in the same scale (bottom).

2.7. Hydrodynamic loads

2.7.1. Westergaard formula

The hydrodynamic pressure is introduced in the design using the Westergaard formula. However, the hydrodynamic pressure is further studied in the next 2 sections, where assumptions on the Westergaard load are amended and justified. Nevertheless, the initial hydrodynamic pressure considered the expression proposed in EN 1998-5 §E.8 :

$$ q(z) \ = \ \frac{7}{8} k_h \cdot \gamma_w \cdot \sqrt{h \cdot z} $$

Where:

$q(z)$ is the hydrodynamic pressure in kN/m^2 ;

γ_w is the specific weight of water in kN/m^3 ;

h is the free water height in m ;

z is the vertical downward coordinate, in m, with the origin at the water surface ;

k_h is the horizontal seismic coefficient, determined according to:

$$ k_h \ = \ \alpha \frac{S}{r} $$

With:

α is the ratio of the design ground acceleration on type A ground, a_g , to the acceleration of gravity g ;

S is the soil amplification factor according to EN 1998-1 [3];

r is equal to 1 for the hydrodynamic pressures as per EN 1998-5 §E.8.

In this case, $\alpha \cdot S$ is obtained directly from the Plaxis dynamic model, as the propagation of the seismic motion is performed.

Finally, the Westergaard formula is approximated in Plaxis by means of three linear loads.

2.7.2. CFD Model

Computational Fluid Dynamics (CFD) models were used to assess the impact of the hydrodynamic pressure on the seismic design of sheet pile walls. For that purpose, CD-adapco® STAR-CCM+® software performs a CFD model aiming to investigate the hydrodynamic pressures from the seawater on the front sheet pile wall during a seismic event. The CFD model is based on the following assumptions:

- 2D model;
- Uncoupled fluid-soil problem and hence a direct fluid-structure interaction is not investigated;
- The simulation is performed just on the front sheet pile wall;
- The sheet pile wall is modeled by a rigid plate;
- The base of the model is the seabed level (-12.5 m);
- The lateral contours are set at a specific distance from the plate to avoid alterations on the results;
- The seawater is considered at one side of the plate;
- The other side of the plate is considered to be "empty", its effect is the movement of the plate;

- The CFD input data (displacements on the front sheet pile wall with respect to time) is obtained from the Plaxis dynamic analysis without considering Westergaard effects (meaning that no water effect is considered);
- Fluid modelling: according to the literature and the current scenario characteristics, the liquid compressibility is neglected;
- The model's initial conditions correspond to still water without waves. Waves arise as a consequence of the seismic acceleration, but their effect on the pressures is negligible;
- Although it is foreseen that turbulence models do not affect the results, the model considers a classical k-epsilon turbulence model;
- The seismic effect was applied in three different ways (plate displacement, plate velocity and a source body force equal to the seismic acceleration), all three giving similar results.

Above assumptions are used to set the CFD model. Then, the hydrodynamic pressure is simulated by enforcing the plate to move towards the water. Hence, imposed deformations on the plate have to be applied. Consequently, for modelling the hydrodynamic pressure due to the seismic motion, the deformations developed on the front sheet pile wall during the earthquake need to be imposed.

The CFD model considers an 11.5 m sea depth, a moving plate on the left side, a still sea condition 250 metres away from the sheet pile wall, a solid surface at the bottom and an air ambient pressure at the top.
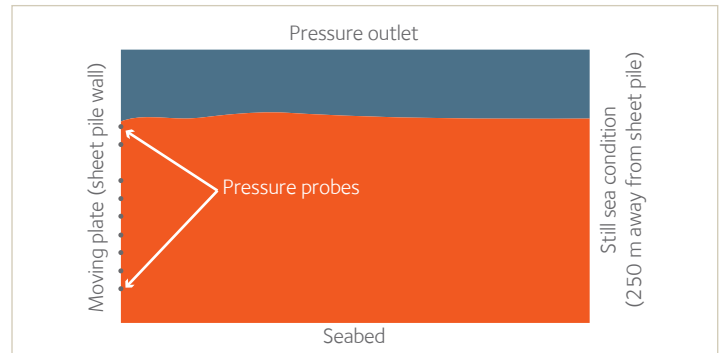

Figure 5. Simple CFD computational domain. Multiphase simulation considering water/air.

Plaxis solves the soil problem while the CFD model is expected to solve the fluid problem. Although the direct interaction could not be modelled at this stage of the study, it is proposed to use Plaxis results as input data for CFD and vice versa. In principle, the process is to be understood as follows:

1. Computing the displacements in Plaxis on the free height of the sheet pile wall during the earthquake;
2. Imposing the Plaxis displacements as input data on the plate of the CFD model;
3. Obtaining the hydrodynamic pressures in the CFD model;
4. Introducing the CFD pressures on the sheet pile wall in the Plaxis model.

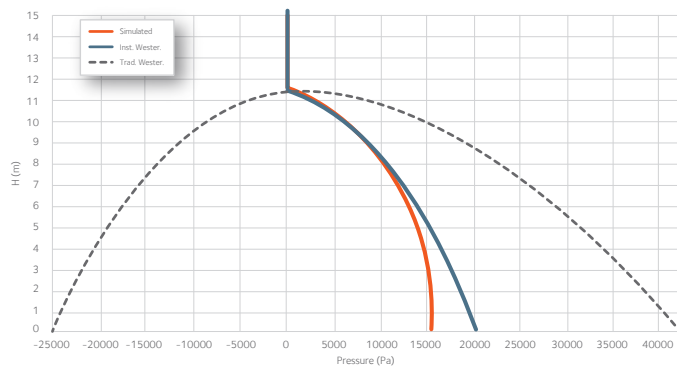The discussion on the hydrodynamic pressure modelling is carried out in Section 2.7.3.

Plaxis 2D model
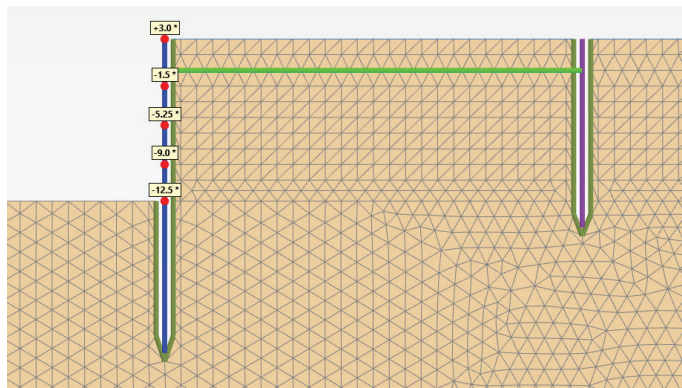
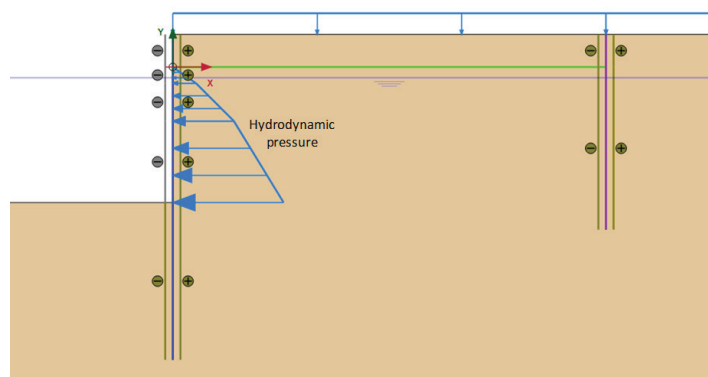1. Computing displacements at different points of the free height of the front sheet pile wall
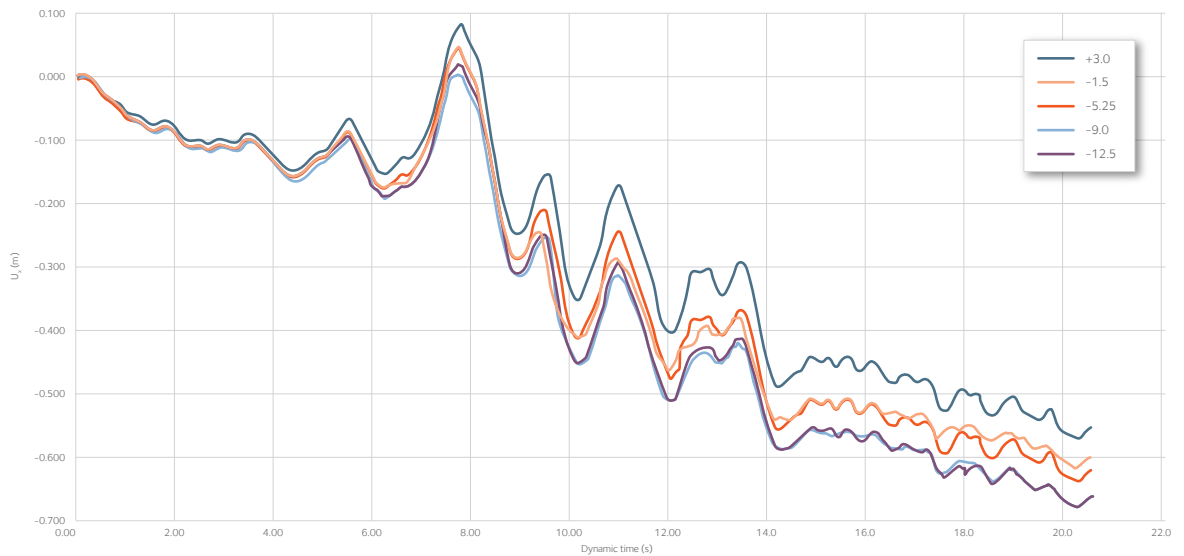
2. Displacements as input data for the CFD model

