الجمهورية الجزائرية الديمقراطية الشعبية

People's Democratic Republic of Algeria

وزارة التعليم العالي والبحث العلمي

Ministry of Higher Education and Scientific Research

المدرسة الوطنية العليا لعلوم البحر وتهيئة الساحل

National Higher School of Marine Science and Coastal Development

Final Year Project Report Submitted in Partition for the Engineering Degree and Master's Degree in Marine Sciences

Option : Coastal Engineering and Development Specialisation.

Theme :

Measuring Sheet Pile Wall Quay
Comparative Study of Calculation Methods
-Case of The Marina Bay of Algiers-

Presented by:

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Presented on 22/06/2026, In front of the jury composed of:

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2025/2026

ACKNOWLEDGMENTS

We begin by expressing our gratitude to Allah for granting us the health, strength, and determination necessary to undertake and complete this thesis.

First and foremost, we would like to express our sincere appreciation to our supervisor, **Mr. A. LAOUEDJ**, whose guidance and support were essential to the realization of this work. His patience, rigor, and constant availability greatly contributed to the development of this thesis. His expertise in marine geotechnics, along with his insightful advice and encouragement during moments of uncertainty, played a crucial role in shaping our understanding of soil–structure interaction and the assumptions behind the different design approaches. The time he generously devoted to discussing technical challenges and encouraging critical thinking deeply enriched this research and motivated us to pursue excellence.

We also extend our thanks to **Mme Salem Cherif.Y**, Head of the Department, for her support and attention throughout our academic journey. Our sincere appreciation also goes to all the professors at **ENSSMAL** who contributed to our education and professional development. We are equally grateful to the technical staff who assisted us in interpreting and understanding the complex geotechnical data from the Marina Bay site. We also wish to thank our colleagues and classmates for their encouragement and moral support during this demanding period. The many hours spent studying, discussing ideas, and overcoming challenges together will remain memorable experiences.

Our sincere gratitude is also addressed to **Mme. Salem Cherif .Y**, President of the jury, and **Mr. Dehmani. AEA** Examinator, for accepting to evaluate this work. Their expertise, constructive remarks, and rigorous questions during the pre-defense were invaluable in improving the quality and clarity of this thesis. We deeply appreciate the time and effort they dedicated to reviewing this manuscript and providing insightful feedback.

We would also like to acknowledge our own perseverance and commitment throughout this journey. Despite the challenges including long nights of research, the difficulty of synthesizing complex theoretical concepts, moments of doubt, and the pressure of balancing coursework with thesis preparation, we remained determined to complete this project. This thesis reflects not only our academic efforts but also our resilience, collaboration, and shared dedication. Through this experience, we learned that research is not only about obtaining answers, but also about asking meaningful questions and embracing intellectual challenges.

Finally, we would like to express our deepest gratitude to our families and friends for their unwavering love, patience, and encouragement. To our parents, whose sacrifices made our education possible and who always believed in our abilities, and to our siblings who supported us during stressful times, this achievement is also yours. We are equally grateful to our friends who understood our absence during this demanding period and who brought moments of encouragement and balance to our lives.

DEDICATION

I dedicate this work to my dearest parents, may Allah protect them. Your sacrifices, guidance, and unconditional love have shaped the person I am today. This thesis is a reflection of your faith in me.

To my very dear brothers Haithem, Sebti, Akrem and Alla eddine for their infectious energy and for reminding me of the importance of joy and laughter.

To my dear aunt, uncle Lazher and my cousins, for taking care of me throughout this time and providing a family atmosphere and being a source of strength.

To my friends Malek, Seif, Nadji, Fares, Soheyb, Ishak, and all my colleagues, for the shared memories, the late-night study sessions, and the unwavering support. You made this journey bearable and even enjoyable.

To everyone who believed in me, thank you.

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GENERAL INTRODUCTIO

Port infrastructures play a crucial role in the economic and social development of a country or region. They facilitate international trade, support local industries, and contribute to the growth of coastal cities. **(Gaythwaite, 2016).** Among the key elements of these infrastructures, sheet pile walls are widely used as retaining structures, particularly in areas where the soil has low bearing capacity and for moderate wall heights **(USACE, 2022).**

The study of sheet pile wall behavior in port structures involves a thorough analysis of the interaction between the soil and the wall. This interaction is influenced by several factors, including soil properties, contact conditions at the soil-wall interface, and applied loads **(Lambe & Whitman, 1979).** To guarantee the stability of port structures, it is essential to conduct rigorous geotechnical studies, which include modeling of soil-structure interaction **(Clayton et al., 2014)**

Several methods are available to study soil-structure interaction, ranging from theoretical approaches to numerical modeling. Classical limit equilibrium methods, such as the Blum method, provide a simplified and rapid approach for preliminary design **(Blum, 1931; Teng, 1962).** More advanced numerical methods, particularly the Finite Element Method (FEM), allow for realistic simulation of non-linear soil behavior and complex boundary conditions. Specialized software such as PLAXIS 2D, GEO5, SPW 2006, and Prosheet are widely used in practice for the analysis and design of sheet pile walls **.**

In this work, we have chosen to use a multi-method approach to dimension the sheet pile quay wall at the Marina Bay d'Alger port, focusing on **Quay G2** (water depth 3.00 m, Overload 10 kN/m²) as defined in the project documentation. **(BTP INFRAESTRUCTURAS 2018)**

The software used include **PLAXIS 2D**, **GEO5**, **SPW 2006**, and **Prosheet**. These tools represent different levels of sophistication: from classical limit equilibrium (Prosheet) to finite element methods (PLAXIS, GEO5) and beam-on-elastic-foundation methods (SPW). Recent research has highlighted the importance of evaluating the safety and reliability of different design codes and methods, especially for structures subjected to high water loadings and complex ground conditions **(Mattos & Garcia, 2025)**

The objective of this work is to model the soil-structure interaction and to perform a comparative study of different calculation methods for sheet pile walls, evaluating their accuracy, limitations, and practical applicability for the Marina Bay project.

To achieve this objective, the work is structured into six chapters.

Chapter I presents generalities on port structures with a specific focus on sheet pile quay walls, including their advantages, limitations, and classification.

Chapter II provides a comprehensive overview of the theoretical foundations of sheet pile walls, covering types, earth pressure theories, classical design methods (simple thrust, embedded, equivalent beam), and numerical modeling approaches.

Chapter III describes the Marina Bay d'Alger project, its geographical location, the geotechnical investigation (SPT, CPT), and the specific characteristics of Quay G2.

Chapter IV is devoted to the predimensioning of the sheet pile wall using three classical analytical methods, providing initial estimates of embedment depth, anchor force, and bending moment.

Chapter V details the design methodology and numerical modeling using PLAXIS 2D, GEO5, SPW 2006, and Prosheet, including model setup, boundary conditions, and load combinations.

Chapter VI presents and discusses the results obtained from the numerical simulations, compares bending moments, displacements, and shear forces.

Through this study, we aim to contribute to a better understanding of sheet pile quay wall design, proposing technical solutions adapted to the specific conditions of the Marina Bay d'Alger port, while providing a comparative reference for engineers working on similar projects in Algeria.

Part 1: Theoretical Study

CHAPTER I: GENERALITIES ON PORT STRUCTURES

I.1. INTRODUCTION

Ports have been vital hubs of human activity for millennia, serving as gateways for trade, cultural exchange, and economic development. From the ancient Phoenician ports to the modern mega-ports of today, these infrastructures have continuously evolved to meet the changing demands of maritime transport. The design and construction of port structures require a deep understanding of maritime environments, geotechnical conditions, and the complex interactions between structures, soil, and water. **(Gaythwaite, 2016)**

I.2. DEFINITION OF A PORT

A port is a maritime or riverine facility comprising one or more harbors where ships can dock to load and discharge cargo and passengers. It typically includes quays, wharves, piers, docks, terminals, and the necessary infrastructure for cargo handling, storage, and transportation. Ports may be natural or artificial and can range from small fishing harbors to massive industrial complexes handling millions of tons of cargo annually. **(Gaythwaite, 2016)**

The essential functions of a port include providing safe access and shelter for vessels, enabling the transfer of goods and passengers between sea and land, and supporting associated industrial and logistics activities. Ports are complex systems that integrate civil, mechanical, and electrical engineering with operational and management considerations. **(Gaythwaite, 2016)**

Figure 1: *Main Physical Elements of a Port.* ***(Notteboom et al., 2021)***

I.3. CLASSIFICATION OF QUAY WALLS

Quay walls can be classified based on their structural behavior and the type of foundation soil. **(Thoresen, 2014)**

I.3.1. BERTHING STRUCTURES ON GOOD BEARING CAPACITY SOIL

When the foundation soil has adequate bearing capacity, gravity-type structures are often the most economical solution. These structures rely on their self-weight to resist overturning and sliding forces. **(Thoresen, 2014)**

I.3.1.1. CONCRETE BLOCK QUAYS

These quays consist of stacked concrete blocks that rely on their self-weight to resist earth pressures and other loads. The blocks are typically precast and placed in a staggered pattern to ensure stability. Key characteristics include **: (Cornick, 1969)**

- **Advantages:** Simple construction, durable, good resistance to wave action, no corrosion issues. Concrete block quays have a long service life and require minimal maintenance.
- **Disadvantages:** Require good foundation soil, heavy, susceptible to differential settlement, time-consuming construction. The weight of the blocks requires careful foundation preparation, and settlement can lead to misalignment.
- **Applications:** Suitable for soils with high bearing capacity, often used in repair and maintenance facilities, fishing ports, and areas where heavy loads are not expected.

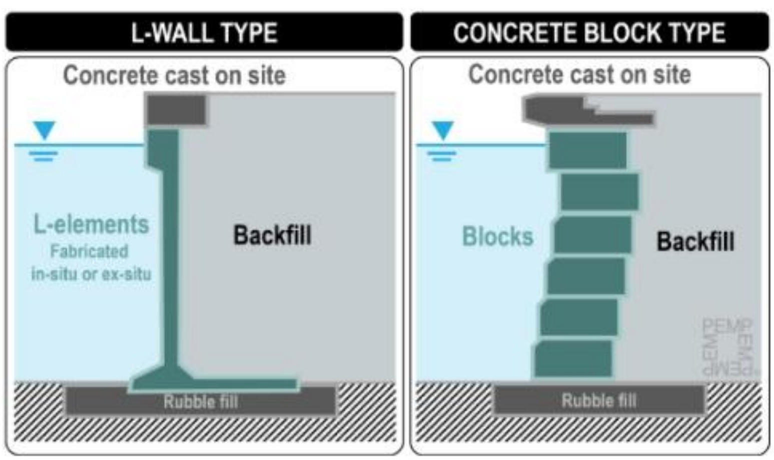

Figure 2: *cross-section of a concrete block quay wall.* ***(Notteboom et al., 2026)***

I.3.1.2. CAISSON QUAYS

Caisson quays are large, hollow concrete structures that are floated to the site, sunk into place, and then filled with sand or rock. They combine the functions of a retaining wall and a foundation. Key characteristics include: **(Gaythwaite, 2016)**

- **Advantages:** Rapid installation, high structural integrity, good seismic resistance if properly designed, can be constructed in dry dock and towed to site. Caissons provide a monolithic structure with excellent durability.
- **Disadvantages:** Require deep water for float-in, specialized construction facilities, susceptible to scour, complex design. The construction of caissons requires a casting basin or dry dock, and the float-in operation requires calm water conditions.
- **Applications:** Deep water ports, exposed locations, major commercial ports where rapid construction is required.

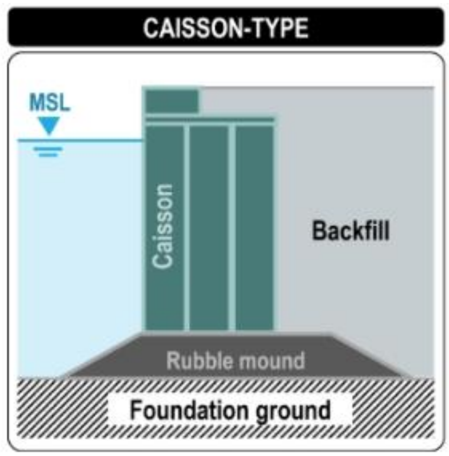

Figure 3: *cross-section of a caisson quay wall.* ***(Notteboom et al., 2026)***

I.3.2. BERTHING STRUCTURES ON LOW BEARING CAPACITY SOIL

When foundation soils have low bearing capacity, such as soft clays, silts, or loose sands, piled or sheet pile structures are preferred as they transfer loads to deeper, more competent strata. **(Thoresen, 2014)**

I.3.2.1. SHEET PILE QUAY WALLS

Sheet pile walls are flexible retaining walls consisting of interlocking vertical elements driven into the ground. They derive their stability from embedment in the soil and, for higher walls, from anchor systems. Key characteristics include: **(USACE, 2022)**

- **Advantages:** Rapid installation, relatively light weight, adaptable to various soil conditions, can be cantilevered or anchored, economical for moderate heights. Sheet pile walls can be installed quickly with minimal site preparation.
- **Disadvantages:** Limited height for cantilever walls, requires good driving characteristics in soil, susceptible to corrosion in marine environments, deflection can be significant. Steel sheet piles require corrosion protection in aggressive marine environments.

- **Applications:** River walls, harbor quays, cofferdams, temporary excavations, waterfront structures. Sheet pile walls are widely used in port development due to their versatility and cost-effectiveness.

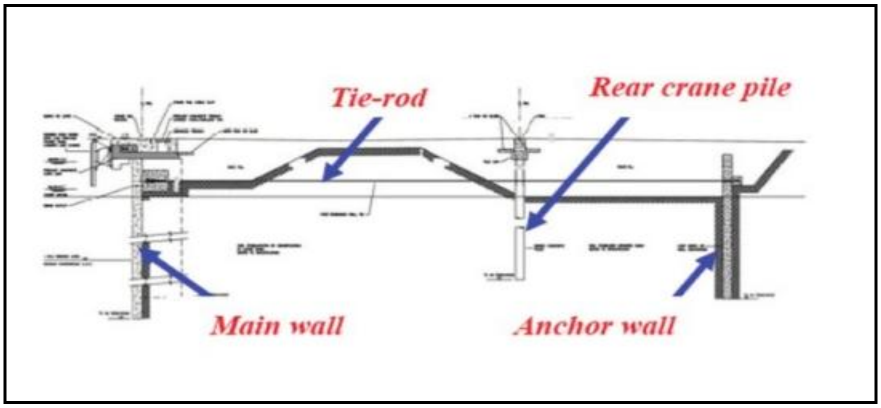

Figure 4: *Cross-section of quay wall system.* ***(Babak Ebrahimian, 2009)***

I.3.2.2. MIXED QUAYS

Mixed quays combine sheet piles with a relieving platform or a concrete superstructure. The relieving platform reduces the earth pressure on the sheet pile by carrying part of the load directly to piles. Key characteristics include: **(PIANC, 2015)**

- **Advantages:** Reduced bending moments in sheet piles, can accommodate larger heights, good for heavy Overload loads. The relieving platform allows for deeper water depths and heavier loads than simple sheet pile walls.
- **Disadvantages:** More complex construction, higher cost than simple sheet pile walls. The combination of structural elements requires careful coordination during construction.
- **Applications:** Deep water quays, container terminals, heavy load areas where sheet piles alone would be insufficient.

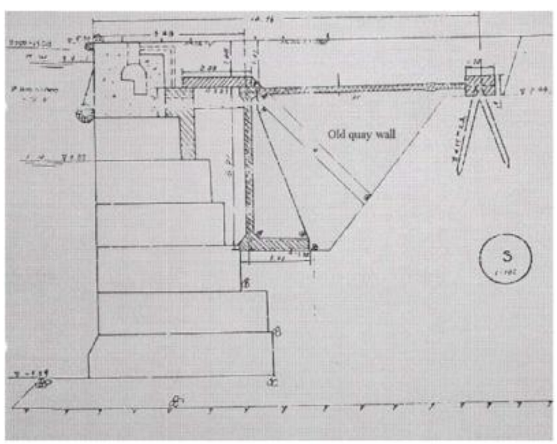

Figure 5: *Cross-section of two types of quay wall constructed between 1915 and 1923 in the port Dalian, China.* ***(J.G. De Gijt, 2010)***

I.3.2.3. PILED QUAYS

Piled quays consist of a deck structure supported on vertical or raked piles. The piles transfer all loads to deeper bearing strata. Key characteristics include: **(PIANC, 2015)**

- **Advantages:** Suitable for very poor soils, minimal excavation, good seismic performance, adaptable to various geometries. Piled quays can be constructed in almost any soil condition and provide excellent seismic resistance due to their flexibility.
- **Disadvantages:** High cost, requires specialized piling equipment, susceptible to corrosion in marine environment. The cost of piles and the need for heavy piling equipment make this option more expensive than sheet pile walls.
- **Applications:** Deep water berths, ferry terminals, offshore structures where soil conditions preclude other solutions.