CFD model

3-4. Hydrodynamic pressures introduced in Plaxis

Plaxis 2D model

Hydrodynamic pressure

2.7.3. Hydrodynamic load assessment

The study simulates the hydrodynamic pressures from the resulting displacements of the front sheet pile wall after a dynamic calculation in Plaxis 2D. The simulation not only compares these pressures with the "Traditional Westergaard", computed with the maximum seismic acceleration, but also with the Westergaard pressure considering the instantaneous acceleration at each interval of time (referred to as the "Instantaneous Westergaard" pressure in the following). Below figures show the simulation at different time steps:

In orange: the simulated hydrodynamic pressure;

In blue: the "Instantaneous Westergaard" pressure;

In dashed line: the "Traditional Westergaard" pressure.

In light of these results, the hydrodynamic pressures obtained using the CFD model fit the "Instantaneous Westergaard" load, meaning that the maximum hydrodynamic pressure due to the earthquake is just developed for a certain time interval. Consequently, during the remaining time of the seismic motion, the hydrodynamic pressures will clearly be lower than the "Traditional Westergaard". This demonstrates that introducing the hydrodynamic pressures using the Westergaard load calculated with the Peak Ground Acceleration (PGA), as a static load in a dynamic calculation, will result in a very conservative approach.

Time step: i

Time step: i+1

Time step: i+2

Time step: i+3

Time step: i+4

Time step: i+5
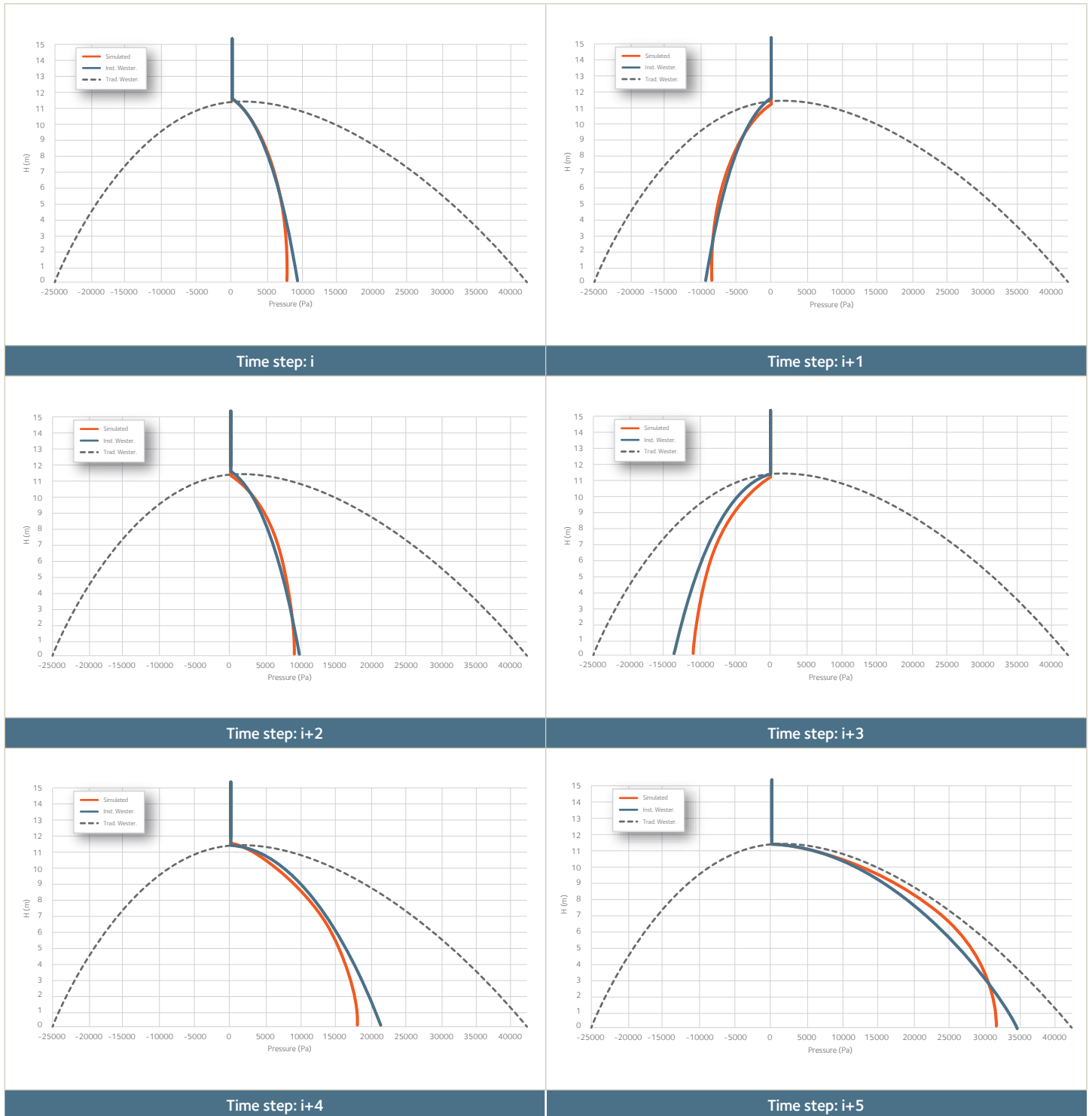
- **Using a dynamic load**

In order to consider a more realistic approach and knowing that the simulated hydrodynamic pressure is consistent with the "Instantaneous Westergaard" pressure, the Westergaard formula defined in Section 2.7.1. can be amended as follow:

$$q_i(z) = \frac{7}{8} \frac{a_i}{g} \gamma_w \sqrt{h \cdot z}$$

Where:

q_i is the "Instantaneous Westergaard" load at a time interval i (kN/m^2);

a_i is the acceleration at a time interval i (m/s^2).

The above expression uses an acceleration-time history to compute the hydrodynamic pressures at each time interval. In order to implement these considerations in Plaxis, the hydrodynamic load is applied as a dynamic load whose values change at each time interval. In other words, the load is introduced in the model with a reference value (Dynamic Linear Load) combined with a load dynamic multiplier. The latter modifies the load at each time step to get the proper value.

The reference Westergaard value (q_{ref}) is selected as the maximum load within the earthquake duration. The load multiplier is defined as the ratio (β_i) at each time step between the "Instantaneous Westergaard" load (q_i) and the reference Westergaard value:

$$\beta_i = \frac{q_i}{q_{ref}}$$

Considering the above assumptions, the hydrodynamic load can be introduced in the model as a dynamic load, fitting exactly the "Instantenous Westergaard" definition.

In order to assess the impact of the hydrodynamic loads on the design, a comparison of the bending moments is carried out for the three seismic design situations: without hydrodynamic loads, with the static "Traditional Westergaard" load and with the dynamic "Instantaneous Westergaard" load.

Figure 6 presents the results of the bending moment distribution of the front sheet pile wall.
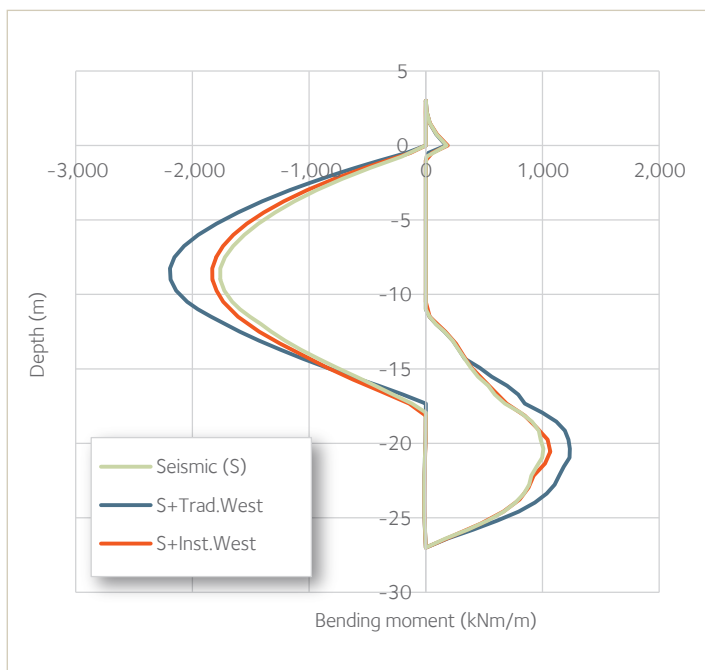

Figure 6. Sensitivity analysis on the Westergaard load consideration.

Comparing both loads, the "Traditional Westergaard" load represents an increase of 24.5 % with respect to the design force obtained for the seismic load; while the "Instantaneous Westergaard" load produces an increment of 3.9 %. These results are in line with the findings of Prof. Gazetas [9], who concluded that hydrodynamic pressures have a small contribution, about 5 %, on the bending moments at the front sheet pile wall.

Sensitivity analysis on Westegaard consideration			
	Bending moment	Absolute increment	Relative increment
	kNm/m	kNm/m	%
Seismic (S)	1761	–	–
S + Traditional Westergaard	2193	431	24.5 %
S+ Instantaneous Westergaard	1830	69	3.9 %

Table 1. Comparison on design bending moment at the front sheet pile wall between the Traditional and the Instantaneous Westergaard load application.

- **Using added masses**

The added masses method, as its name implies, consists of adding the mass which contributes to the hydrodynamic pressures to the structure under analysis.

Extrapolating the Westergaard formula, the added mass along the seawater column is:

$$m_{ai} = \frac{7}{8} \rho_w \sqrt{H \cdot z_i} A_i$$

Where:

H is the water depth in m;

z_i is the depth from the water surface in m;

A_i is the tributary surface area at point i in m^2;

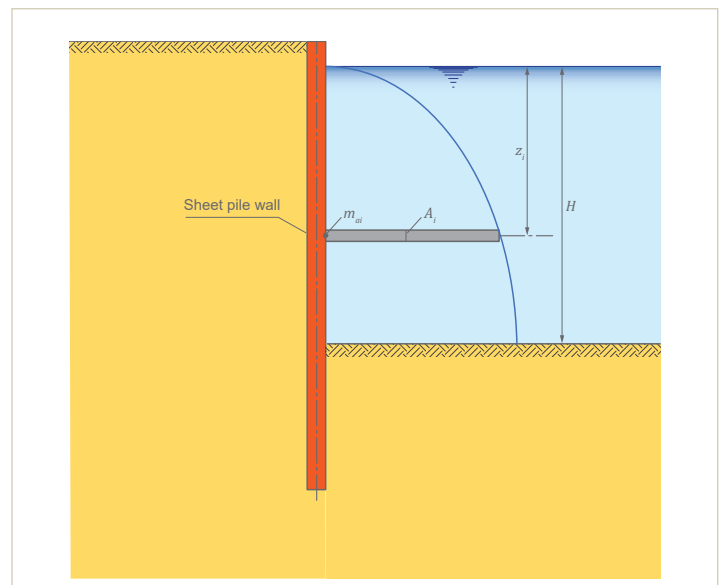ρ_w is the density of water in kg/m^3.

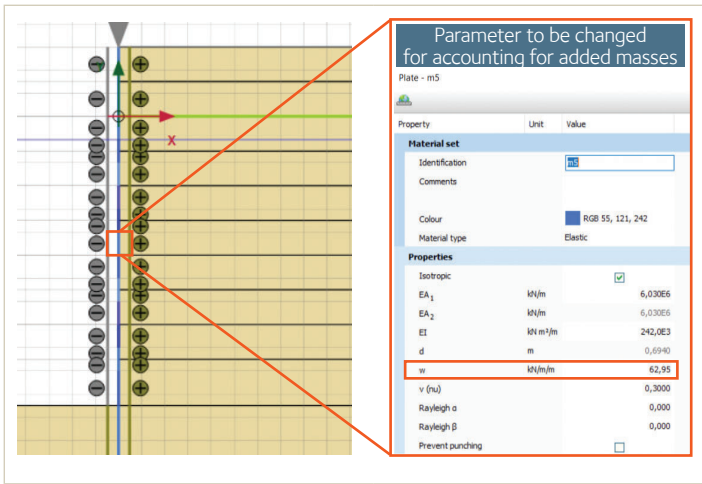Figure 7. Westergaard added-mass representation.

Figure 8. Implementation of added masses in Plaxis 2D.

In the model, it is implemented by discretising the front sheet pile wall in elements of one metre length and modifying the weight of each plate by allocating the corresponding mass at each metre of depth. Afterwards, the seismic analysis is performed under the seismic action only, without additional load on the front sheet pile wall.

2.8. Results and verifications

2.8.1. Front wall

The following design forces are obtained from the dynamic analysis performed in Plaxis 2D:

	Units	LGPC signal	L'Aquila signal	Jensen signal
Bending moment	kNm/m	1865	2055	1915
Shear force	kN/m	465	520	495
Axial force	kN/m	565	520	630

Table 2. Summary of internal forces for the front wall.

The structural resistance of the sheet pile system is verified below using the Durability software, based on EN 1993-5 [1]. They take into account the results from the dynamic analysis of LGPC, L'Aquila and Jensen seismic motions. The sheet pile section of the front wall is AZ 46-700N, as an optimised solution, in steel grade S 460 AP.

For L'Aquila motion (highest bending moment):

2.8.2. Anchor wall

The following design forces are obtained from the dynamic analysis performed in Plaxis 2D:

	Units	LGPC signal	L'Aquila signal	Jensen signal
Bending moment	kNm/m	380	420	375
Shear force	kN/m	285	350	325
Axial force	kN/m	165	200	215

Table 3. Summary of internal forces for the anchor wall.

The sheet pile section for the anchor wall is AZ 12-770 in steel grade S 430 GP.

Figure 9. Structural verifications using the Durability software based on EN 1993-5: main wall (left), anchor wall (right).

3. Comparison of design methods

In the previous chapter, special aspects of the dynamic design method were presented using a reference case.
In this chapter, the dynamic design method is compared to the pseudo-static method through a parametric study covering eleven cases. The next sections describe the main features characterizing the study.

In addition, below assumptions are made for simplification purposes:

- Geotechnical soil conditions are characterized by the Hardening Soil Small Strain constitutive model with undrained conditions;
- Sheet pile solutions are designed under a non-collapse basis;
- Liquefaction is outside the scope of this study.

3.1. Parametric study

The study covers 3 cases subdivided to 11 sub-cases.

- Case 1: dense sandy soil, low acceleration level (0.10 g) and two seabed levels;
- Case 2: dense sandy soil, two seismic action levels: medium (0.30 g) and high (0.40 g), and four seabed levels;
- Case 3: clayey silty soil, high acceleration level (0.50 g) and one seabed level.