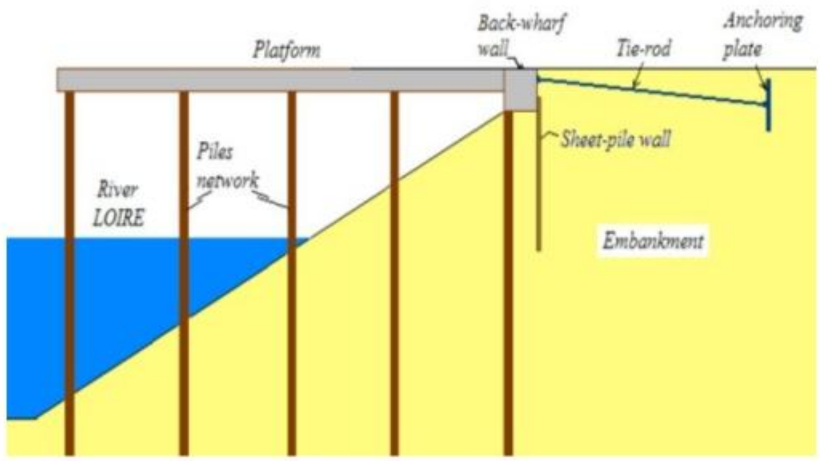

Figure 6: *Cross-section of an on-piles quay wall.* ***(Yáñez-Godoy et al., 2008)***

Table 1: *Comparison of quay wall types*

Type	Advantages	Disadvantages	Typical Applications
Concrete Block	Durable, simple construction	Heavy, requires good soil	Fishing ports, small harbors
Caison	Rapid installation, high integrity	Specialized construction	Deep water ports, Major terminals
Sheet Pile	Economical, rapid installation	Corrosion, limited height	River walls, harbor quays
Mixed	Reduced moments, heavy installation	Complex construction	Container terminals
One Pile	Suitable for poor soils	High cost	Deep water berths

I.4. CONCLUSION

This chapter has presented a general overview of port structures, highlighting the diversity of quay wall types and the factors influencing their selection. The choice between gravity structures (blocks, caissons) and flexible structures (sheet piles, piled quays) depends primarily on foundation conditions, wall height, loads, and economic considerations.

For the Algiers Bay Marina project, with its specific geotechnical conditions involving sandy soils with variable density, the sheet pile quay wall solution emerges as an appropriate variant, particularly for moderate-height sections. The following chapters will delve deeper into the theoretical foundations of sheet pile wall design and describe the specific project context.

CHAPTER II: SHEET PILE WALLS

II.1. INTRODUCTION

Sheet pile walls are among the most common types of retaining structures used in port and waterway engineering. Their flexibility, ease of installation, and cost-effectiveness make them an attractive solution for a wide range of applications, from temporary cofferdams to permanent quay walls. This chapter provides a comprehensive overview of sheet pile walls, including their types, materials, design methods, and key design considerations. **(USACE, 2022)**

II.2. DEFINITION AND GENERAL PRINCIPLES

Sheet piles are long, interlocking structural sections driven into the ground to form a continuous wall. They are typically made of steel, although concrete, wood, aluminum, and vinyl sheet piles are also available for specific applications. The interlocking joints (clutches) allow the piles to be connected, forming a watertight or near-watertight barrier. **(ArcelorMittal, 2020)**

The fundamental principle of sheet pile wall behavior is the mobilization of earth pressures to resist the applied loads. The wall derives its stability from two primary mechanisms: **(USACE, 2022)**

- **Embedment Resistance:** The portion of the wall embedded below the dredge level develops passive earth pressure that resists the active thrust from the retained soil. The depth of embedment must be sufficient to provide the required passive resistance.
- **Anchor Resistance (for anchored walls):** Tie rods connected to an anchor wall or deadman provide additional resistance, reducing the required embedment depth and bending moments. The anchor system transfers part of the load to the soil behind the active zone.

The wall must also have adequate structural capacity to resist bending moments and shear forces induced by the applied loads. The sheet pile section must be selected to provide the required section modulus while accounting for potential corrosion in marine environments.

II.3. TYPES OF SHEET PILES

II.3.1. STEEL SHEET PILES

Steel is the most common material for sheet piles due to its high strength-to-weight ratio, good driving characteristics, and availability in various profiles. Steel sheet piles are manufactured by hot-rolling or cold-forming processes. **(ArcelorMittal, 2020)**

Figure 7: *Steel sheet pile sections.**(U Sections | ArcelorMittal Sheet Piling, n.d.)***

Hot-Rolled Sections:

Hot-rolled sheet piles are produced by passing heated steel billets through a series of rollers to achieve the desired profile. This process results in sections with high strength and good interlock performance. Common profiles include:**(ArcelorMittal, 2020)**

U-type: Symmetrical profile with a single interlock at each end. U-type piles are often used in cellular cofferdams and applications where interlock tension is critical. They have moderate section modulus and are suitable for a wide range of applications.

Z-type: Asymmetrical profile with high section modulus, ideal for retaining walls where bending resistance is the primary concern. Z-type piles have a shape that resembles the letter Z, with interlocks at the flange tips. They provide excellent bending resistance and are widely used in quay walls and deep excavations.

Straight Web: Flat profile used for cellular cofferdams and applications where interlock tension is critical. Straight web piles have a simple rectangular cross-section and are designed primarily for tension rather than bending.

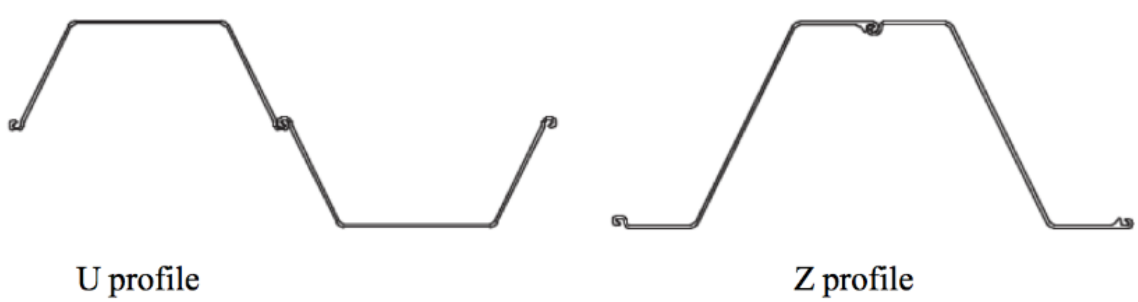

Figure 8: *Hot-rolled steel sheet pile profiles (U-type, Z-type).* ***(Sheet Piles - Steel Piling Group, n.d.)***

Cold-Formed Sections:

Cold-formed sheet piles are produced by cold forming from steel coils. This process is more economical than hot-rolling but results in lower strength and stiffness. Cold-formed sections are typically used for temporary works or low-height walls where structural demands are modest. **(ArcelorMittal, 2020)**

Steel sheet piles are manufactured according to various standards, including **ASTM A328, A572**, and **A690**. For marine environments, **ASTM A690** includes alloying elements (copper and nickel) that provide enhanced corrosion resistance. **(USACE, 2022)**

Table 2: *Geometric properties of some standard steel sheet piles (GU, AZ series)*

Profile	Height (mm)	Web Thickness (mm)	Section Area (cm^2/m)	Section Modulus (cm^2/m)	Moment of Inertia (cm^4/m)
GU 6N	309	6.0	120.1	1245	45.003
AZ 12	312	8.5	140.2	1500	54.200
AZ 18	360	9.5	180.5	2100	78.500
AZ 26	420	10.5	220.8	2800	112.000

II.3.2. CONCRETE SHEET PILES

Concrete sheet piles are precast reinforced or prestressed elements with tongue-and-groove or grouted joints. They are typically 6 to 12 inches thick and 30 to 48 inches wide. Key characteristics include: **(USACE, 2022)**

- **Advantages:** Good durability in marine environments, high resistance to abrasion, can support axial loads, no corrosion concerns. Concrete sheet piles are particularly suitable for areas with high abrasion from sediment-laden water or where steel corrosion is a major concern.
- **Disadvantages:** Heavy, difficult to drive, susceptible to damage during handling and installation, requires careful quality control. The weight of concrete piles requires heavy lifting equipment, and they are more susceptible to damage during driving than steel piles.
- **Applications:** Marine structures, stream beds with high abrasion, permanent retaining walls where steel corrosion is a concern. Concrete sheet piles are often used in combination with steel piles in mixed wall systems.

Figure 9: *Concrete sheet pile sections.* ***(Speed Concrete Industries Sdn Bhd - , n.d.)***

II.3.3. WOOD SHEET PILES

Wood sheet piles are typically used for temporary structures or low-height walls in non-critical applications. They can be independent or tongue-and-groove sections. Key characteristics include: **(USACE, 2022)**

- **Advantages:** Low cost, readily available, easy to handle. Wood piles are lightweight and can be installed with simple equipment.
- **Disadvantages:** Limited strength, susceptible to decay and marine borers, short service life. Wood piles require preservative treatment and are not suitable for permanent marine structures in aggressive environments.
- **Applications:** Temporary cofferdams, low-height retaining walls in freshwater environments, and applications where long-term durability is not required.

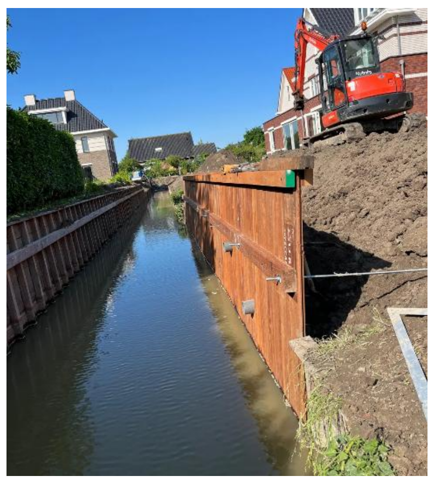

Figure 10: *Wooden sheet pile sections.* ***(Damwand Plaatsen - Stolze Grondverzet, n.d.)***

II.3.4. ALUMINUM AND VINYL SHEET PILES

Aluminum sheet piles are available as interlocking corrugated sheets or Z-type sections. They are lightweight and corrosion-resistant but have lower strength than steel. Key characteristics include: (USACE, 2022)**(USACE, 2022)**

- **Advantages:** Lightweight, corrosion-resistant, easy to handle and install. Aluminum piles are particularly suitable for applications where weight is a concern or where corrosion is severe.
- **Disadvantages:** Lower strength than steel, higher cost, susceptible to galvanic corrosion when in contact with other metals. Aluminum piles require careful design to avoid overstressing.

Vinyl sheet piles are used for shoreline erosion control and low-height bulkheads where structural loads are minimal. They are made from PVC or other polymers and offer excellent corrosion resistance but very low strength. These materials require careful evaluation of material properties and are typically used with tiebacks. **(USACE, 2022)**

Table 3: *Material properties for different sheet pile types*

Material	*Yield strength (MPa)*	*Modulus of Elasticity (GPa)*	*Density (kg/m³)*	*Corrosion Resistance*
Steel S240 GP	*240*	*210*	*7850*	*Moderate (requires protection)*
Steel S355 GP	*355*	*210*	*7850*	*Moderate (requires protection)*
Concrete HA-35	*35 (compressive)*	*30*	*2400*	*Excellent*
Aluminum Alloy	*150-250*	*70*	*2700*	*Good*
Wood	*10-20 (bending)*	*10*	*600-800*	*Poor in marine environment*
Vinyl/PVC	*10-30*	*2-3*	*1300-1500*	*Excellent*

II.4. STRUCTURAL SYSTEMS

II.4.1. CANTILEVER WALLS

Cantilever sheet pile walls derive their support solely from the embedment in the soil. They are suitable for limited wall heights (typically less than 5-6 m) where the passive resistance of the soil below the dredge line is sufficient to resist the active thrust from the retained soil. The wall behaves as a vertical cantilever beam, with the maximum bending moment occurring near the dredge line.**(USACE, 2022)**

The behavior of a cantilever wall can be described as follows:

- The wall rotates about a point near the toe, with the upper portion moving away from the retained soil and the lower portion moving into the retained soil.
- Active earth pressures develop on the retained side above the rotation point.
- Passive earth pressures develop on the dredge side below the rotation point.
- A counteracting passive pressure develops on the retained side below the rotation point (counterbalance).

The design of cantilever walls involves determining the required embedment depth such that the passive resistance provides an adequate factor of safety against rotational failure. The classical approach assumes that the wall rotates as a rigid body, with a linear distribution of earth pressures. More advanced methods account for wall flexibility and non-linear soil behavior.

II.4.2. ANCHORED WALLS

For wall heights exceeding the economical limit for cantilever walls, an anchor system is provided near the top of the wall. The anchor significantly reduces the bending moment and required embedment depth. The anchor system typically consists of: **(USACE, 2022)**

- **Tie Rods:** Steel bars or tendons that transfer the anchor force from the wall to the anchor. Tie rods are typically installed horizontally or with a slight inclination and are spaced at regular intervals along the wall.
- **Wales:** Horizontal beams attached to the sheet piles that distribute the tie rod forces to the wall. Wales are typically made of steel channels or wide-flange beams and are designed to resist bending between tie rod locations.
- **Anchor:** A structure that develops the required resistance through interaction with the soil or rock. Common types include:
    - **Continuous Anchor Wall:** A sheet pile or concrete wall installed parallel to the main wall. The anchor wall develops passive pressure on its front face and active pressure on its back face, with the net resistance providing the anchor force.
    - **Deadman:** A concrete block or short wall that provides resistance through passive pressure. Deadman are typically used for smaller walls or where space is limited.
    - **Grouted Anchors:** Tendons grouted into competent soil or rock. Grouted anchors are installed by drilling a hole, inserting a tendon, and grouting the bonded length. They are suitable for deep anchors where other types are not feasible.
    - **Pile Anchors:** Tension piles or pile groups that develop resistance through skin friction or end bearing. Pile anchors are used where soil conditions are suitable for driven or bored piles.

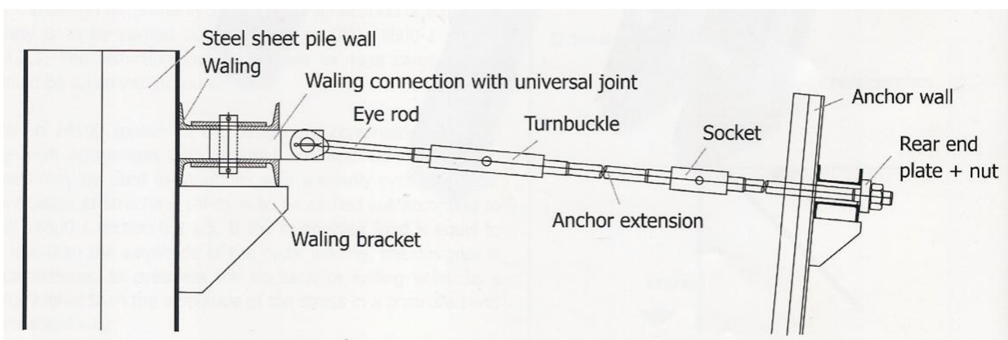

Figure 11: *Schematic of an anchored sheet pile wall.* ***(Anchorages - Piles&machinery, n.d.)***

The design of anchored walls can be performed using two classical approaches: **(USACE, 2022)**

Free Earth Support Method: Assumes that the wall rotates as a rigid body about the anchor point, with no fixity at the toe. The embedment depth is determined from moment equilibrium about the anchor, and the anchor force is determined from horizontal force equilibrium. This method results in a longer embedment but a simpler analysis.

Fixed Earth Support Method: Assumes that the toe is clamped, creating a point of contra-flexure. This method results in a smaller maximum bending moment but requires a more complex analysis to locate the point of contra-flexure. The fixed earth method is more representative of the actual behavior of flexible walls.