The design features for each case are detailed below:

The main aim of the study is to evaluate the seismic design when using either the FEM dynamic method or the pseudo-static method. For this reason, the study checks the structural resistance of the front sheet pile wall. The service requirements in terms of allowable displacements are outside the scope of the study.

		Soil	Seabed level	PGA	Spectra
Case 1	Case 1.1.	Sand	–7.5 m	0.10 g	Type 2
	Case 1.2.	Sand	–9.5 m	0.10 g	Type 2
Case 2	Case 2.1.1.	Sand	–7.5 m	0.30 g	Type 1
	Case 2.1.2.	Sand	–9.5 m	0.30 g	Type 1
	Case 2.1.3.	Sand	–11.5 m	0.30 g	Type 1
	Case 2.1.4.	Sand	–13.5 m	0.30 g	Type 1
	Case 2.2.1.	Sand	–7.5 m	0.40 g	Type 1
	Case 2.2.2.	Sand	–9.5 m	0.40 g	Type 1
	Case 2.2.3.	Sand	–11.5 m	0.40 g	Type 1
	Case 2.2.4.	Sand	–13.5 m	0.40 g	Type 1
Case 3	Case 3	Clayey Silt	–9.5 m	0.50 g	Type 1

Table 4. Design features of each case.

3.2. Geotechnical profile and seismic action

The study considers two soil profiles with one soil layer along the whole depth of each profile. The first profile is characterized by a sandy soil and the second by a clayey silty soil.

Table 5 shows the geotechnical properties of the soil layers considered in this study.

The geometry of the soil profiles considers the following depth:

- Top surface: +4.0 m;
- Water level: +0.0 m;
- Seabed level: depending on the case, it varies between –7.5 m; –9.5 m; –11.5 m and –13.5 m;
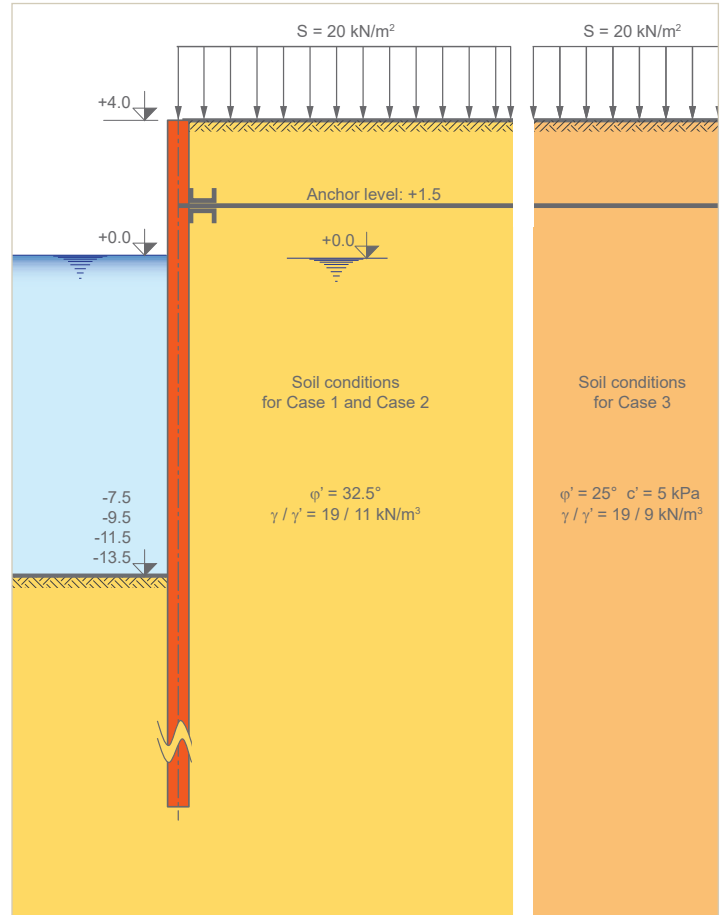- Bottom soil level: –50.0 m.

	Sand	Clayey Silt	
γ_{dry}	19	19	kN/m³
γ_{sat}	21	19	kN/m³
γ'	11	9	kN/m³
ϕ	32.5	25	°
c	0	5	kPa
Static parameters			
E_{ref}^{50}	20000	12000	kPa
E_{ref}^{oed}	16000	9600	kPa
E_{ref}^{ur}	60000	36000	kPa
m	0.5	0.5	–
Dynamic parameters			
G_{max}	93502	67500	kPa
$\gamma_{0.7}$	0.0002	0.0002	–
Modulus of subgrade reaction			
k	5000	2500	kN/m²/m

Table 5. Geotechnical properties for the soil used in the study.

Figure 9. Design cross sections.

3.3. Loads and load combinations

The analysis takes into account the self-weight of the structures, the static earth pressures, the hydrostatic pressures, the surface live loads, the seismic earth pressures and the hydrodynamic pressures due to the seismic action. For reasons of simplification, mooring, berthing, liquefaction and scouring actions are not included in the design.

The self-weight of the structure, the earth pressures and the hydrostatic pressures are directly calculated by the software, once the designer introduces the structural and the geotechnical data.

The live loads considered are equal to 20 kN/m² and the adopted PGA of the earthquake (which depends on the studied case) characterizes the seismic action. The definition of the seismic action varies from dynamic analysis to pseudo-static analysis. However, both analyses neglect the effect of the vertical seismic action as per §4.3.3.5.2 of EN 1998-1.

As presented in Section 2.6, the dynamic analysis introduces the seismic action as a base motion through acceleration-time histories. Each seismic action level considers a set of three accelerograms. All of them are already fitted to the corresponding elastic response spectrum, as previously shown for the reference case (see Figure 4). On the other hand, the pseudo-static analysis accounts for the seismic action through the Mononobe-Okabe formula, which defines the expression to compute the seismic earth coefficients, as indicated in §E.4 of EN 1998-5. In order to compute them, the study assumes:

- For active and passive earth pressures, the friction angle between the soil and the structure is ±1/3 of the friction angle of the soil;
- The definition of the horizontal seismic coefficient is:

$$k_h = \alpha \frac{S}{r}$$

Where:

α is the ratio of the design ground acceleration on type A ground, a_g, to the acceleration of gravity g;

S is the soil amplification factor according to EN 1998-1;

r is the reduction factor, equal to 2.

Besides, the seismic action produces hydrodynamic pressures. At the earth side of the front sheet pile wall, the design assumes no hydrodynamic pressure as the soil's permeability is considered to be lower than $5\cdot10^{-4}$ m/s (§7.3.2.3 in EN 1998-5). At the seaside of the sheet pile wall, the seawater develops hydrodynamic pressures. Again, in the dynamic analysis it is treated differently from the pseudo-static analysis.

Following the conclusions of Section 2.7.3, two methods are recommended to consider hydrodynamic loads for the dynamic analysis: the dynamic Westergaard load or the added masses. The latter, being more practical, was selected for this study. Therefore, added masses attached to the front sheet pile wall account for the hydrodynamic pressures in the dynamic analysis.

Alternatively, the pseudo-static analysis uses the Westergaard formula according to EN 1998-5 for introducing the hydrodynamic pressures:

$$q(z) \;=\; \frac{7}{8}\,k_h \cdot \gamma_w \cdot \sqrt{h\cdot z}$$

Where:

$q(z)$ is the hydrodynamic pressure in kN/m^2 ;

k_h is the horizontal seismic coefficient, determined according previous definition and considering reduction factor (r) equal to 1 as per §E.8 in EN 1998-5.

3.4. Design methods

3.4.1. Pseudo-static analysis

The pseudo-static design is performed using RIDO software, based on an elasto-plastic subgrade reaction modulus calculation (SGRM). The soil profile is entered into the model by means of the geotechnical parameters defined in Section 3.2 and the corresponding depths of the soil profile. The software computes the static earth pressures and the hydrostatic pressures. In addition, the sheet pile wall is represented by its flexural stiffness and top and toe levels. The Tie rods are considered as anchorage with an equivalent elastic stiffness to account for the overall system stiffness (including the stiffness and the performance of the anchorage element).

Additional considerations:

- For static earth pressure coefficients, the friction angle between the soil and the structure (δ) is set to $(+1/3)\cdot\varphi$ for the active earth pressure and $(-2/3)\cdot\varphi$ for the passive earth pressure. These values give a good calibration between the elasto-plastic subgrade reaction model and the FE model under static conditions;

- The seismic situation considers a soil-structure friction angle equal to $(\pm1/3)\cdot\varphi$ for active and passive earth pressures;

- Soil characterization is entered in the model using RIDO functions. When performing the pseudo-static calculation, the soil parameters are updated according to their corresponding seismic values;

- The surface live load is introduced in the model as a surcharge load;

- When modelling the excavation down to the seabed level, the subgrade modulus below the seabed level at the seaside of the sheet pile wall is multiplied by three in order to take into account an increase of soil stiffness under unloading behaviour;

Finally, the design uses the accidental load combination for the seismic design, which, according to §6.4.3.4 in EN 1990 [5], is:

$$\sum_{j\geq1} G_{k,j} \;+\; A_{E,d} \;+\; \sum_{i\geq1} \psi_{2,i}\, Q_{k,i}$$

Where:

$G_{k,j}$ is the characteristic value of the permanent action "j";

A_{Ed} is the design value of the seismic action;

$Q_{k,i}$ is the characteristic value of the variable action "i"; For the present analysis, the only variable action is the live load on top of the surface;

$\psi_{2,i}$ is the combination factor for quasi-permanent values of a variable action. It is defined as per BS 6349-2 [7] equal to 0.30.

- The Westergaard load is imposed in the software as a linear (static) load along the free height of the front sheet pile. It considers a linear simplification of the Westergaard parabolic function.

Finally, the pseudo-static analysis performs a calculation by phases where the construction sequence is considered. For this reason, the model accounts for four phases:

- Phase 0: Initial phase, definition of the geotechnical profile and the sheet pile wall;

- Phase 1: Installation of the sheet piles and the tie rods;

- Phase 2: Dredging, excavation down to the corresponding seabed level;

- Phase 3: Surcharge load application;

- Phase 4: Seismic action, update of the soil parameters according to their seismic values and application of the Westergaard load.

3.4.2. Dynamic analysis

The dynamic analysis is performed using the Plaxis software. This section describes the main modelling features, as they are thoroughly presented in Chapter 2.

The dynamic analysis is carried out through a symmetrical FE model. This allows studying both horizontal directions of the seismic action when applying only one accelerogram (in one phase calculation). The main modelling features are:

- Hardening Soil Small Strain (HSSmall) is the constitutive soil model. For the static calculation, it is set as Drained whereas

for the dynamic calculation it is set to Undrained (A) in order to account for the excess of pore water pressure during the earthquake action;

- The soil-structure interaction is accounted for through the R_{inter} factor, which is equal to 0.66 for static and dynamic calculations;

- The tieback support system of the front sheet pile wall is a classic passive sheet pile anchor wall. Both sheet pile walls are modelled as plate elements while the tie rods are node-to-node anchor elements;

- In the dynamic calculation, the lateral boundary conditions are set to "Tied degrees of freedom" as dynamic conditions and "Free" as deformation conditions. The bottom boundary is set to "Compliant base" condition;

- An imposed horizontal line displacement at the bottom of the model defines the seismic action. The accelerogram is

introduced as a displacement multiplier in terms of acceleration with an initial value equal to 0.5 (see Section 2.6);

- Hydrodynamic pressures are considered using the added masses method. The additional mass is calculated using the Westergaard formula, as defined in Section 2.7.3.

Finally, the Plaxis calculation considers the following construction sequence:

- Phase 0: Initial phase to determine the initial conditions of the soil;

- Phase 1: Installation of the sheet pile system;

- Phase 2: Dredging down to the seabed level;

- Phase 3: Application of the loads (surface line load and added masses);

- Phase 4: Dynamic analysis of the seismic action, setting the undrained soil conditions and activating the line displacement.

3.5. Results

3.5.1. Case 1

In this case, a PGA of 0.10 g is considered in a sandy soil profile. The seabed level is at -7.5 m in case 1.1 and at -9.5 m in case 1.2.

The seismic calculations are performed under both design methods, the pseudo-static and the dynamic, and the structural resistance of the front sheet pile wall is assessed in terms of bending moments.

The results show that SGRM pseudo-static calculations overestimate largely the bending moments, whereas the FEM dynamic calculations allow the optimization of the sheet pile solution, that can be directly translated into material cost savings.

We can see that even in the case of a rather moderate earthquake (0.10 g PGA), there is a good optimization of the sheet pile solution when using FEM dynamic design.
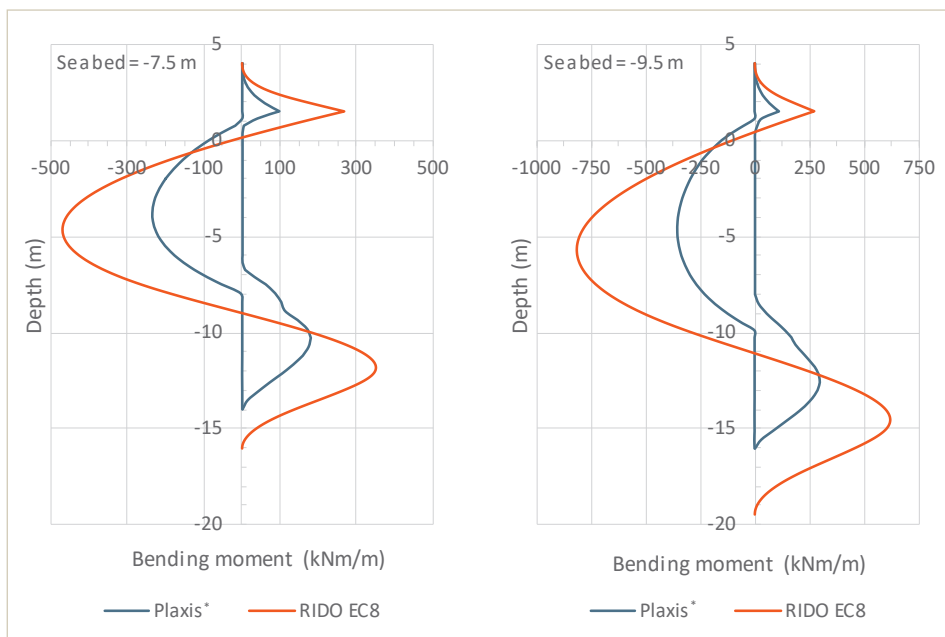
* For Plaxis dynamic calculations, the envelope of the bending moments from all the calculation steps is shown.

Figure 10. Bending moments at the front sheet pile wall for Case 1 (PGA = 0.10 g).

PGA 0.10 g	Pseudo-static (EN 1998-5)		FEM design		Material cost savings
	Length	Section	Length	Section	
	m	–	m	–	%
Case 1.1.	20.0	AZ 18-800**	18.0	AZ 18-800	10 %
Case 1.2.	23.5	AZ 20-800	20.0	AZ 18-800**	22 %