II.5. EARTH PRESSURE THEORIES

II.5.1. ACTIVE, PASSIVE, AND AT-REST PRESSURES

The magnitude of earth pressure on a retaining wall depends on the movement of the wall relative to the soil Three distinct states can be identified: **(Terzaghi & Peck, 1967)**

- **At-Rest Pressure (K_0):** The lateral pressure exerted by the soil when the wall is perfectly rigid and does not move. This state occurs when the wall is supported against any lateral displacement, such as in braced excavations or massive concrete structures. For normally consolidated soils, Jaky (1944) proposed the empirical relationship:

$$K_0 = 1\_sin\,\phi' \quad (1)$$

- **Active Pressure (K_a):** The minimum lateral pressure exerted when the wall moves away from the soil, mobilizing its shear strength. This state occurs when the wall is allowed to deflect sufficiently to develop a failure surface in the soil. For a vertical wall with horizontal backfill, Rankine's theory gives:

$$K_a = \frac{1 - sin\,\emptyset'}{1 + sin\emptyset'} \quad (2)$$

- **Passive Pressure (K_p):** The maximum lateral resistance the soil can provide when the wall is pushed into the soil. This state occurs when the wall moves toward the soil, compressing it and developing a failure surface. Rankine's theory gives:

$$K_p = \frac{1+sin\,\emptyset'}{1-sin\emptyset'} \quad (3)$$

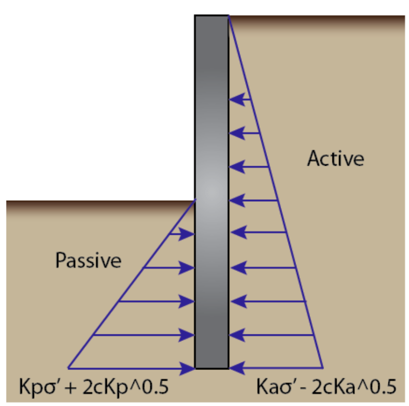

Figure 12: *Active and passive earth pressure distribution*. ***(Lateral Earth Pressure ReviewCivilPE, n.d.)***

II.5.2. INFLUENCE OF WATER PRESSURES AND SEEPAGE

For a quay wall, water pressures are often as significant as earth pressures. Hydrostatic pressure on the water side and differential water pressures due to tidal variations or seepage around the wall must be considered in design. Where seepage occurs, the differential water pressure is dissipated by vertical flow beneath the sheet pile wall. The distribution of unbalanced water pressure can be obtained from seepage analysis using methods such as: **(Terzaghi & Peck, 1967)**

- **Flow Nets:** Graphical representation of flow lines and equipotential lines, allowing determination of pore pressures and gradients.
- **Method of Fragments**: Analytical method that divides the flow domain into segments for which closed-form solutions are available.
- **Finite Element Analysis:** Numerical method that solves the seepage equations for complex geometries and heterogeneous soil conditions.

Upward seepage in front of the wall reduces the effective weight of the soil, thus reducing its ability to offer lateral support and potentially leading to piping or liquefaction. The critical hydraulic gradient for piping is given by Terzaghi's formula:

$$icr = \frac{\gamma'}{\gamma w} \tag{4}$$

II.5.3. OVERLOAD LOADS

Additional vertical loads on the retained soil, such as from stored cargo, cranes, or traffic, increase the horizontal active pressure on the wall. These are typically considered as: **(USACE, 2022)**

- **Uniformly Distributed Overload:** For general area loads, the additional pressure is simply $P_q = K_a . q$, where q is the Overload intensity.
- **Strip Loads:** For loads of finite width parallel to the wall, elastic solutions are available to determine the lateral pressure distribution.
- **Line Loads and Point Loads:** For concentrated loads, Boussinesq or elastic solutions can be used to estimate the induced lateral pressures.

II.6. DESIGN METHODS

II.6.1. CLASSICAL LIMIT EQUILIBRIUM METHOD (BLUM)

The Blum method (Blum, 1931) is a simplified analytical approach for anchored sheet pile walls based on the free earth support assumption. It involves the following steps: **(Blum, 1931)**

1. Calculate Earth Pressures: Determine the active earth pressure distribution on the retained side and the passive earth pressure distribution on the dredge side using appropriate earth pressure coefficients. Include the effects of Overload and water pressures.

2. Assume Failure Mechanism: Assume that the wall rotates about the anchor point and that the passive resistance is fully mobilized along the embedded length. This is the free earth support assumption.

3. Determine Embedment Depth: Calculate the required embedment depth ***D*** by taking moments about the anchor point. The moment equation balances the overturning moment from active pressures against the resisting moment from passive pressures.

4. Calculate Anchor Force: Determine the anchor force ***A*** from horizontal force equilibrium, considering the active and passive forces with an appropriate factor of safety on the passive resistance.

5. Determine Maximum Bending Moment: Locate the point of zero shear along the wall by integrating the net pressure distribution. The maximum bending moment occurs at this point and is calculated by integrating the shear diagram.

6. Apply Factor of Safety: Apply a factor of safety to the passive pressure (typically 1.5 to 2.0) to obtain the design embedment depth. Alternatively, factors of safety can be applied to the soil strength parameters.

For cantilever walls, a similar approach is used, but the wall is assumed to rotate about a point near the toe. The moment equation is written about the toe, and the embedment depth is found by solving for the point where the net moment is zero.

The Blum method has the advantage of simplicity and is widely used for preliminary design. However, it has several limitations: **(Mattos & Garcia, 2025)**

1. It assumes rigid body behavior and does not account for wall flexibility.
2. It uses simplified pressure distributions that may not reflect actual soil behavior.
3. It does not provide information about wall deflections.
4. It may be unconservative for certain soil conditions or loading scenarios.

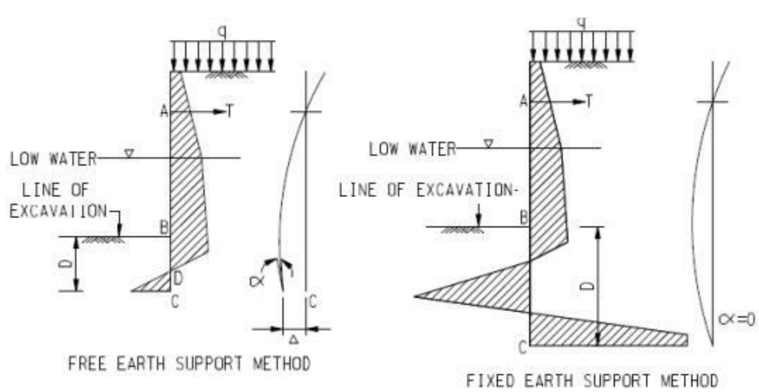

Figure 13: *Principle of the Blum method (free earth support).* ***(Free-Earth-Support-Method-and-Fixed-Earth-Support-Method.JPG, n.d.)***

II.6.2. NUMERICAL MODELING (FINITE ELEMENT METHOD)

Finite element method (FEM) using software like **PLAXIS 2D** are the most advanced approach for sheet pile wall design. The soil and wall are discretized into finite elements, allowing for: **(Seequent, 2025)**

- **Complex Stratigraphy:** Modeling of multiple soil layers with different properties.
- **Non-Linear Soil Behavior:** Use of advanced constitutive models such as Mohr-Coulomb, Hardening Soil, or Soft Soil Creep to represent the non-linear stress-strain behavior of soils.
- **Staged Construction:** Simulation of construction sequences including pile driving, anchor installation, and dredging.
- **Realistic Soil-Structure Interaction:** Automatic generation of active and passive zones based on wall displacements.
- **Deformation Analysis:** Calculation of wall deflections, settlements, and ground movements.
- **Global Stability Assessment:** Use of strength reduction techniques to evaluate safety factors against overall failure.

II.6.3. SPECIALIZED SOFTWARE FOR SHEET PILE ANALYSIS

Several specialized software packages are available for sheet pile wall design:

PROSHEET (ArcelorMittal): Dedicated software for sheet pile wall design based on classical methods, including the Blum method and Rowe's moment reduction. It incorporates a database of sheet pile profiles from major manufacturers and allows rapid preliminary design and optimization. **(ArcelorMittal, 2020)**

SPW (Sheet Pile Wall): Developed at Delft University, this software implements the subgrade reaction method with non-linear soil springs (P-Y curves). It allows analysis of both cantilever and anchored walls with multiple construction stages and provides detailed output of displacements, bending moments, and shear forces. **(Verruijt, 2006)**

GEO5: A comprehensive geotechnical software suite that includes modules for sheet pile wall design using both limit equilibrium and finite element methods. It offers integrated analysis of earth pressures, stability, and structural verification, with user-friendly interfaces and extensive material databases.

PLAXIS 2D: A finite element software suite for advanced geotechnical analysis of deformation, stability, and groundwater flow. It applies continuum mechanics with multiple constitutive models (e.g., Hardening Soil, Soft Soil Creep) to simulate complex soil-structure interaction for excavations, embankments, tunnels, and foundations. The software includes fully coupled consolidation, dynamic analysis, and staged construction, providing detailed output of displacements, stresses, and pore pressures. **(Seequent, 2025)**

II.7. STABILITY ANALYSIS

II.7.1. ROTATIONAL STABILITY

The primary geotechnical requirement is that the embedded length of the wall is sufficient to develop the passive resistance needed to prevent rotational failure around the anchor point (for anchored walls) or the toe (for cantilever walls). This is typically verified using limit equilibrium methods with appropriate factors of safety applied to the passive pressure or to the soil strength parameters. **(USACE, 2022)**

The factor of safety against rotational failure can be expressed as:

$$\boldsymbol{F = \frac{M_{resisting}}{M_{driving}}} \tag{5}$$

where $\boldsymbol{M_{resisting}}$ is the moment from passive pressures and $\boldsymbol{M_{driving}}$ is the moment from active pressures and other destabilizing forces.

II.7.2. STRUCTURAL RESISTANCE

The maximum bending moment $\boldsymbol{M_{max}}$ calculated from the analysis (under ULS combinations) is used to select a sheet pile profile. The required elastic section modulus $\boldsymbol{S_{req}}$ is given by: **(Eurocode, 2004)**

$$\boldsymbol{S_{req} = \frac{M_{max}}{f_y/\gamma_{M0}}} \tag{6}$$

where $\boldsymbol{f_y}$ is the steel yield strength and $\boldsymbol{\gamma_{M0}}$ is a partial factor for the material (typically 1.1 for steel according to Eurocode 3).

The selected section must also satisfy shear resistance requirements and, for anchored walls, must have adequate strength at the anchor connection points.

II.7.3. ANCHOR FORCE CALCULATION

For anchored walls, the anchor system (tie rods, wales, and anchor wall) must be designed to safely carry the calculated anchor force. The tie rod force is given by: **(USACE, 2022)**

$$\boldsymbol{T_{rod} = A.s} \tag{7}$$

where $\boldsymbol{A}$ is the anchor force per unit length of wall and **s** is the tie rod spacing.

The required net area of the tie rod is:

$$\boldsymbol{A_{net} = \frac{T_{rod}}{f_t}} \tag{8}$$

where f_t is the allowable tensile stress (typically $0.4f_y$ for steel according to AISC specifications).

II.7.4. OVERALL STABILITY

For anchored walls, it is necessary to check the overall stability of the wall-soil-anchor system, ensuring that a deep-seated rotational slip failure does not occur through the anchor block and below the sheet pile tip. This is typically assessed using slope stability analysis methods (exp: Bishop) with appropriate factors of safety. **(Clayton et al., 2014)**

The analysis must consider:

- The geometry of the wall and anchor system
- Soil strength parameters along potential failure surfaces.
- The effect of the anchor force on the stability of the soil mass.
- Pore water pressures and seepage conditions.

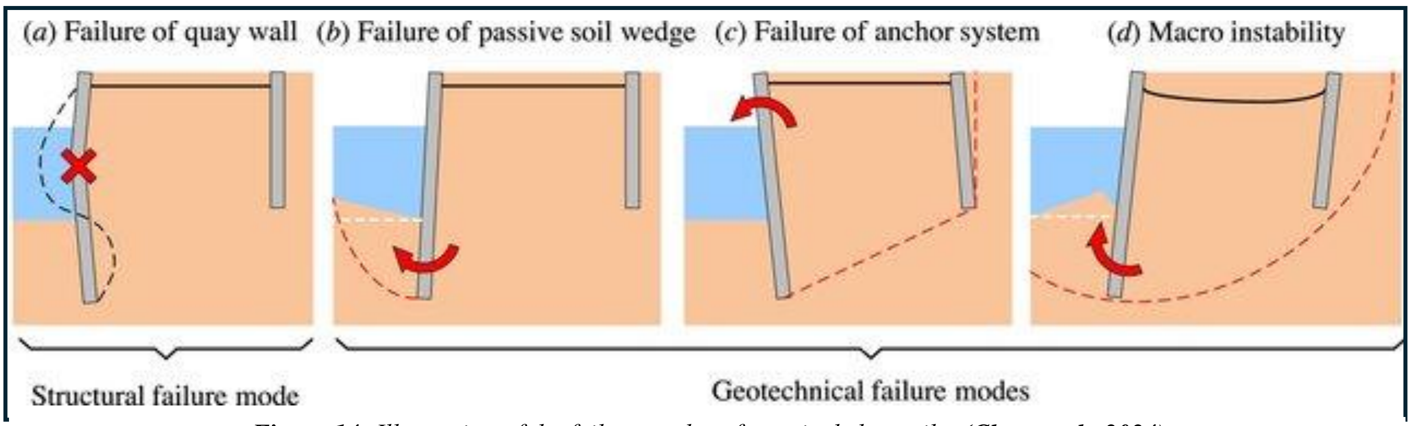

Figure 14: *Illustration of the failure modes of a typical sheet pile.* ***(Chen et al., 2024)***

II.8. CODES AND STANDARDS

II.8.1. EUROCODE 7 (NF EN 1997)

Eurocode 7 provides the framework for geotechnical design in Europe. It defines: **(Eurocode, 2004)**

Limit States: Ultimate Limit States (ULS) for stability and strength, and Serviceability Limit States (SLS) for deformations and displacements.

Design Approaches: Three different approaches with varying combinations of partial factors applied to actions (F), soil parameters (M), and resistances (R). For sheet pile walls, Design Approach 1 is commonly used, with two combinations:

- **Combination 1:** Factors applied to actions
- **Combination 2:** Factors applied to soil strength parameters

Partial Factors: Typical values for ULS GEO are:

- **Permanent actions:** $\gamma G = 1.35$
- **Variable actions:** $\gamma Q = 1.5$
- **Soil friction angle:** $\gamma \phi' = 1.25$ (for Design Approach 1, Combination 2)

Verifications: For sheet pile walls, Eurocode 7 requires verification of:

- Rotational failure (GEO)
- Structural failure (STR)
- Anchor failure
- Hydraulic heave and piping (HYD)

Table 4: *Partial factors for ULS GEO (Eurocode 7)*

Design Approach	***Permanent Actions***	***Variable Actions***	***Soil Parameters (tan $\emptyset'$)***
DA1-C1	*1.35*	*1.5*	*1.0*
DA1-C2	*1.0*	*1.3*	*1.25*
DA2	*1.35*	*1.5*	*1.0*
DA3	*1.0*	*1.3*	*1.25*

II.8.2. FRENCH STANDARDS (NF P94-282)

The French standard NF P94-282 provides specific rules for the design of embedded retaining walls, complementing Eurocode 7. It details: **(AFNOR, 2013)**

- Methods for calculating earth pressures, including the effects of wall friction and adhesion.
- Procedures for determining subgrade reaction coefficients for use in Winkler models.
- Verification requirements for both temporary and permanent works.
- Guidelines for considering construction sequences and their effects on wall behavior.

II.8.3. INTERNATIONAL STANDARDS (USACE, EAU)

USACE (USA): The U.S. Army Corps of Engineers provides extensive guidance on the design of sheet pile walls in its engineering manuals, particularly EM 1110-2-2504. The manual covers: **(USACE, 2022)**

- Geotechnical investigation requirements
- Earth pressure theories and calculations
- Design procedures for cantilever and anchored walls
- Structural design of sheet piles and anchor systems
- Construction considerations and quality control

EAU (Germany): The "Recommendations of the Committee for Waterfront Structures, Harbours and Waterways" (EAU, 2012) is a highly regarded set of guidelines widely used in Europe and internationally for port structures. It provides: **(EAU, 2012)**

- Design rules based on limit state principles
- Recommendations for earth pressure coefficients considering wall friction
- Guidelines for anchor design and verification
- Special provisions for marine environments

II.8.4. ALGERIAN SPECIFIC REGULATIONS

RPA 99/2003: The Algerian seismic code classifies Algiers in Zone IIa (moderate seismicity). Key requirements include: **(Ministère de l'Habitat et de l'Urbanisme, 2003)**

- **Acceleration coefficient:** a=0.15g to 0.25g
- **Importance factor for port structures:** 1.25
- **Behavior coefficient for sheet pile walls:** 1.5

Requirements for considering dynamic earth pressures using the Mononobe Okabe method.

DTR BC 2.33.2: This document provides guidance on deep foundation calculations, which can be applied to anchor piles or walls. It includes: **(Ministère de l'Habitat et de l'Urbanisme, 2003)**

- Methods for determining axial and lateral capacity of piles
- Group effect considerations
- Settlement analysis procedures
- Construction control requirements

II.9. CONCLUSION

This chapter has provided a comprehensive overview of sheet pile walls, including types, design methods, and relevant codes and standards. The classical limit equilibrium method (Blum) remains a valuable tool for preliminary design, while numerical methods (FEM) offer more realistic simulations for final design. The choice of method depends on the project requirements, available data, and desired level of accuracy.

The theoretical foundations presented in this chapter will be applied to the specific case of the Algiers Bay Marina project in the following chapter.

CHAPTER III: DESCRIPTION OF THE MARINA BAY OF ALGIERS PROJECT

III.1. INTRODUCTION

This chapter presents the context of the Algiers Bay Marina project, its geographical location, and the characteristics of the sheet pile quay wall that is the subject of this study. The information presented is based on the technical documentation provided by BTP Infraestructuras (2018) for the Marina Bay of Algiers development.

III.2. PRESENTATION OF THE ALGIERS BAY MARINA PROJECT

III.2.1. PROJECT CONTEXT

The Algiers Bay Marina project is part of the development of Algerian port infrastructure, aimed at modernizing and expanding the reception capacity for pleasure craft in the Algiers region. This strategic project responds to the growing demands of nautical tourism in the Mediterranean and the need to diversify the national economy **(Ministry of Public Works and Transport, 2018)**

The marina is designed to accommodate vessels of various sizes, with optimized berthing capacity for pleasure craft up to 35 meters in length. The project includes:

- A protected basin with approximately 800 m of quay length
- A slipway for boat launching and recovery
- Marginal quays along the basin perimeter
- Associated infrastructure including utilities, access roads, and service buildings

The geographical location of the project takes advantage of the natural protection of the Bay of Algiers, while requiring additional protection structures to ensure the safety of vessels. **(BTP INFRAESTRUCTURAS, 2018)**

III.2.2. GEOGRAPHICAL LOCATION OF THE STUDY AREA

Administrative and Urban Location:

The study site is located in the municipality of Mohammadia, in the eastern part of Algiers. It is positioned between two major urban landmarks: **(BTP INFRAESTRUCTURAS, 2018))**

- **East:** Hilton Hotel
- **West:** Ardis shopping center

Geographical Boundaries:

The study area has the following natural and artificial boundaries:

- **North:** Mediterranean Sea
- **South:** National Road No. 05
- **East:** Lido military camp
- **West:** Oued El-Harrach, near its mouth (distance: 1,500 m)

Coordinates:

The site is referenced according to the UTM WGS84 zone 31N coordinate system:

- **Latitude:** between 36°44'30"N and 36°44'43.94"N
- **Longitude:** between 3°12'20"E and 3°12'40"E

Figure 15: *Satellite maps showing the geographical location of Marina Bay in the Bay of Algiers (Marina Rapport July 2020)*

III.2.3. CHARACTERISTICS OF THE COASTLINE

The Marina Bay develops a coastal length of approximately 800 meters along the Bay of Algiers. The coastline in this area is characterized by:

- Sandy beaches with occasional rocky outcrops
- Moderate wave exposure, with dominant waves from the northwest
- Tidal range of approximately 0.4 m (microtidal)
- Presence of the Oued El-Harrach river mouth to the west

The bathymetry shows a relatively gentle slope, with water depths reaching -5 to -6 m at a distance of about 200 m from the shoreline. **(BTP INFRAESTRUCTURAS, 2018)**

III.3. DESCRIPTION OF THE SHEET PILE QUAY WALL

The sheet pile quay system adopted for the Algiers Bay Marina comprises three typical sections, as detailed in the design documentation. **(BTP INFRAESTRUCTURAS, 2018)**

III.3.1. GEOMETRIC CONFIGURATION

Quay Types:

- **Quay C1 (Slipway):** Located at the slipway area, with water depth **2.50 m** and operational Overload **15 kN/m²**. This section has the highest Overload due to boat handling operations.
- **Quay G1 (Marginal quay, first section):** The first section of the marginal quay, with water depth **2.50 m** and operational Overload **10 kN/m².**
- **Quay G2 (Marginal quay, second section):** The second section of the marginal quay, with water depth **3.00 m** and operational Overload **10 kN/m².**

Table 5: *Geometric characteristics of sheet pile quay sections (after BTP Infraestructuras, 2018)*

Parameter	***Quay C1***	***Quay G1***	***Quay G2***
Water depth (m)	*2.50*	*2.50*	*3.00*
Overload (kN/m²)	*15*	*10*	*10*
Sheet pile profile	*GU 6N*	*GU 6N*	*GU 6N*

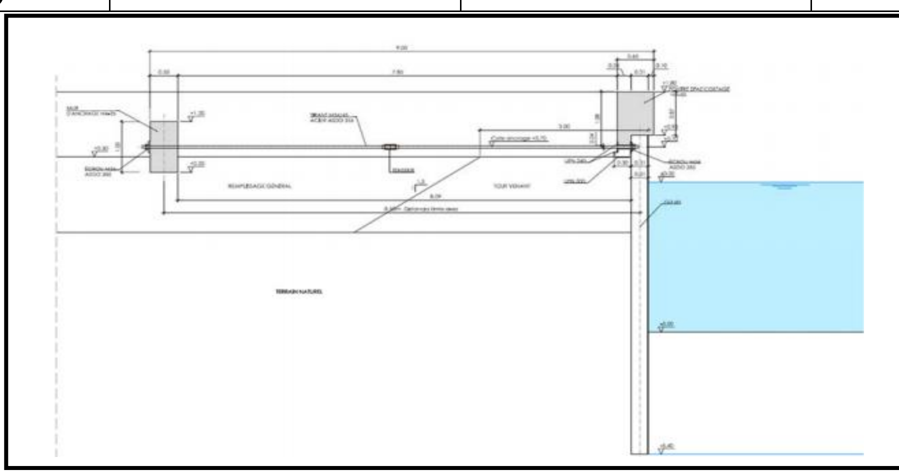

Figure 16: *Typical cross-section of sheet pile quay wall for Quay C1 (after BTP infraestructuras, 2018)*

Anchorage System:

All three sections share the same anchorage system:

- **Tie rods:** M56/45 bars in ASDO 355 steel
- **Tie rod length:** approximately 8.80 m
- **Tie rod spacing:** 3.60 m
- **Anchor wall:** Reinforced concrete HA-35, height 1.0 m, width 0.5 m, founded at elevation +0.20 m

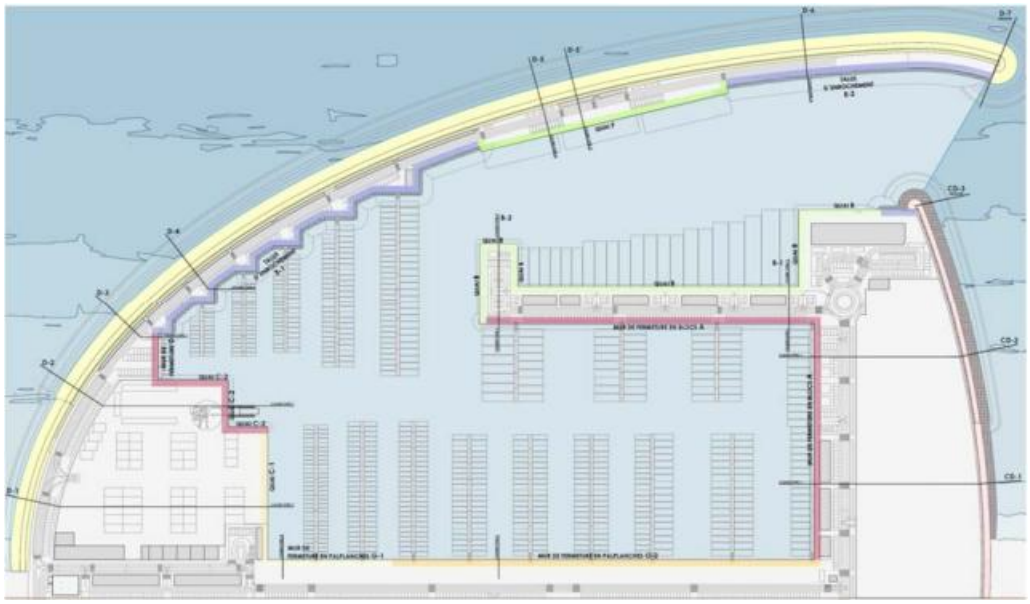

Figure 17: *Plan view showing sheet pile zones with orange and yellow (after BTP Infraestructuras, 2018)*

III.3.2. SHEET PILE PROFILE AND MATERIAL

The **GU 6N** sheet pile is a hot-rolled Z-profile manufactured by ArcelorMittal. Its geometric properties are: **(ArcelorMittal, 2020; BTP INFRAESTRUCTURAS, 2018)**

- Section height: **309 mm**
- Web thickness: **6 mm**
- Flange width: **448.5 mm**
- Sectional area: **120.1 cm²** per meter of wall
- Section modulus: **1245 cm³** per meter of wall
- Moment of inertia: **45,003** cm⁴ per meter of wall

The steel grade is **S240 GP**, with the following mechanical properties:

- Yield strength: $\boldsymbol{fy = 240\ MPa}$
- Ultimate tensile strength: $\boldsymbol{fu = 360\ MPa}$
- Modulus of elasticity: $\boldsymbol{E = 210\ GPa}$

III.3.3. ANCHORAGE SYSTEM

Tie Rods:

The tie rods are M56/45 bars, meaning they have a nominal diameter of 56 mm at the threaded ends and a shaft diameter of 45 mm. They are manufactured from ASDO 355 steel, with properties: **(BTP INFRAESTRUCTURAS, 2018)**

- Yield strength : $\boldsymbol{fy = 350\ MPa}$
- Ultimate tensile strength: $\boldsymbol{fu = 510\ MPa}$
- Tensile capacity: $\boldsymbol{Ft, Rd = 497\ kN}$

Wales:

The wales are composed of two UPN 240 profiles in S355 JR steel, acting as continuous beams to transfer tie rod forces to the sheet piles. The UPN 240 profile has: **(BTP INFRAESTRUCTURAS, 2018)**

- Section height: 240 mm
- Flange width**:** 85 mm
- Web thickness: 9.5 mm
- Section modulus: 300 cm^3

Anchor Wall:

The anchor wall is a continuous reinforced concrete wall (HA-35) with rectangular cross-section: **(BTP INFRAESTRUCTURAS, 2018)**

- Height: 1.0 m
- Width: 0.5 m
- Founding elevation: +0.20 m
- Concrete blinding: 10 cm HM-15

III.3.4. CAPPING BEAM

The capping beam is a reinforced concrete element cast on top of the sheet piles to provide a continuous finished edge and to distribute loads. Its characteristics are: **(BTP INFRAESTRUCTURAS, 2018)**

- Width: 0.65 m
- Overhang toward basin: 0.10 m
- Capping elevation: +1.80 m
- Concrete grade: HA-35

III.4. SEISMICITY OF THE ALGIERS REGION

Algeria is located in a region of moderate to high seismic activity due to the convergence of the African and Eurasian tectonic plates. According to the Algerian Seismic Code **RPA 99/2003** (version 2003), the region of Algiers is classified in Zone IIa, characterized by moderate seismicity. **(Ministère de l'Habitat et de l'Urbanisme, 2003)**

Table 6: *Seismic parameters for Algiers region (RPA 99/2003)*

Parameter	***Value***
Seismic Zone	*Ila*
Acceleration Coefficient (A)	*0.15-0.25 g*
Site Class (assumed)	*S2(firm soil)*
Importance Factor (for port structures)	*1.25*
Behavior Coefficient (for sheet pile walls)	*1.5*

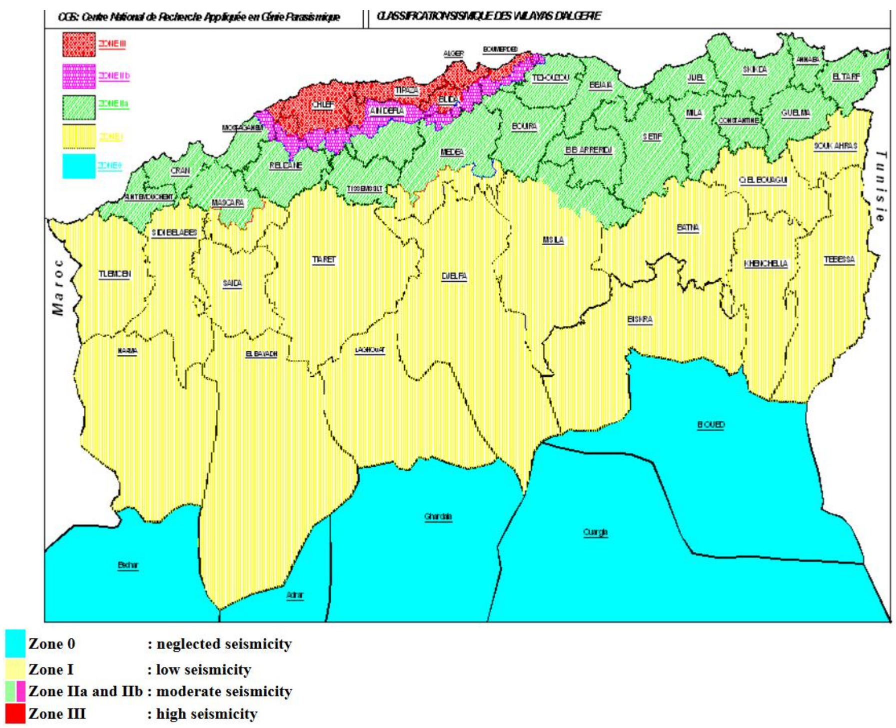

Figure 18: *Seismic zoning map of Algeria (RPA 99/2003)*

III.4.1. SEISMIC DESIGN CONSIDERATIONS

The seismic design of the sheet pile quay wall must account for: **(Ministère de l'Habitat et de l'Urbanisme, 2003; USACE, 2022)**

Inertial Forces: The mass of the structure itself, including the sheet piles, capping beam, and any attached elements, generates inertial forces during an earthquake. These forces are calculated based on the acceleration coefficient and the importance factor.

Dynamic Earth Pressures: During an earthquake, the earth pressures on the wall increase significantly. The Mononobe-Okabe method is commonly used to calculate seismic active and passive pressure coefficients:

$$\boldsymbol{Kae} = \frac{\boldsymbol{cos\ 2(\phi' - \theta - \psi)}}{\boldsymbol{cos\ \psi cos^2\theta cos\ (\delta + \theta + \psi)[1 + \sqrt{\frac{sin\ (\phi' + \delta) sin\ (\phi' - \beta - \psi)}{cos\ (\delta + \theta + \psi) cos\ (\theta - \beta)}}]^2}} \quad (9)$$

where $\boldsymbol{\theta = tan^{-1}(K_h/(1 \pm K_v))}$, K_h is the horizontal seismic coefficient, and K_y is the vertical seismic coefficient.

Liquefaction Potential: Loose saturated sands are susceptible to liquefaction during earthquakes, which could lead to loss of soil strength and wall instability. The potential for liquefaction must be assessed based on the soil properties and seismic loading.

Hydrodynamic Pressures: Water pressures on the wall increase during earthquakes due to the acceleration of the water mass. These pressures must be considered in the design, particularly for the submerged portion of the wall.

Permanent Displacements: Flexible structures like sheet pile walls may experience permanent displacements during strong earthquakes. These displacements must be estimated and checked against allowable limits.