** Resulting section based on bending moment capacity. The recommended section might be different based on driveability and local conditions.
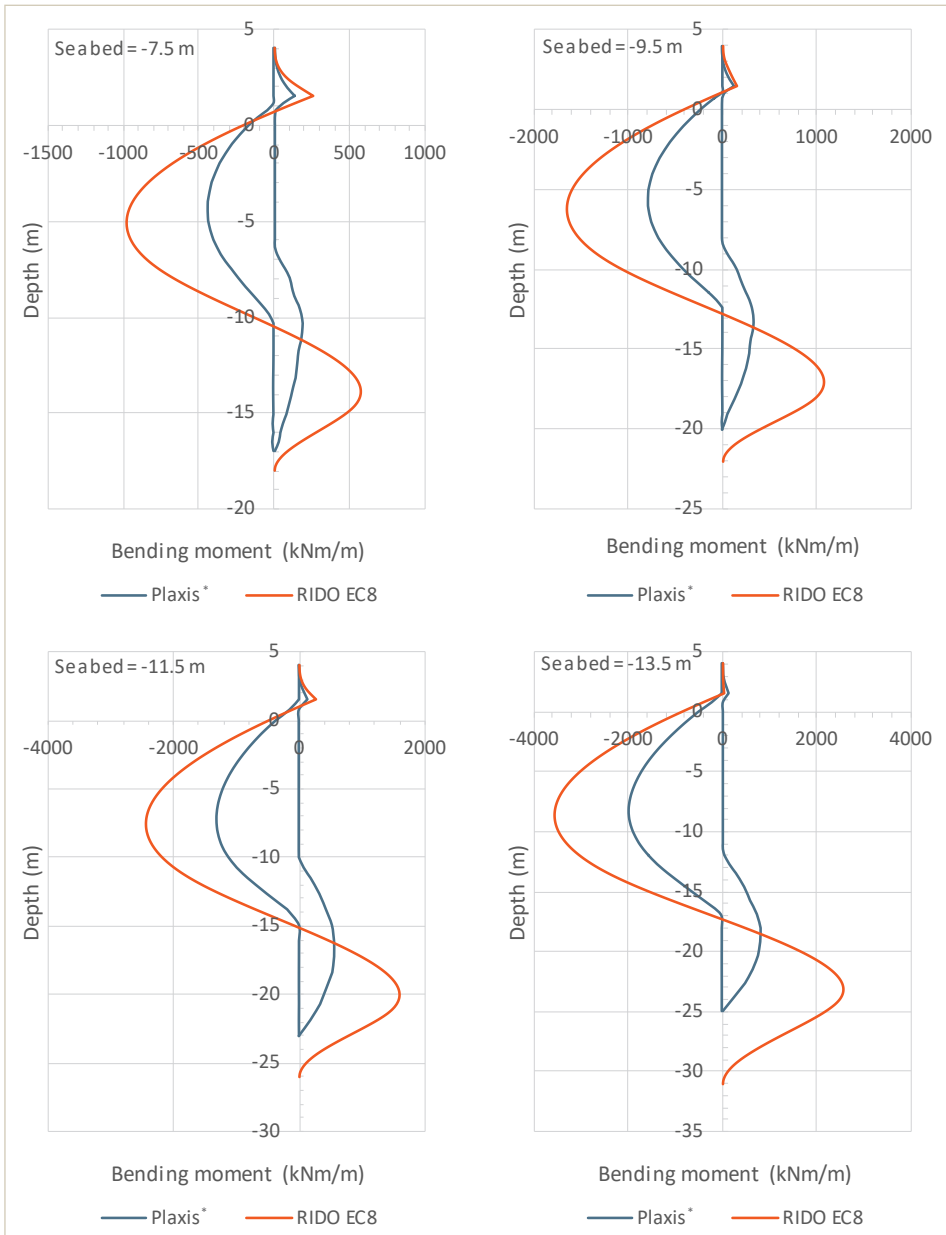Table 6. Summary of sheet pile properties for Case 1 (PGA = 0.10 g).

3.5.2. Case 2

- ### Case 2.1.

In this case, a PGA of 0.30 g is considered in a sandy soil profile. The seabed level is at –7.5 m, –9.5 m, –11.5 m, –13.5 m in cases 2.1.1, 2.1.2, 2.1.3, 2.1.4 respectively.

Again, and as expected, the results show that FEM calculations provide always lower bending moments compared to the pseudo-static approach.

[*] For Plaxis dynamic calculations, the envelope of the bending moments from all the calculation steps is shown.

Figure 11. Bending moments at the front sheet pile wall for Case 2.1 (PGA = 0.30 g).

When translated in terms of material cost, substantial savings are achieved (up to 48 %):

PGA 0.30 g	Pseudo-static (EN 1998-5)		FEM design		Material cost savings
	Length	Section	Length	Section	
	m	–	m	–	%
Case 2.1.1.	22.0	AZ 25-800	21.0	AZ 18-800[**]	25 %
Case 2.1.2.	26.0	AZ 36-700N	24.0	AZ 22-800	34 %
Case 2.1.3.	30.0	AZ 52-700	27.0	AZ 30-750	48 %
Case 2.1.4.	35.0	HZ 1080M C 12/ AZ 25-800	29.0	AZ 42-700N	46 %

[**] Resulting section based on bending moment capacity. The recommended section might be different based on driveability and local conditions.
Table 7. Summary of sheet pile properties for Case 2.1 (PGA = 0.30 g).

• Case 2.2.

In this case, a PGA of 0.40 g is considered in a sandy soil profile. The seabed level is at –7.5 m, –9.5 m, –11.5 m, –13.5 m in cases 2.2.1, 2.2.2, 2.2.3, 2.2.4 respectively.

As for the previous cases, the results show that FEM calculations always provide lower bending moments compared to the pseudo-static approach.
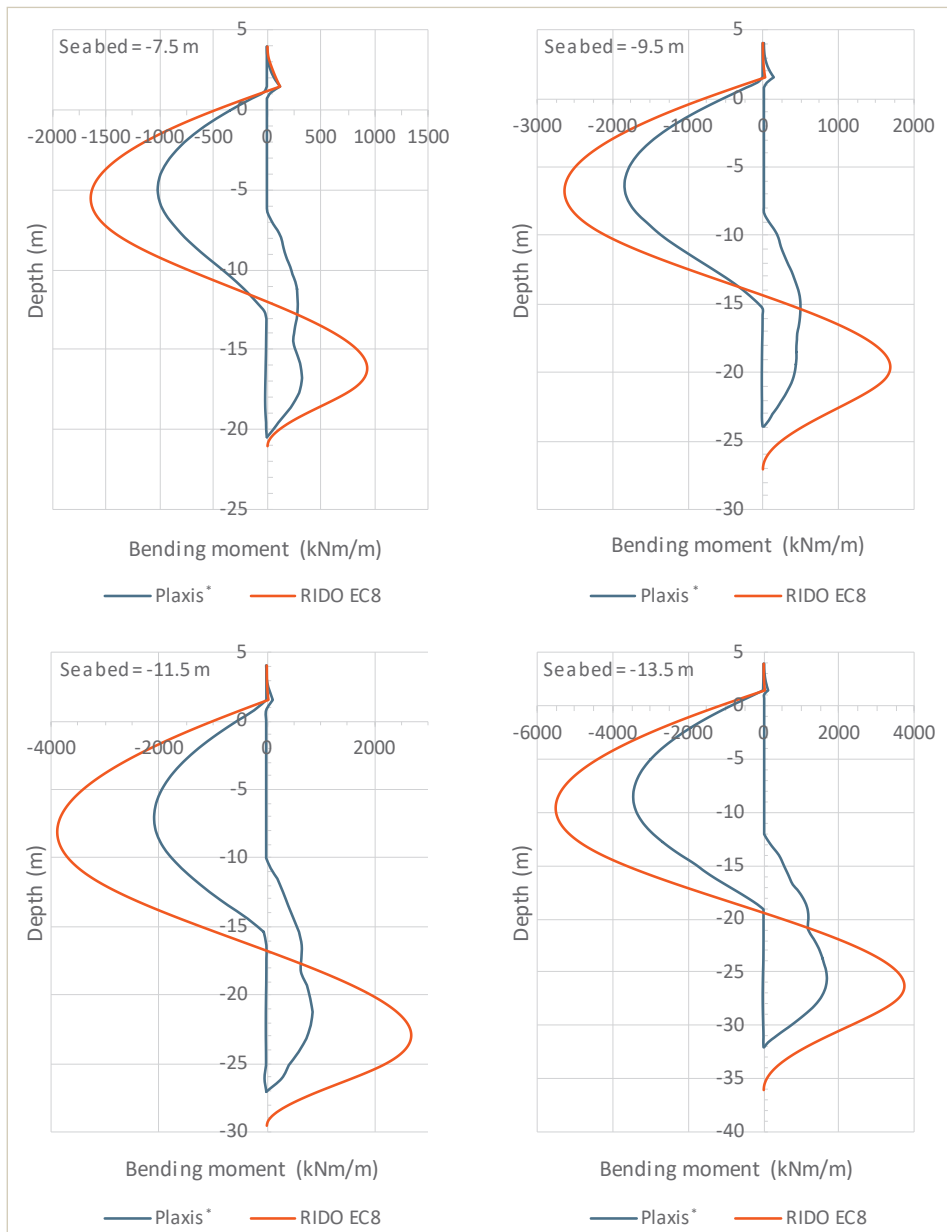
* For Plaxis dynamic calculations, the envelope of the bending moments from all the calculation steps is shown.

Figure 12. Bending moments at the front sheet pile wall for Case 2.2 (PGA = 0.40 g).

For the 4 studied cases, the FEM design provides a substantial optimization of the sheet pile solution. The subsequent material cost savings are up to 41 % for this case.

PGA 0.40 g	Pseudo-static (EN 1998-5)		FEM design		Material cost savings
	Length	Section	Length	Section	
	m	-	m	-	%
Case 2.2.1.	25.0	AZ 36-700N	24.5	AZ 25-800	26 %
Case 2.2.2.	31.0	HZ 1080M A 12/ AZ 25-800	28.0	AZ 40-700N	36 %
Case 2.2.3.	33.5	HZ 1080M D 12/ AZ 25-800	31.0	AZ 44-700N	37 %
Case 2.2.4.	40.0	HZ 1080M C 24/ AZ 25-800	36.0	HZ 1080M C 12/ AZ 25-800	41 %

Table 8. Summary of sheet pile properties for Case 2.2 (PGA = 0.40 g).

3.5.3. Case 3

This section studies the seismic situation with a PGA of 0.50 g in soft soil conditions. Under these conditions, the Mononobe–Okabe formula does not provide a solution, due to mathematical limitation.

Consequently, the FEM dynamic calculation can provide a design for high seismic actions in soft soil conditions whereas the SGRM pseudo–static approach is not adequate for carrying out a seismic design under such conditions.

In this case, the front sheet pile solution is:

PGA 0.50 g	Length	Section
	m	-
FEM design	34.0	AZ 40-700N

Table 9. Summary of sheet pile properties for Case 3 (PGA = 0.50 g).
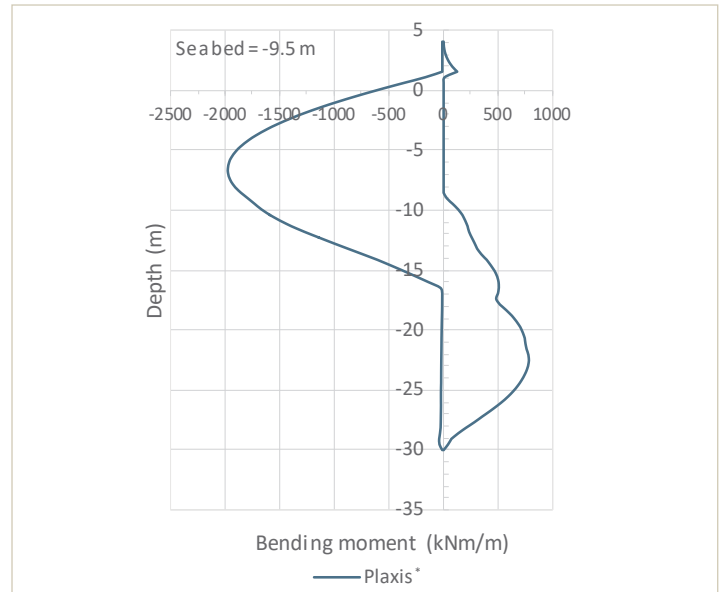
* The envelope of the bending moments from all the calculation steps is shown.
Figure 13. Bending moments at the front sheet pile wall for Case 3 (PGA = 0.50 g).

3.6. Italian Standard NTC 2018

The Italian standard NTC 2018 [7], which follows the same philosophy as EN 1998-5, proposes some amendments to the definition of the horizontal seismic coefficient. According to this standard, the seismic coefficient is defined as:

$$k_h = \alpha \cdot \beta \, \frac{a_{max}}{g} \quad with \quad a_{max} = (S_s \cdot S_T) \cdot a_g$$

Where:

S_S is the soil amplification factor;

S_T is the topographic factor;

a_g is the PGA of the seismic action.

α and β are factors accounting for the deformability of the soil and the deformability of the sheet piles respectively. They are determined using Figure 14.

In the graphs:

H is the total height of the retaining wall;

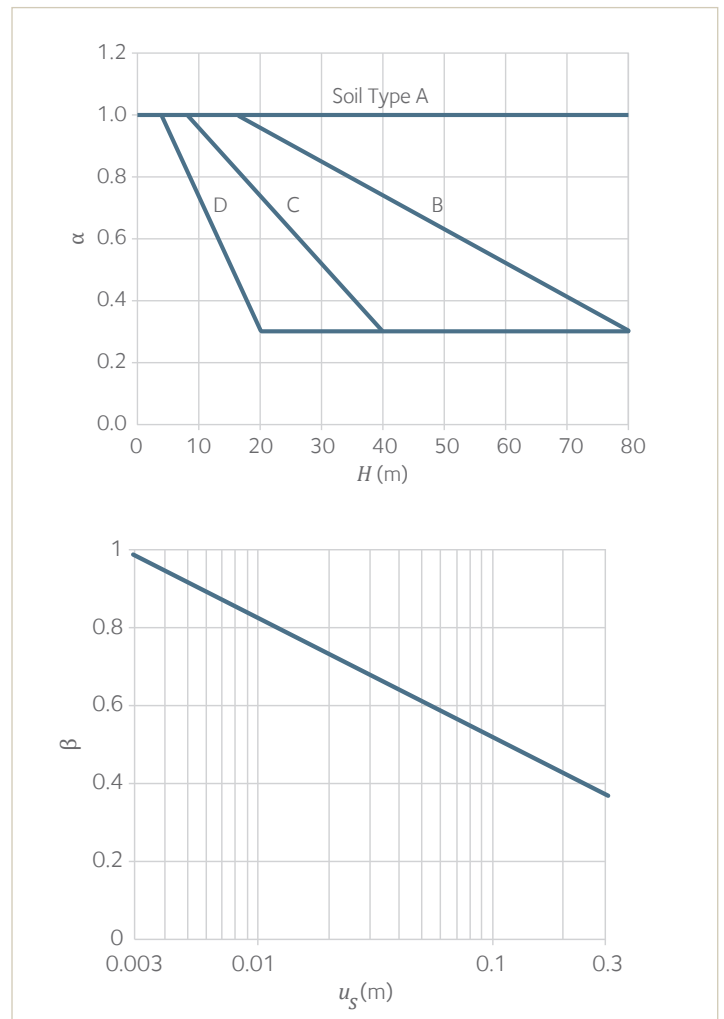u_s is the maximum permanent displacements of the retaining wall.

Figure 14. Graphics for the determination of α and β factors to obtain the seismic coefficient according to the NTC 2018 standard (Figures 7.11.2 and 7.11.3).

The previous definition of the horizontal seismic coefficient allows a better adjustment of the seismic action to each design scenario. Firstly, the soil amplification factor is not a constant value, it has a certain dependency on the peak ground acceleration value of the seismic action. Secondly, α and β parameters allow to consider the deformability of the sheet pile system.

On the one hand, NTC 2018 introduces the influence of the deformability of the soil when interacting with the structure through the α factor. On the other hand, the reduction factor β accounts for the deformation capacity of the sheet pile wall, which depends on the horizontal permanent displacements.

Comparing the NTC 2018 standard to the Eurocode standard, the Italian standard provides reduction factors higher than the Eurocode for all the studied cases.