III.5. GEOTECHNICAL CHARACTERIZATION

III.5.1. SITE STRATIGRAPHY

Figure 19: *Borehole and CPT tests (after BTP Infraestructuras, 2018)*

The site stratigraphy, as revealed by borehole SC-05, consists of a relatively homogeneous layer of fine to medium sands, slightly silty, characteristic of Mediterranean littoral formations. The sand exhibits variable consistency from medium to dense with depth. No distinct clay layers or organic deposits were encountered within the depth investigated. **(BTP INFRAESTRUCTURAS, 2018)**

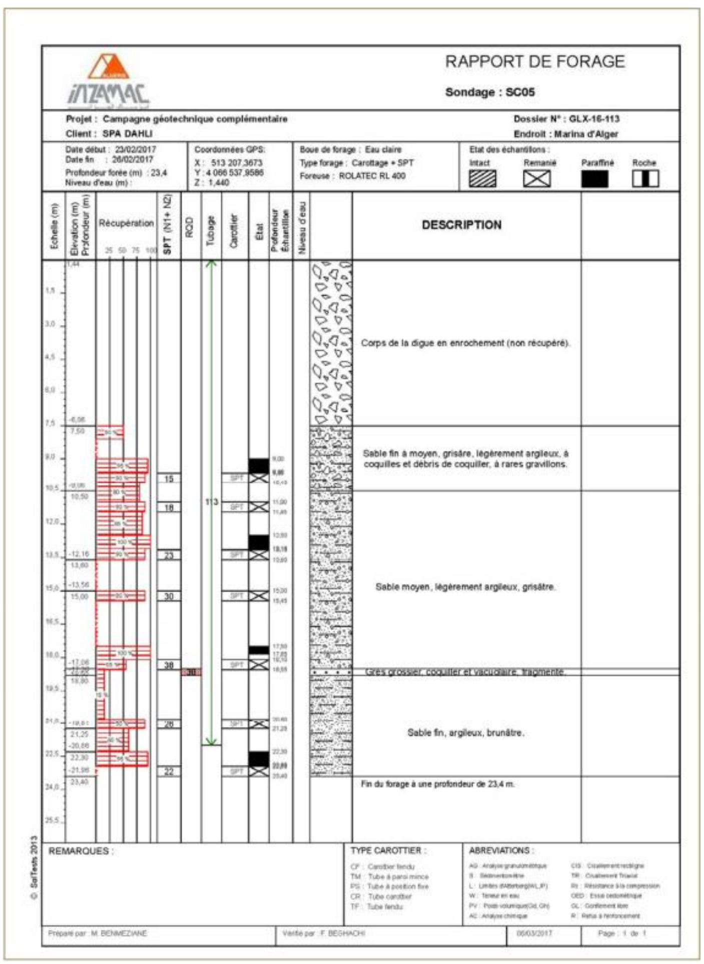

Figure 20: Description of borehole SC05 (after BTP Infraestructuras, 2018)

III.5.2. ANALYSIS OF SPT TESTS

III.5.2.1. REULTS OF SPT TESTS IN BOREHOLE SC05

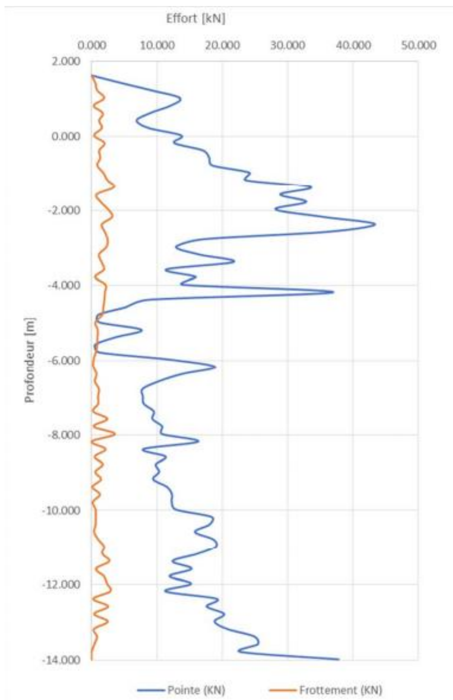

Figure 21: *CPT 01 Test. (after BTP Infraestructuras, 2018)*

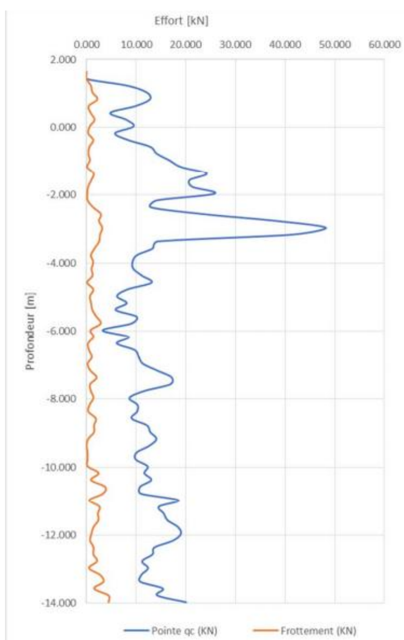

Figure 22: *CPT 02 Test. (after BTP Infraestructuras, 2018)*

The SPT results from borehole SC05 are summarized in Table 7 The depths are measured from the working platform elevation.

Table 7: *SPT test results in borehole SC05 (after BTP Infraestructuras, 2018)*

SONDEO SC 05						
h(m)	Clasificación	σ´vo(t/m2) SPT	SPT	SPT norm(N1)60	Dr	ϕ°
9.69(-8.24)	arena	10.688	15	15.51	61	37.16
11(-9.89)	arena	12.338	18	19.99	64	37.99
13.19(-11.75)	arena	14.198	23	27.41	69	39.4
15(-13.56)	arena	16.008	30	37.96	75	41.13
18.1(-16.66)	arena	19.108	38	52.53	80	42.14
20.80(-19.36)	arena	21.808	26	38.40	63	37.53
22.60(-21.16)	arena	23.608	22	33.80	56	35.4
MEDIA						38.68

Average: 38.7°

III.5.2.2. INTERPRETATION OF FRICTION ANGLE (SCHMERTMANN FORMULA)

The internal friction angle φ' was estimated using the Schmertmann formula as recommended by ROM 0.5-05: **(BTP INFRAESTRUCTURAS, 2018)**

$$\tan \phi' = \left(\frac{N_{SPT}}{\frac{N_{SPT}}{\sigma'_{v0}} + 20.3\,\frac{\sigma'_{v0}}{P_a}} \right)^{0.34} \tag{10}$$

where:
- N_{SPT} = measured blow count
- σ'_{v0} = effective vertical stress at test depth (t/m²)
- P_a = reference pressure = 100 kPa

The computed friction angles range from 35.4° to 42.1°, with an average value of 38.7°. The lower values correspond to shallower depths, indicating a slight increase in density with depth. The one anomalously low value at 9.69 m depth (16.1°) is likely due to disturbance or local variability and is not considered representative of the overall soil behavior.

III.5.3. ANALYSIS OF CPT RESULTS

III.5.3.1. CPT-1 TEST RESULTS

Table 8: *CPT-1 test results (after BTP Infraestructuras, 2018)*

CPT-1				
COTA	σ_{Vo}(t/m2)	σ'_{vo}(t/m2)	q_p(t/m2)	φ´
1.626	0.00	0.00		
-6.374	15.51	9.14	1700.00	43.86
-7.374	17.51	10.14	700.00	38.47
-8.374	19.51	11.14	1600.00	42.54
-9.374	21.51	12.14	900.00	38.39
-10.374	23.51	13.14	1800.00	42.30
-11.374	25.51	14.14	1600.00	41.28
-12.374	27.51	15.14	1100.00	38.77
-13.374	29.51	16.14	2100.00	42.02
-14.374	31.51	17.14	3800.00	44.71
MEDIA				41.37

Average: 41.37°

III.5.3.2. CPT-2 TEST RESULTS

Table 9: *CPT-2 test results (after BTP Infraestructuras, 2018)*

CPT-2				
COTA	σ_{Vo}(t/m2)	σ'_{vo}(t/m2)	q_p(t/m2)	φ´
1.633	0.00	0.00		
-6.367	15.51	9.14	800.00	39.85
-7.367	17.51	10.14	1400.00	42.34
-8.367	19.51	11.14	1000.00	39.99
-9.367	21.51	12.14	1400.00	41.38
-10.367	23.51	13.14	1200.00	40.09
-11.367	25.51	14.14	1400.00	40.54
-12.367	27.51	15.14	1700.00	41.23
-13.367	29.51	16.14	1100.00	38.39
-14.367	31.50	17.14	2000.00	41.44
MEDIA				40.58

Average: 40.58

III.5.3.3. INTERPRETATION OF FRICTION ANGLE FROM CPT (ROBERTSON FORMULA)

The friction angle from CPT data was estimated using the Robertson formula as recommended by ROM 0.5-05: **(BTP INFRAESTRUCTURAS, 2018)**

$$\tan \phi' = 0.10 + 0.38\log_{10}\left(\frac{q_c - \sigma_{v0}}{\sigma'_{v0}}\right) \quad (11)$$

The computed friction angles average 41.37° for CPT-1 and 40.58° for CPT-2, slightly higher than the SPT-derived values. This difference is typical and reflects the different testing mechanisms and interpretation methods.

III.5.4. RELATIVE DENSITY (MEYERHOFF FORMULA)

The relative density of the sand was estimated using the Meyerhoff formula:

$$D_r = \sqrt{\frac{N_{SPT}}{23\sigma'_{v0}+16}} \quad (12)$$

where N_{SPT} is the uncorrected blow count and σ'_{v0} is in kp/cm². The results indicate a medium to dense sand, with relative density increasing from about 15% at shallow depths to over 50% at depth. This increase in density with depth is typical of normally consolidated granular deposits.**(BTP INFRAESTRUCTURAS, 2018)**

III.5.5. SYNTHESIS AND RECOMMENDED DESIGN VALUES

The three sets of friction angle estimates (SPT, CPT-1, CPT-2) yield an overall average of approximately 40.2°. However, to maintain consistency with the original project assumptions and to provide a conservative design that accounts for potential variability and degradation during construction, a representative value of $\phi' = 37.5°$ is adopted for all design calculations. This value is slightly lower than the calculated averages but provides an appropriate safety margin.**(BTP INFRAESTRUCTURAS, 2018)**

The unit weight of the sand is taken as:

- Above water table: $\gamma = 18$ **kN/m³**
- Below water table (saturated)**:** $\gamma_{sat} = 20$ **kN/m³**
- Submerged unit weight: $\gamma' = 10$ **kN/m³**

Table 10: *Summary of geotechnical parameters (after BTP Infraestructuras, 2018)*

Parameter	***Value***
Soil type	*Fine to medium sand, slightly silty*
Friction angle $\emptyset'$ (design)	*37.5 °*
Cohesion c'	*0 kPa*
Unit weight γ (above water)	*18 kN/m³*
Saturated unit weight γsat	*20 kN/m³*
Submerged unit weight γ'	*10 kN/m³*
Relative density	*Medium to dense*

III.6. GEOTECHNICAL MODELING

III.6.1. HORIZONTAL SUBGRADE REACTION MODULUS (TERZAGHI METHOD)

For the numerical modeling of soil-structure interaction, particularly in software utilizing the subgrade reaction method (such as SPW and some modules of GEO5), the horizontal subgrade reaction modulus K_h is required. This parameter was calculated using Terzaghi's (1955) formulation: **(Terzaghi, 1955; Verruijt, 2006)**

$$\boldsymbol{K_h = 0.75 \cdot \frac{E_0}{D}} \tag{13}$$

where:
E_0 = deformation modulus
D = equivalent width of the sheet pile (taken as 0.309 m for GU 6N profile)

For submerged granular soil, the deformation modulus is given by:

$$\boldsymbol{E_0 = 0.6 \times 1.60 \times Z \text{ (in cm)}} \tag{14}$$

where $\boldsymbol{Z}$ is the depth in cm. This empirical relationship accounts for the increase in soil stiffness with depth.

The calculated values of K_h for the sheet pile wall are presented in Table 11

Table 11: *Values of horizontal subgrade reaction modulus for sheet pile wall.(after BTP Infraestructuras, 2018)*

Depth Z (m)	**H (m)**	**E0 (kN/m^2)**	**Kh (kN/m^3)**
0.5	0	0	0
-0.5	1	9600	23300
-1.5	2	19200	46600
-2.5	3	28800	69900
-3.5	4	38400	93200
-4.5	5	48000	116500
-5.5	6	57600	139800
-6.5	7	67200	163100
-7.5	8	76800	186400
-8.5	9	86400	209700
-9.5	10	96000	233000
-10.5	11	105600	256300
-11.5	12	115200	279600
-12.5	13	124800	302900

III.6.2. MATERIALS

The materials used in the structure are:

- **Sheet piles:** S240 GP steel ($f_y = 240$ MPa, $E = 210$ GPa)
- **Tie rods:** ASDO 355 steel ($f_y = 350$ MPa, $f_u = 510$ MPa)
- **Wales:** UPN 240 profiles in S355 JR steel
- **Anchor wall and capping beam:** Reinforced concrete HA-35 ($f_{ck} = 35$ MPa)

- **Reinforcing steel:** B500S ($f_y = 500$ MPa)

III.7. CONCLUSION

This chapter has presented the context of the Algiers Bay Marina project and the characteristics of the sheet pile quay wall that will be analyzed in this study. Three typical sections (C1, G1, G2) have been identified, each with specific water depths and Overload loads. The anchorage system is common to all sections, consisting of tie rods, wales, and a continuous anchor wall.

The seismicity of the Algiers region requires that seismic effects be considered in the design, including dynamic earth pressures and potential liquefaction. The theoretical foundations established in the previous chapters, combined with the project-specific information presented here, provide the basis for the detailed design and analysis that would follow in a complete study.

Part 2: Pratical Study

CHAPTER IV: PREDIMENSIONING OF THE SHEET PILE WALL

IV.1. INTRODUCTION

Predimensioning is an essential preliminary step in the design of sheet pile walls. It allows rapid estimation of key design parameters embedment depth, anchor force, and maximum bending moment using simplified analytical methods. These preliminary results serve as a basis for selecting an initial sheet pile section and for defining input parameters for more detailed numerical analyses.

This chapter presents three traditional analytical methods for predimensioning anchored sheet pile walls: **(Bouafia, 2018a)**

1. **Simple Thrust Method** (Méthode des butée simples): also known as the free earth support method with a factor of safety applied to passive pressure.
2. **Sheet Pile Embedded Method** (Méthode de rideau encastrée): also known as the fixed earth support method, which assumes a point of contra-flexure.
3. **Equivalent Beam Method** (Méthode de les poutres équivalente): a simplified approach that divides the wall into two equivalent beams.

These methods are applied to the loaded section of the Marina Bay project:

Quay G2 (water depth **3.00 m**, Overload **10 kN/m²**). The results are then used to select a preliminary sheet pile profile.

$$\boldsymbol{\varphi = 37.5°, \quad H_{tot} = 3.9\, m, \quad H = 3\, m, \quad \gamma = 16\, kN/m^3, \quad \gamma w = 10\, kN/m^3, \quad {K_a}^q = K_a = 0.243}$$

IV.2. SIMPLE THRUST METHOD (METHODE DES BUTEE SIMPLES)

IV.2.1. PRINCIPLE AND ASSUMPTIONS

The simple thrust method, also known as the free earth support method (Blum, 1931), is based on the following assumptions: **(Bouafia, 2018a)**

- The sheet pile wall rotates as a rigid body about the anchor point (point $\boldsymbol{T}$ in Figure 23). This is why it is called "free earth support" the toe is not fixed.
- The passive earth pressure is fully mobilized along the entire embedment depth.
- The active earth pressure is fully mobilized on the retained side.
- A factor of safety (typically $\boldsymbol{F_s = 2.0}$) is applied to the passive resistance to account for uncertainties.

  Remark: $F_s = 2$ *Taken from the RPA (Réglement Parasismique Algerien)*
- The anchor is considered as a simple support (no moment transfer).