The evaluation of the adequacy of the NTC 2018 standard for seismic analysis is performed in this section considering case 2.1, with a PGA equal to 0.30 g and a sandy soil profile. Figure 15 shows smaller differences between the NTC pseudo-static approach and the dynamic FEM approach, when compared to the Eurocode pseudo-static approach.
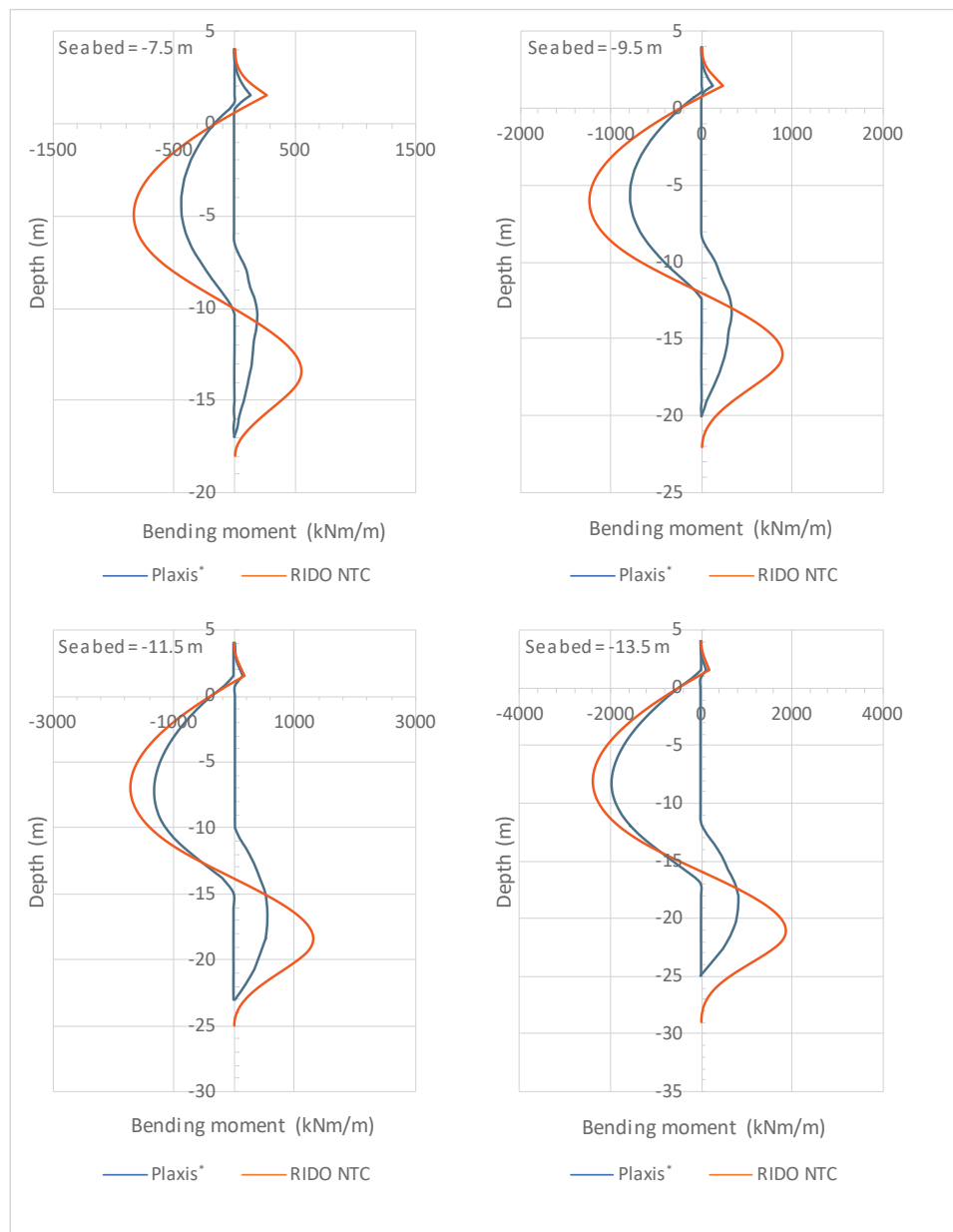
* For Plaxis dynamic calculations, the envelope of the bending moments from all the calculation steps is shown.
Figure 15. Bending moments at the front sheet pile wall for Case 2.1 (PGA = 0.30 g) using the NTC pseudo-static approach.

3.7. Summary

The study performed by SENER analysed the seismic design of sheet piles. It focused on comparing the pseudo-static design method, according to EN 1998-5 standard, with the FEM dynamic method. A quay wall design with steel sheet piles should include a complete set of geotechnical and structural verifications according to standards and best practices. However, the theoretical purpose of the study leads to some design simplifications:

- Liquefaction and scouring effects are not considered;

- Verification of the sheet pile system is based on a non-collapse basis. A performance-based design in terms of deformations and displacements is not included in the study. Generally, the performance-based design depends on the typology of the quay wall and the contractor's requirements;

- The study selects the Hardening Soil Small Strain constitutive model with undrained conditions in order to characterize the geotechnical seismic conditions. Other constitutive models may provide a different characterization of the soil conditions and the excess of pore water pressure. Normally, the selection of the constitutive model is subject to the local specific site conditions and the available geotechnical information.

Under the above-mentioned assumptions, the seismic design can reach a more realistic characterization with the dynamic method, since it accounts for:

- The seismic wave propagation;

- The influence of hydrodynamic pressure;

- The excess of pore water pressure.

Taking into account previous assumptions, the study has conducted the seismic design for various cases, considering different seismic actions with a PGA from 0.10 g to 0.50 g, and seabed levels from -7.5 m to -13.5 m. All the cases have used sandy soil conditions except for case 3 which used a clayey silty soil.

Once the geotechnical profile, the sheet pile system and the accelerogram are defined, the dynamic analysis can perform a calculation accounting for soil-structure interaction and its associated effects under seismic action. On the other hand, the pseudo-static approach needs particular implementation for substitution parameters. Soil-structure interaction effects, like for instance the seismic action and the hydrodynamic pressure, are difficult to model in an appropriate manner. Hence, the FEM dynamic analysis provides a more realistic approach, accounting for all the seismic design factors impacting the sheet pile systems.

As a result, the study shows that the FEM dynamic analysis provides lower values of design forces which is more realistic as being witnessed in experimental studies (centrifuge testing), and thus leads to a substantial optimisation of the sheet pile solution. Moreover, the Mononobe-Okabe formula is not suitable for high seismic actions especially in soft soil conditions, because it reaches the limits of its application spectrum.

Finally, the study has evaluated the definition of the seismic horizontal coefficient according to the Italian standard NTC 2018. This standard makes amendments to the expression used in EN 1998-5 for the seismic coefficient. It introduces two factors taking into account the deformability of soil and the flexural behaviour of the sheet pile wall. As a result, the seismic coefficient can be better adjusted for each design case..

4. Conclusion

The results of the studies carried out by SENER highlight the importance of developing more advanced methods for sheet pile design. Unfortunately, the current European standards provide overly conservative approaches when dealing with sheet piles' seismic design, and the rules for using advanced design methods like Finite Element Modelling are not clearly defined. ArcelorMittal Sheet Piling and its partners are taking an active part in the upcoming update of the European standards to fill this gap.

This study clearly shows that accurate FEM dynamic design allows significant savings and yield economical sheet pile solutions even for cases not accessible with the traditional approach. If combined with a performance-based design, further savings can be achieved in a wider geographical range.

Advanced design methods used in an efficient way yield economical sheet pile solutions in high seismic areas.

5. References

[1]   EN 1993-5: 2007. Eurocode 3: Design of steel structures. Piling.

[2]   EN 1997-1: 2004. Eurocode 7: Geotechnical design. General rules.

[3]   EN 1998-1: 2004. Eurocode 8: Design of structures for earthquake resistance.
       General rules, seismic actions and rules for buildings.

[4]   EN 1998-5: 2004. Eurocode 8: Design of structures for earthquake resistance.
       Foundations, retaining structures and geotechnical aspects.

[5]   EN 1990:2002. Basis of Structural Design.

[6]   PIANC Working Group No 34. Seismic Design Guidelines for Port Structures.

[7]   BS 6349-2: 2019. Maritime works. Code of practice for the design of quay walls, jetties and dolphins.

[8]   NTC:2018. Norme Tecniche per le Costruzioni.

[9]   Gazetas, G., et. al (2016). Seismic analysis of tall anchored sheet-pile walls. Soil Dynamics and Eartquake Engineering.

[10]   US Army Corps of Engineers (2003).
       Time-History Dynamic Analysis of Concrete Hydraulic Structures. Engineering Manual EM 1110-2-6051.

[11]   PLAXIS Material Models Manual (2019).

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