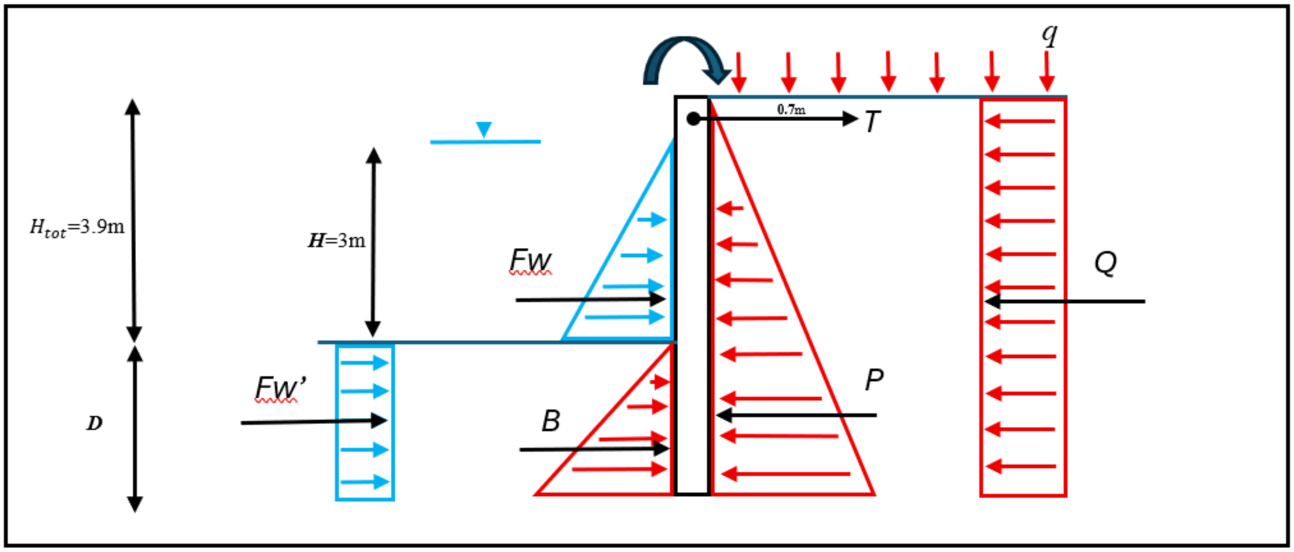

Figure 23:*Simple thrust method – forces and pressures distribution*

IV.2.2. CALCULATION PROCEDURE (WITHOUT A SAFETY FACTOR)

The calculation proceeds as follows:

Step 1: Earth pressure coefficients:

$$\boldsymbol{K_a = \frac{1 - sin37.5°}{1 + sin37.5°} = 0.243} \tag{15}$$

$$\boldsymbol{K_p = \frac{1 + sin37.5°}{1 - sin37.5°} = 4.12} \tag{16}$$

Step 2: Enter the thrust forces P and the resistance forces ***B*** according to sheet ***D***.

$$P = \frac{1}{2}.K_a.\gamma.(H_{tot} + D)^2 \tag{17}$$

$$B = \frac{1}{2}.K_p.\gamma.D^2 \tag{18}$$

$$Q = K_a{}^q.q(H_{tot} + D) \tag{19}$$

$$Fw = \frac{1}{2}.\gamma w.H^2 \tag{20}$$

$$Fw' = \gamma w.H.D \tag{21}$$

$$P = \frac{1}{2}.K_a.\gamma.(H_{tot} + D)^2 = \mathbf{1.944D^2 + 15.163D + 29.568}$$

$$B = \frac{1}{2}.K_p.\gamma.D^2 = \mathbf{32.96D^2}$$

$$Q = K_a{}^q.q(H_{tot} + D) = \mathbf{2.43D + 9.477}$$

$$Fw = \frac{1}{2}.\gamma w.H^2 = \mathbf{45\ kN/m}$$

$$Fw' = \gamma w.H.D = \mathbf{30D}$$

Step 3: Write that the sum of the bending moments about point ***T*** is zero, which leads to an equation in ***D*** to be solved.

$$\sum \boldsymbol{M/T = 0}$$

$$P\left[\frac{2}{3}(H_{tot} + D) - 0.2\right] + Q\left[\frac{H_{tot} + D}{2} - 0.2\right] + Fw\left[\frac{2}{3}H + 0.7\right] + Fw'\left[\frac{D}{2} + (H - 0.2)\right] - B\left[\frac{2}{3}D + (H_{tot} - 0.2)\right] = 0$$

(without a safety factor)

$\boldsymbol{Terme\ P}$: $1.296D^3 + 14.774D^2 + 56.103D + 70.963$

$\boldsymbol{Terme\ Q}$: $1.215D^2 + 8.991D + 16.585$

$\boldsymbol{Terme\ Fw}$: 121.5

$\boldsymbol{Terme\ Fw'}$: $15D^2 + 111D$

$\boldsymbol{Terme\ B}$: $-21.973D^3 - 121.952D^2$

$$20.677D^3 + 90.963D^2 - 176.094D - 209.048 = 0$$

$$\boxed{\textbf{D=2.07 m}}$$

(with a safety factor)

$\boldsymbol{Terme\ P}$: $1.296D^3 + 14.774D^2 + 56.103D + 70.963$

$\boldsymbol{Terme\ Q}$: $1.215D^2 + 8.991D + 16.585$

$\boldsymbol{Terme\ Fw}$: 121.5

$\boldsymbol{Terme\ Fw'}$: $15D^2 + 111D$

$\boldsymbol{Terme\ B}$: $-10.987D^3 - 60.976D^2$

$$9.6907D^3 + 29.987D^2 - 176.094D - 209.048 = 0$$

$$\boxed{\mathbf{D=3.60\ m}}$$

Step 4: Calculate the forces ***P*** and ***B***.

(without a safety factor)

$$P = 69.285\ kN/m$$

$$Q = 14.507\ kN/m$$

$$Fw = 45\ kN/m$$

$$Fw' = 62.1\ kN/m$$

$$B = 141.23\ kN/m$$

(with a safety factor)

$$P = 109.349\ kN/m$$

$$Q = 18.225\ kN/m$$

$$Fw = 45\ kN/m$$

$$Fw' = 108\ kN/m$$

$$B = 213.58\ kN/m$$

Step 5: Consider a safety factor ***Fs*** equal to 2 on the bearing force and deduce ***T*** from the equilibrium of horizontal forces :

(without a safety factor)

$$\sum F = 0$$

$$T = P + Q + Fw + Fw' - B$$

$$T = 69.285 + 14.507 + 45 + 62.1 - 141.23$$

$$\boxed{\boldsymbol{T = 49.67\ kN/m}}$$

(with a safety factor)

$$\sum F = 0$$

$$T = P + Q + Fw + Fw' - B$$

$$T = 109.349 + 18.225 + 45 + 108 - 213.58$$

$$\boxed{T = 67\ kN/m}$$

IV.3. SHEET PILE EMBEDDED METHOD (METHODE DE LA FICHE ENCASTREE)

IV.3.1. PRINCIPLE AND ASSUMPTIONS

The embedded method, also known as the fixed earth support method, assumes that the sheet pile is sufficiently embedded to develop a point of contra-flexure (zero moment) at some depth below the dredge line. This method was developed by Blum (1931) and later refined by Rowe (1952, 1955). The key assumptions are: **(Bouafia, 2018a)**

- The wall is assumed to be fixed at the toe, creating a clamped condition.
- A point of contra-flexure exists at a depth z_0 below the dredge line, where the bending moment is zero.
- The passive pressure below the contra-flexure point is simplified as a concentrated force.
- The embedment depth d is greater than that required by the free earth support

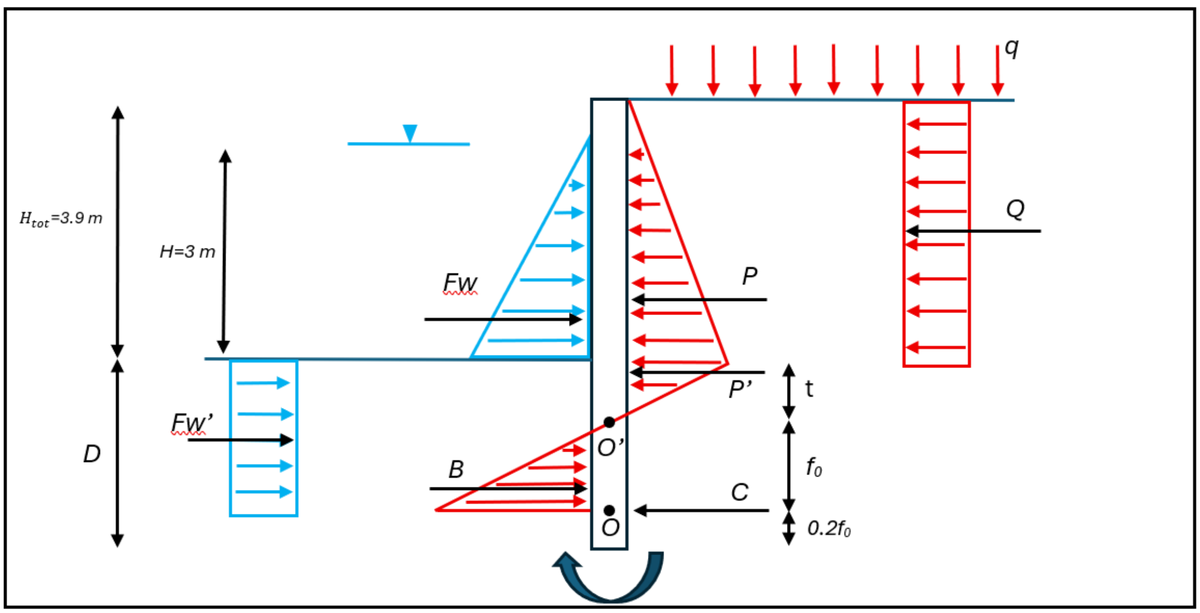

Figure 24: *Embedded method (fixed earth support) – pressure distribution*

IV.3.2. CALCULATION PROCEDURE

Step 1: Finding the depth t (the point of zero net pressure below the dredge line) in the Equivalent Beam Method:

$$K_a.\gamma(H_{tot} + t) + K_a.q = K_P.\gamma.t \tag{22}$$

$$\boldsymbol{t} = \frac{\boldsymbol{K_a(\gamma . H_{tot} + q)}}{\boldsymbol{\gamma(K_a - K_p)}}$$

$$\boxed{\boldsymbol{t = 0.284\ m}}$$

Step 2: Write the thrust forces ***P*** and the stop forces ***B*** as a function of sheet ***D*** (or $\boldsymbol{f_0}$),

$$P = \frac{1}{2}.Ka.\gamma.{H_{tot}}^2 = \boldsymbol{29.568\ kN/m}$$

$$P' = \frac{1}{2}.Ka.\gamma.H_{tot}.t = \boldsymbol{2.153\ kN/m}$$

$$B = \frac{1}{2}.Kp.\gamma.{f_0}^2 = \boldsymbol{32.96{f_0}^2}$$

$$Fw = \boldsymbol{45\ kN/m}$$

$$Fw' = 30(f_0 + t) = \boldsymbol{30f_0 + 8.52}$$

$$Q = {K_a}^q.q.H_{tot} = \boldsymbol{9.477\ kN/m}$$

Step 3: Write that the sum of the bending moments about the center of rotation ***O*** is zero, which leads to an equation in $\boldsymbol{f_0}$,

$$\sum \boldsymbol{M/O = 0}$$

$$\boldsymbol{B.\frac{1}{3}.f_0 + Fw'.\frac{1}{2}(f_0 + t) + Fw(1 + t + f_0) - P\left(\frac{H_{tot}}{3} + t + f_0\right) - P'\left(\frac{2}{3}t + f_0\right) - Q(\frac{H_{tot}}{2} + t + f_0) = 0}$$

Step 4: Calculate $\boldsymbol{f_0}$ and deduce the forces ***P*** and ***B***, considering a safety factor $\boldsymbol{F_s}$ equal to 2 on the passive resistance force.

(without a safety factor)

$\boldsymbol{Terme\ P}: -38.434 - 8.397 - 29.568f_0$

$\boldsymbol{Terme\ P'}: -0.408 - 2.153f_0$

$\boldsymbol{Terme\ Q}: -21.172 - 9.477f_0$

$\boldsymbol{Terme\ Fw}: 57.780 + 45f_0$

$\boldsymbol{Terme\ Fw'}: 15f_0{}^2 + 8.52f_0 + 1.21$

$\boldsymbol{Terme\ B}: 10.987f_0{}^3$

$$10.987f_0{}^3 + 15f_0{}^2 + 12.322f_0 - 9.421 = 0$$

$$\boldsymbol{f_0 = 0.445\ m}$$

(with a safety factor)

$\boldsymbol{Terme\ P}: -38.434 - 8.397 - 29.568f_0$

$\boldsymbol{Terme\ P'}: -0.408 - 2.153f_0$

$\boldsymbol{Terme\ Q}: -21.172 - 9.477f_0$

$\boldsymbol{Terme\ Fw}: 57.780 + 45f_0$

$\boldsymbol{Terme\ Fw'}: 7.5f_0{}^2 + 4.26f_0 + 0.605$

$\boldsymbol{Terme\ B}: 5.493f_0{}^3$

$$5.493f_0{}^3 + 7.5f_0{}^2 + 8.062f_0 - 10.026 = 0$$

$$\boxed{\boldsymbol{f_0 = 0.654\ m}}$$

Step 5: Derive the passive resistance ***C*** from the equilibrium of horizontal forces.

$$\sum F = 0$$

(without a safety factor)

$$C + P' + P + Q - Fw - Fw' - B = 0$$

$$C = B + Fw + Fw' - P' - P - Q$$

$$C = 6.527 + 45 + 21.87 - 2.153 - 29.568 - 9.477$$

$$\boxed{\boldsymbol{C = 32.2\ kN/m}}$$

(with a safety factor)

$$C + P' + P + Q - Fw - Fw' - B = 0$$

$$C = B + Fw + Fw' - P' - P - Q$$

$$C = 14.115 + 45 + 28.152 - 2.153 - 29.568 - 9.477$$

$$\boxed{\boldsymbol{C = 46.07\ kN/m}}$$

Step 6: Calculate the embedment depth $\boldsymbol{D}$ of the sheet pile wall

(without a safety factor)

$$D = t + 1.2f_0$$

$$D = 0.244 + 1.2(0.445)$$

$$\boxed{\boldsymbol{D = 0.82\ m}}$$

(with a safety factor)

$$D = t + 1.2f_0$$

$$D = 0.244 + 1.2(0.654)$$

$$\boxed{\boldsymbol{D = 1.07\ m}}$$

IV.4. EQUIVALENT BEAM METHOD (METHODE DE LA POUTRE EQUIVALENTE)

IV.4.1. PRINCIPLE AND ASSUMPTIONS

The equivalent beam method is based on the same assumptions as the " Sheet Pile Embedded Method " method. However, as shown in Figure 25, the problem involves three unknowns: the embedment depth $\boldsymbol{D}$ (or $\boldsymbol{f_0}$), the passive resistance force $\boldsymbol{C}$, and the anchor force $\boldsymbol{T}$. The static equilibrium equations are insufficient, and a third equation must be added in the form of an additional assumption. It is in fact assumed that the center of pressure $\boldsymbol{O'}$, located at a depth $\boldsymbol{t}$ below the excavation bottom, coincides with the point of zero moment, which allows the wall to be decomposed into two simply supported beams. **(Bouafia, 2018a)**

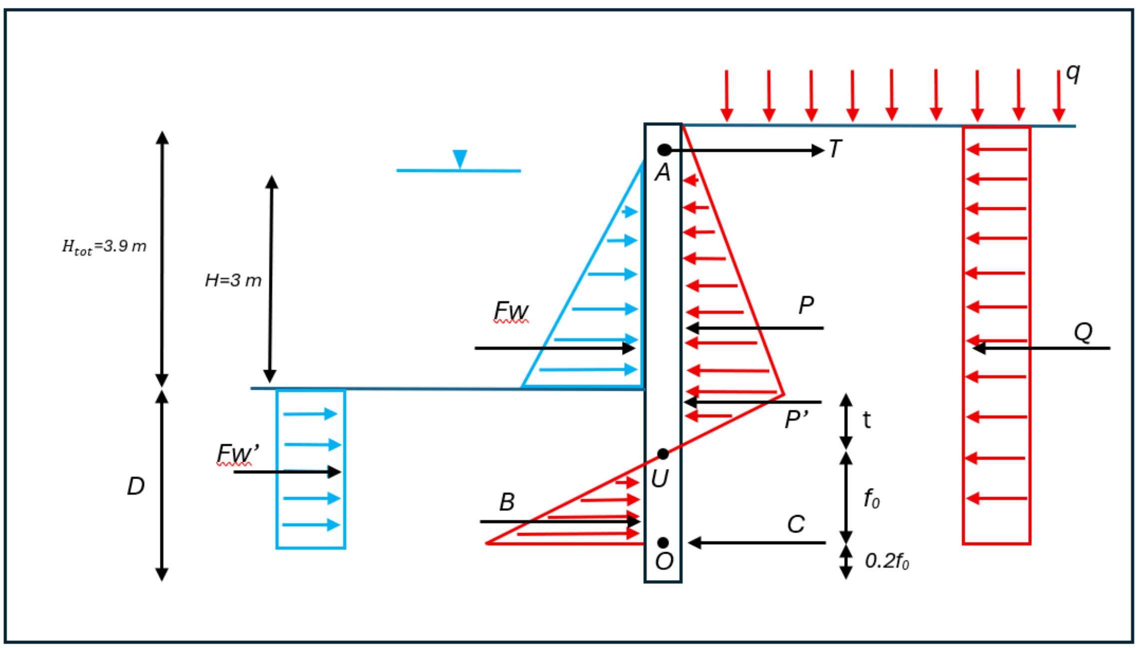

Figure 25: *Equivalent beam method – division into two equivalent beams*

IV.4.2. CALCULATION PROCEDURE

Step 1: Determine the depth $\boldsymbol{t}$ of point $\boldsymbol{U}$ by equating the active and passive earth pressures,

$$K_a.\gamma(H_{tot}+t)+K_a.q=K_P.\gamma.t$$

$$t=\frac{K_a(\gamma.H_{tot}+q)}{\gamma(K_a-K_p)}$$

$$\boxed{t=0.284\ m}$$

Step 2: Write the expression for the active thrust force $\boldsymbol{P}$ in the upper beam,

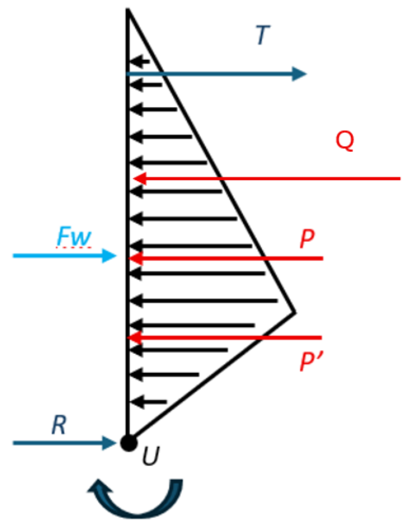

Figure 26: *Upper beam*

$$P=\frac{1}{2}.Ka.\gamma.H_{tot}{}^{2}=\mathbf{29.568\ kN/m}$$

$$P'=\frac{1}{2}.Ka.\gamma.H_{tot}.t=\mathbf{2.153\ kN/m}$$

$$Fw=\mathbf{45\ kN/m}$$

$$Q=K_a{}^{q}.q.H_{tot}=\mathbf{9.477\ kN/m}$$

$$R=\mathbf{0}$$

Step 3: Write that the sum of the bending moments about the center of pressure $\boldsymbol{U}$ is zero, which allows us to deduce the force $\boldsymbol{T}$ in the tie rod.

$$\sum M/U=0$$

$$T(H_{tot}+t-0.7)-P\left(\frac{H_{tot}}{3}+t\right)-P'\left(\frac{2}{3}t\right)-Q\left(\frac{H_{tot}}{2}+t\right)-Fw\left(\frac{H}{2}+t\right)=0$$

$$T(H_{tot}+t-0.7)=P\left(\frac{H_{tot}}{3}+t\right)+P'\left(\frac{2}{3}t\right)+Q\left(\frac{H_{tot}}{2}+t\right)+Fw\left(\frac{H}{2}+t\right)$$

$$T=\frac{148.696}{3.484}$$

$$\boxed{T=42.68\ kN/m}$$

Step 4: From the equilibrium of horizontal forces in the upper beam, deduce the reaction ***R*** at point ***U***.

$$\sum F=0$$

$$P+P'+Q+Fw-T-R=0$$

$$R=P+Q+P'+Fw-T$$

$$R=29.568+9.477+2.153+45-42.68$$

$$\boxed{R=43.518\ kN/m}$$

Step 5: Write that the sum of moments about point ***O*** in the lower beam is zero, which makes it possible to deduce f_0 and the embedment depth ***D***. It should be noted that in this method the net passive resistance ***B*** is not reduced by a safety factor ***Fs = 1***

(Without a safety factor)

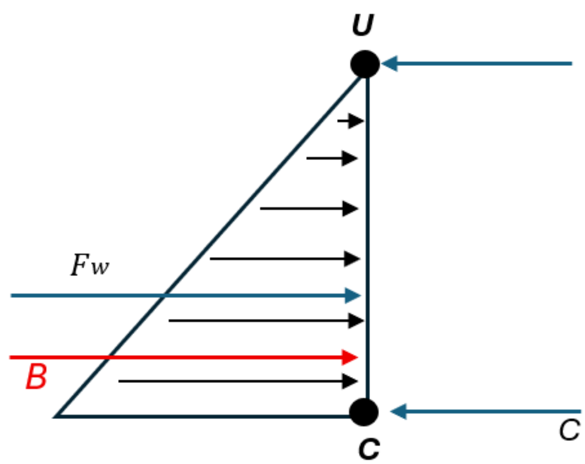

Figure 27: *Lower beam*

$$\sum M/O = 0$$

$$B\left(\frac{f_0}{3}\right) + Fw'\left(\frac{1}{2}(f_0 + t)\right) - R.f_0 = 0$$

Terme B: $10.99{f_0}^3$

Terme Fw': $15{f_0}^2 + 8.52f_0 + 1.21$

Terme R: $-43.518f_0$

$$10.99{f_0}^3 + 15{f_0}^2 - 34.998f_0 + 1.21 = 0$$

$$\boldsymbol{f_0 = 1.204\ m}$$

$$D = t + 1.2f_0$$

$$\boxed{\boldsymbol{D = 1.73\ m}}$$

Step 6 : From the horizontal force equilibrium of the lower beam, deduce the passive resistance force ***C.***

$$\sum \boldsymbol{F = 0}$$

$$B + Fw' - R - C = 0$$

$$C = B + Fw' - R$$

$$C = 47.8 + 44.6 - 43.518$$

$$\boxed{\boldsymbol{C = 48.9\ kN/m}}$$

Step 7: Verify that the sum of destabilizing moments about point ***A*** is less than the sum of stabilizing moments about the same point.

Point ***A*** is the anchor level, located 0.70 m below the top of the sheet pile (elevation $z_A = -0.70$ m).

The destabilizing forces (active earth pressures and water pressures) and the stabilizing forces (passive resistance ***B*** and reverse passive resistance ***C***) are positioned as follows:

Destabilizing Forces :

$$P = 29{,}57\ kN/m$$

$$Q = 9{,}477\ kN/m$$

$$Fw = 45kN/m$$

$$P' = 2{,}15\ kN/m$$

$$Fw' = 44{,}64\ kN/m$$

$$\boldsymbol{\sum M_{destab} = P.\left(\frac{2}{3}(H_{tot} + t) - 0.2\right) + Q.\left(\frac{H + D}{2} - 0.2\right) + P'.(\frac{1}{3}t + H_{tot} - 0.2) + Fw.(\frac{2}{3}H + 0.7) + Fw'.(\frac{1}{2}D + (H - 0.2))}$$

$$\boldsymbol{\sum M_{destab} = 394.61\ kN.m/m}$$

stabilizing Forces :

$$B = 47.8\ kN/m$$

$$C = 48.9\ kN/m$$

$$\sum M_{stab} = B.(\frac{2}{3}D + (H_{tot} - 0.2)) + C.(f_0 + t + H_{tot} - 0.2)$$

$$\boldsymbol{\sum M_{stab} = 485.68\ kN.m/m}$$

So: $\boldsymbol{\sum M_{stab} > \sum M_{destab}}$

So, the condition of step 7 is therefore satisfied: safety against overturning about the anchor is ensured (even with safety factor of 1 on the net passive resistance, the stabilizing moments remain greater than the destabilizing moments).

Remark:

- **Surcharge (**q**)**: Acts on the backfill surface, directed vertically (which adds to the horizontal earth pressure Q), its role is Driving.
- **Active Earth Pressure (**P / P'**)**: Acts on the back face of the wall, directed seaward; its role is Driving.
- **Passive Earth Pressure (**B**)**: Acts on the front face (embedded part), directed landward, its role is Stabilizing.
- **Tie-Rod Force (**T**)**: Acts at the top of the wall (connection point), directed landward, its role is **Stabilizing**.
- **Water Pressure (**Fw / Fw'): Acts on both faces of the wall, directed from high water to low water, its role can be Driving or Stabilizing depending on the water level difference.
- **Counter-Passive (**C**)**: Acts on the back face (below point O), directed seaward, its role is Stabilizing (it prevents rotation).

IV.5. BEHAVIOR OF A SHEET PILE WALL USING SPW 2006

IV.5.1. PRESENTATION OF THE SPW 2006 SOFTWARE

SPW 2006 is a specialized software for the analysis and design of sheet pile walls, employing beam on elastic foundation and limit equilibrium methods.

It models cantilever, anchored, strutted, and Deadman systems, calculating wall embedment, bending moments, shear forces, and anchor loads under soil, water, and Overload conditions.
The user-friendly interface enables rapid parametric studies, producing safe, efficient designs for deep excavations and waterfront structures. **(Verruijt, 2006)**

IV.5.2. CALCULATION METHODOLOGY ON SPW 2006

IV.5.2.1. GEOMETRY AND SOIL MODEL (FROM THE INPUT TABLES)

Properties of Soil Layers

Loading Step : 1 Right Side

No.	Soil Name	H	Wd	Ws	Zw	Cap	q	c	Ka	Kp	Kn	Dw
		m	kN/m³	kN/m³	m	m	kN/m²	kN/m²	--	--	--	m
1	Soil 1	0.700	10.000	16.000	0.500	0.000	10.000	0.000	0.243	4.120	0.500	0.070
2	Soil 2	3.200	10.000	16.000	0.500	0.000	10.000	0.000	0.243	4.120	0.500	0.070
3	Soil 3	2.400	10.000	16.000	0.500	0.000	10.000	0.000	0.243	4.120	0.500	0.070

Properties of Soil Layers

Loading Step : 1 Left Side

No.	Soil Name	H	Wd	Ws	Zw	Cap	q	c	Ka	Kp	Kn	Dw
		m	kN/m³	kN/m³	m	m	kN/m²	kN/m²	--	--	--	m
1	Soil 1	0.700	0.000	10.000	0.900	0.000	0.000	0.000	1.000	1.000	1.000	1.000
2	Soil 2	3.200	0.000	10.000	0.900	0.000	0.000	0.000	1.000	1.000	1.000	1.000
3	Soil 3	2.400	10.000	16.000	0.900	0.000	0.000	0.000	0.243	4.120	0.500	0.070

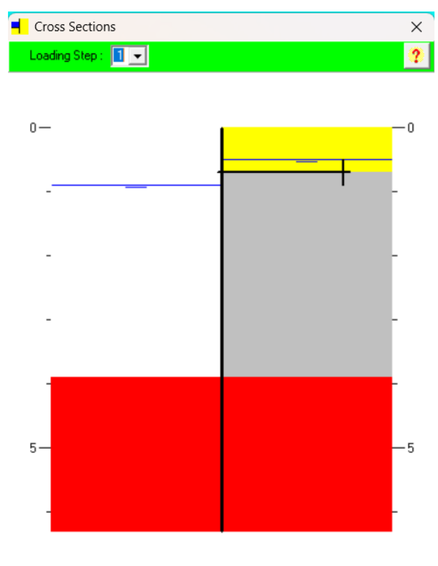

Figure 28: *Illustration of the slice data "SPW" (input)*

Left side (excavation side)

- **Layers 1 and 2 (0 – 3.9 m):** $W_d = 0,\ \ W_S = 10\ kN/m^3,\ \ K_a = K_p = 1.$

  These layers model the **excavated zone**. Setting the dry unit weight $\boldsymbol{W_d}$ to zero removes the soil, the active/passive coefficients equal to 1 are conventional and ensure only water pressure acts if a water table is present.

- **Layer 3 (3.9 – 7.4 m):** $W_d = 10\ kN/m^3,\ \ W_S = 16\ kN/m^3,\ \ K_a = 0.243,\ \ K_p = 4.12.$
  This is the **passive resistance zone** below dredge level. The large $\boldsymbol{K_p}$ value (>1) means that, when the wall moves into this side, the soil mobilizes **passive earth pressure**. The low cohesion and the high $\boldsymbol{K_p}$ indicate a **dense sand** ($\varphi = 37.5°$ as derived from Rankine's relation).

Right side (retained side)

- **All three layers are identical:** $W_d = 10kN/m^3,\ W_S = 16kN/m^3$, q=10kN/m^3, $K_a = 0.243,$

  $K_p = 4.12.$

  This is the **retained soil** under a uniform Overload of $\boldsymbol{10\ kN/m^3}$. The water table is at **0.5 m** depth, in order to make **an imbalance**. The active coefficient $\boldsymbol{K_a = 0.243}$ corresponds to the theoretical **active limit state** .

IV.5.3. INTERPRETATION OF THE OUTPUT GRAPHS (LOADING STEP 1)

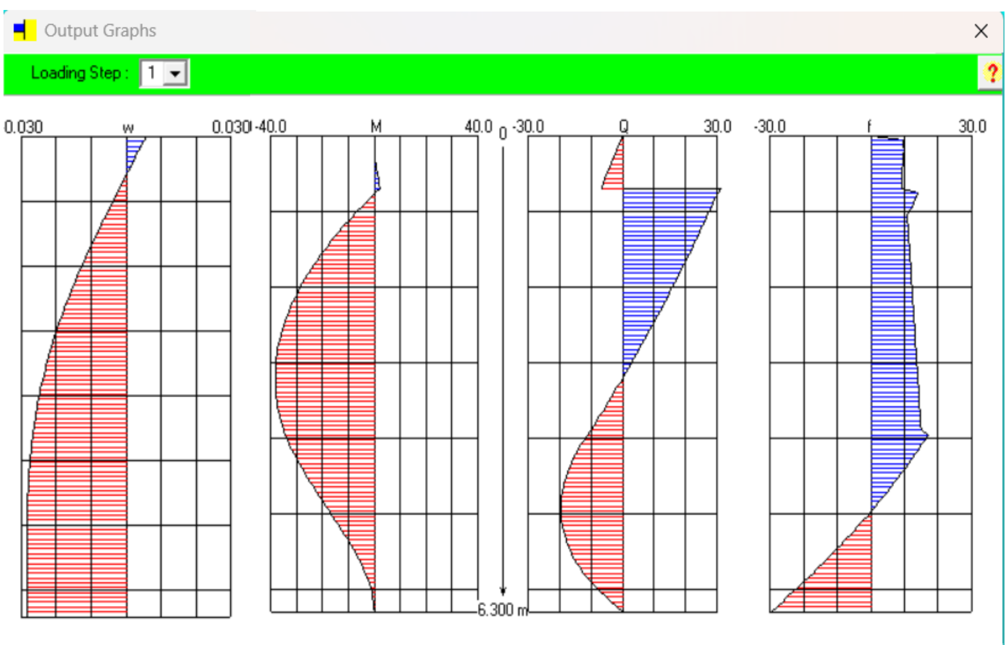

Figure 29: *SPW Simulation Results*

1. Net pressure distribution $f(z)$ (right and left)

In the SPW program, the resultant soil pressure $P(z)$ is the net force per unit height acting on the wall, positive to the right (towards the *retained* side). Therefore:

- **Positive** f means net pressure pushes the wall **to the right** (passive resistance from the excavation side dominates).
- **Negative** f means net pressure pushes the wall **to the left** (active pressure from the retained side dominates).

The displacement curve $w(z)$ is positive to the right (retained side) in the upper part of the wall and negative to the left (excavation side) near the toe. Consequently:

- **Upper zone (0-0.5 m)**: wall moves right which the retained soil mobilizes active pressure (pushes left) so f **is negative**.
- **Lower zone (0.5 m-toe)**: wall moves left which the excavation side mobilizes passive pressure (pushes right) so f **is positive**.
- **Zero-crossing of** f marks the depth where the net pressure changes sign, which corresponds to the **center of pressure**.

2. Force and Moment Values

Anchor Force

- **Per meter width of wall**: $T = 40.05$ kN/m
- **Spacing**: $S = 3.6$ m
- **Force per anchor**: $T_{\text{anc}} = 40.05 \times 3.6 = 144.18$ kN

Maximum Bending Moment

- $M_{\max} = 40$ kN · m/m at about 3.5 m depth, based on the earlier graph shape.

3. Ultimate Limit States (ULS)

a) Anchor pull-out

The chosen tie rods are **M56/45 in steel ASDO 355**, with an admissible load $T_a = 497$ kN (*after BTP infraestructuras, 2018*).
The safety factor against pull-out failure is:

$$\boldsymbol{F_s = \frac{497}{151.38} = 3.28}$$

This comfortably exceeds the typical minimum of **1.5-2.0** required by the ROM 0.5-05 and the principles of Stability against rotational failure, So, **Anchor design is safe.**

b) Bending resistance of the sheet pile (GU 6N)

- **Steel grade**: S 240 GP ($f_y = 240$ MPa)
- **Partial safety factor:** $\gamma_{M1} = 1.10$ design yield stress $f_{yd} = 218.18$ MPa
- **Section modulus:** $W_x = 1240\ cm^3/m$
- **Design bending resistance:**

$$\boldsymbol{M_{RD} = \frac{\beta_B \cdot f_y \cdot W_x}{\gamma_{M1}} = \frac{1 \cdot 240 \times 10^3 \cdot 1245 \times 10^{-6}}{1.10} = 271.6\ \mathrm{kN \cdot m/m}}$$

$$\boldsymbol{M_{max} = 40\ \mathrm{kN \cdot m/m} < M_{RD} = 271.6\ \mathrm{kN \cdot m/m}}$$

So, Bending resistance is amply sufficient.

4. Serviceability Limit State (SLS)

The maximum lateral displacement is about **28 mm**. This value should be checked against the project-specific tolerance for the quay structure and adjacent facilities. Provided it is acceptable, **the SLS is satisfied.**

5. Overall Assessment

The sheet pile wall designed with the P-Y method. All examined limit states bending, anchor pullout, and displacements are met with substantial safety margins. The analysis confirms that the wall behaves as a flexible anchored structure, and the real pressure distribution (as obtained by SPW) differs from the simplified limit equilibrium diagrams, highlighting the value of the **Soil Structure Interaction Modeling** approach for a realistic design.

IV.6. BEHAVIOR OF A SHEET PILE WALL USING PROSHEET

IV.6.1. PRESENTATION OF THE PROSHEET SOFTWARE

ProSheet is ArcelorMittal's dedicated sheet pile design software, enabling static and seismic analysis of U, Z, combined, and tubular steel walls via the subgrade reaction method. It automates profile selection and optimization, calculating embedment depth, bending moments, deflections, and anchor forces under complex soil and loading conditions. A fully code-compliant workflow (Eurocodes and other international standards) rapidly delivers safe, material efficient designs for quay walls, flood defences, and deep excavations. **(ArcelorMittal, 2020)**

IV.6.2. CALCULATION METHODOLOGY ON PROSHEET

IV.6.2.1. INPUT PARAMETERS

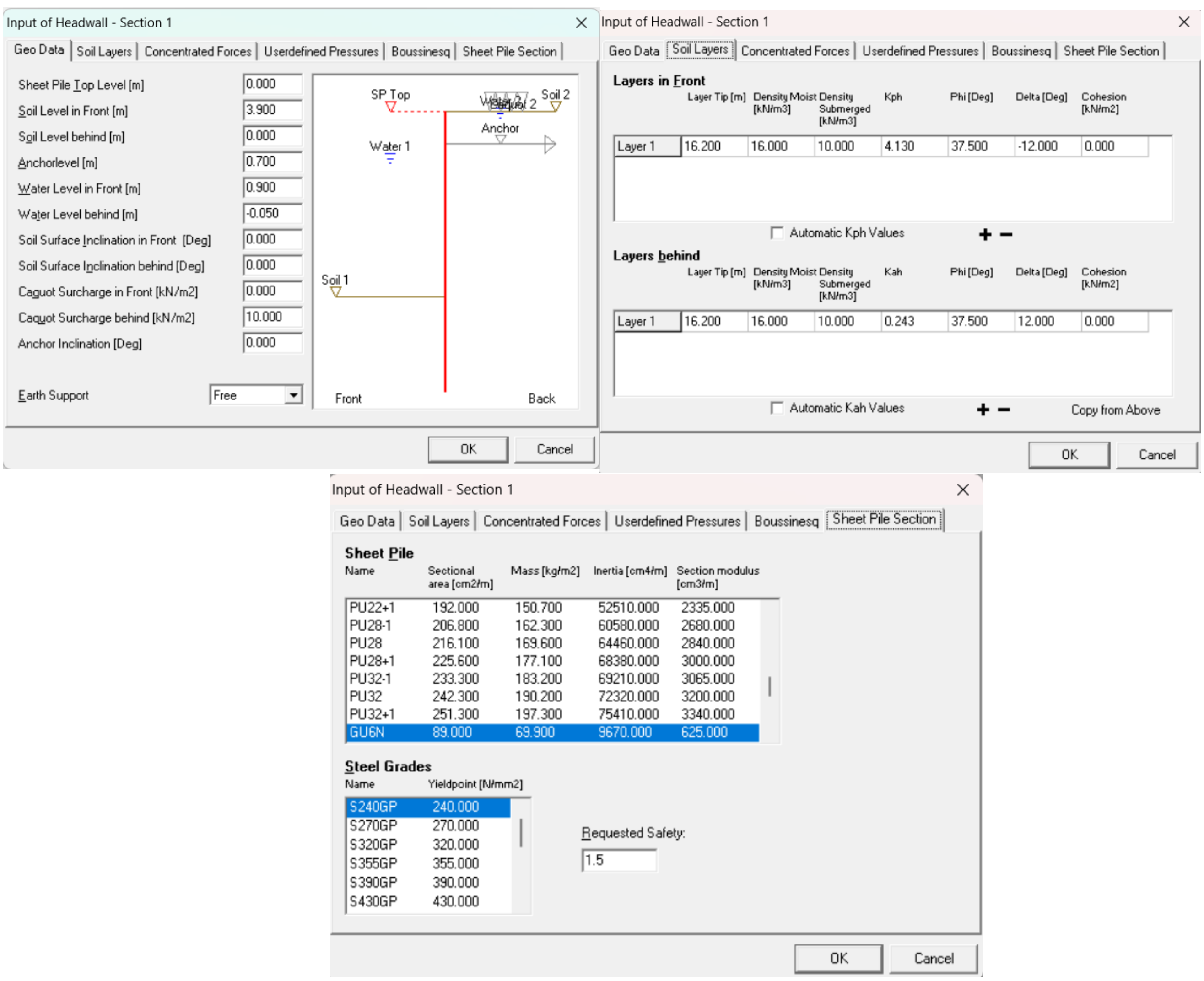

Figure 30: *Illustration of the slice data "PROSHEET" (input)*

The input parameters were consistent with the geotechnical characterization from Chapter III. Key values include: sheet pile top level at -0.90 m, soil level behind wall at -0.90 m, soil level in front (dredge level) at –4.80 m, anchor level at -0.70 m, water level behind at -0.85 m and in front at -0.90 m. Overload behind the wall was 10 kN/m² for both methods, Overload in front was 0 kN/m² for both methods. Soil parameters for the passive side (in front): moist density 16 kN/m³, submerged density 10 kN/m³, friction angle 37.5°, wall friction angle -12°, cohesion 0 kPa, passive coefficient Kp = 4.13. For the active side (behind): same densities and friction angle, wall friction angle +12°, active coefficient Ka = 0.243. The sheet pile section was GU 6N with moment of inertia 9,670 cm⁴/m, section modulus 625 cm³/m, sectional area 89 cm²/m, mass 69.9 kg/m², steel grade S240 GP (yield strength 240 MPa), and required safety factor 2.0.

IV.6.3. INTERPRETATION OF THE OUTPUT GRAPHS

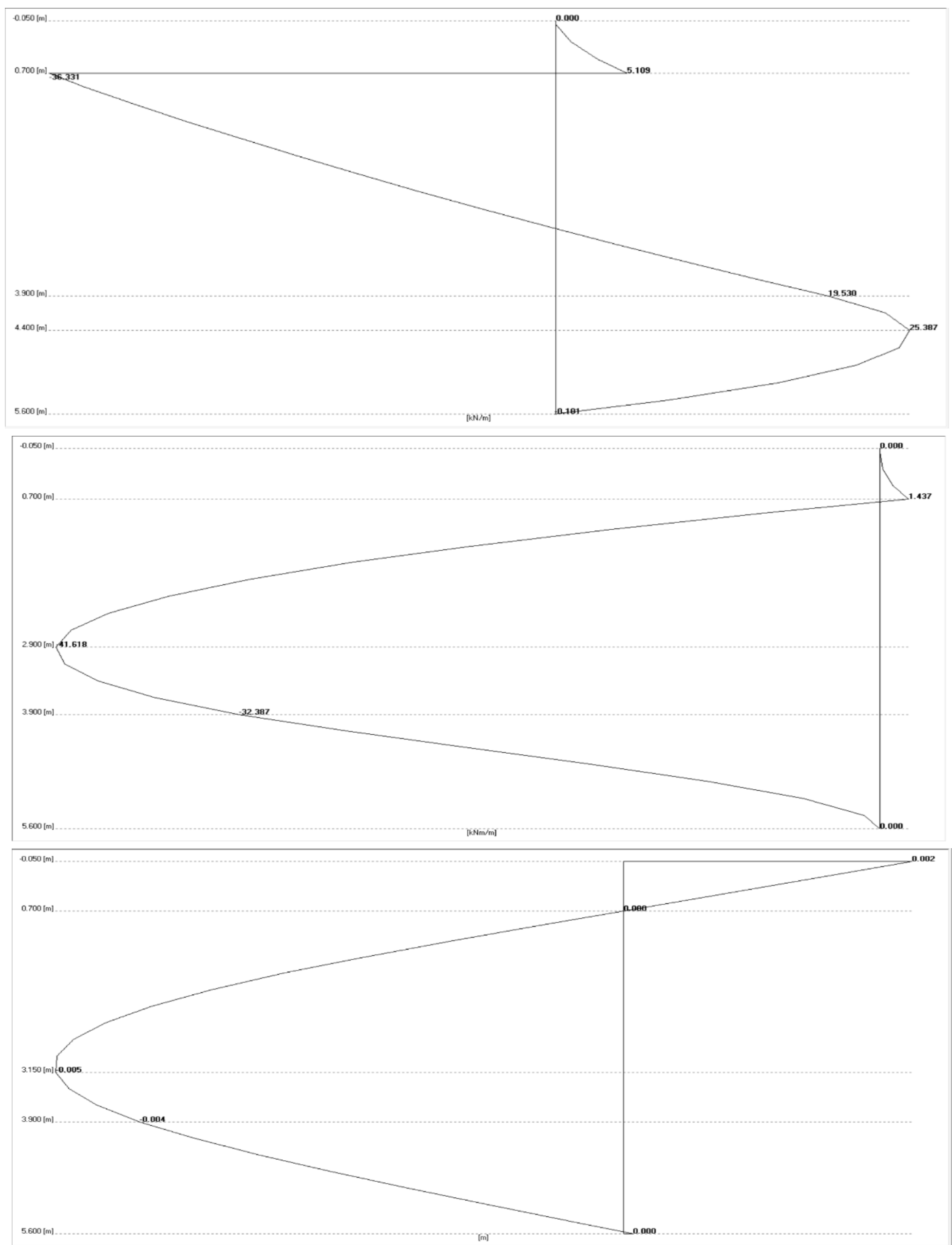

Figure 31: *PROSHEET Free Earth Support Simulation Results(Bending Moments, Shear Force and Displacment)*

Table 12: *Prosheet outputs*

		Depth [m]
Name	GU6N	
Inertia [cm4/m]	9670.000	
Modulus [cm3/m]	625.000	
Area [cm2/m]	89.000	
Mass [kg/m2]	69.900	
Steel Grade [N/mm2]	240.000	
Minimal Moment [kNm/m]	-41.618	2.900
Maxmimal Moment [kNm/m]	1.437	0.700
Normal Forces at Max. Moment [kN/m]	3.828	2.900
Normal Forces at Min. Moment [kN/m]	0.509	0.700
Deflection at Min. Moment [m]	-0.005	2.900
Deflection at Max. Moment [m]	0.000	0.700
Min. Stress at Min. Moment [N/mm2]	-66.188	2.900
Max. Stress at Min. Moment [N/mm2]	67.048	2.900
Min. Stress at Max. Moment [N/mm2]	-2.243	0.700
Max. Stress at Max. Moment [N/mm2]	2.357	0.700
Safety > Req. Safety = 1.500	3.580	
Sheet Pile Top Level [m]	0.000	
Sheet Pile Tip Level [m]	5.600	
Sheet Pile Length [m]	5.600	
Included OverLength [m]	0.000	
Vertical Equilibrium [kN/m]	-1.699	
Anchor Force (horiz.) [kN/m]	-41.440	

- **Geometry**: Total embedded length of 5.600 m, with no overlength included.
- **Loading & Moments**: The horizontal anchor force is -41.43 kN/m. The governing design location is at a depth of 2.900 m, where the minimum bending moment is -41.62 kN·m/m occurs at the anchor level (0.700 m).
- **Deflections**: Maximum deflection toward the excavation side (-0.005 m) occurs at the 2.900 m depth, with zero deflection at the anchor level.
- **Safety**: The calculated safety factor is 3.580, exceeding the required 1.500.

IV.7. EXCAVATION BOTTOM STABILITY

IV.7.1. FAILURE MECHANISMS

For a sheet pile wall retaining an excavation in granular soil, two potential failure modes must be checked at the excavation bottom: **(Bouafia, 2018b)**

Bearing capacity failure (heave): The vertical load transmitted by the retained soil and the Overload exceeds the resistance of the soil below the excavation.

According to **NF P94-282** and **Eurocode 7**, the stability against bearing capacity failure is verified using the following criterion :

$$q_{dst} \leq \frac{q_{stb}}{F_s} \tag{23}$$

where:

- q_{dst} = destabilising vertical stress at the base of the wall (on the retained side)
- q_{stb} = stabilising vertical stress (bearing capacity of the soil below the excavation)
- F_s = required safety factor, taken as **2.0** for permanent work

IV.7.2. GENERAL FORMULATION FOR A c' – Φ' SOIL

The stabilizing stress is given by the **Prandtl-Reissner** bearing capacity formula (strip footing analogy):

$$q_{stb} = N_q.\sigma'_{v2} + \frac{N_q - 1}{tan\emptyset'}.c' \tag{24}$$

where:

- N_q = bearing capacity factor:

$$N_q = \frac{1 + \sin\emptyset'}{1 - \sin\emptyset'}\exp\,(\pi\tan\emptyset') \tag{25}$$

- σ'_{v2} = effective vertical stress at the base of the wall on the **excavation side** (after removal of the soil)
- c' = effective cohesion of the soil below the excavation.

The destabilizing stress is the effective vertical stress at the same depth on the **retained (land) side**:

$$q_{dst} = \sigma'_{v1} = \gamma_d(H_{tot} + D) + q_{surcharge} \tag{26}$$

where:

- H_{tot} = excavation depth from original ground surface to dredge level
- D = embedment depth of the sheet pile below dredge level
- γ_d = dry unit weight of the soil
- $q_{\mathrm{surcharge}}$ = uniform Overload on the retained side.

Step 1 – Compute the bearing capacity factor N_q

$$N_q = \frac{1+\sin\phi'}{1-\sin\phi'}\exp\left(\pi\tan\phi'\right) = \frac{1.6088}{0.3912}\exp\left(3.1416\times 0.7673\right)$$

$$\boxed{N_q = 45.8}$$

Step 2 – Effective vertical stress on the excavation side (σ'_{v2})

At the base of the wall D depth below dredge level), the soil on the excavation side has been removed. The effective vertical stress is only due to the submerged weight of the remaining soil below the excavation bottom:

$$\sigma'_{v2} = \gamma_d \cdot D = 16.7\times 1.73 = \boxed{28.89\ \mathrm{kN/m2}}$$

Step 3 – Stabilising stress (q_{stb}) using the full formula

$$q_{stb} = N_q\cdot\sigma'_{v2} + \frac{N_q - 1}{\tan\phi'}\cdot c'$$

Because $c' = 0$ (purely frictional sand), the second term vanishes:

$$q_{stb} = 45.8\times 28.89 = \boxed{1323.162\ \mathrm{kN}/m^2}$$

Note: The full expression is retained here for completeness. If the soil had a non-zero cohesion, this term would contribute significantly to safety.

Step 4 – Destabilizing stress on the land side (q_{dst})

The retained soil is not excavated, its effective vertical stress at the same depth (wall base) includes the weight of the soil above dredge level plus the Overload:

$$q_{dst} = \gamma_d \,(H_{\text{tot}} + D) \; + \; q$$

$$q_{dst} = 16.7 \times (3.90 + 1.73) + 10 = 16.7 \times 5.63 + 10 = 94.02 + 10 = \boxed{104.02 \text{ kN}/m^2}$$

Step 5 – Safety factor against base bearing capacity failure

$$q_{dst} \le \frac{q_{stb}}{F_s}$$

$$104.02 < \frac{1323.16}{2} = 661.58$$

$$\mathbf{104.02 < 661.58} \ \textit{so the} \ \textbf{base bearing capacity is fully satisfied.}$$

IV.8. RECAP OF PREDIMENSIONING THE SHEET PILE WALL

This chapter applied three traditional analytical methods (Simple Thrust, Sheet Pile Embedded, and Equivalent Beam) to the most loaded section (Quay G2: water depth 3.00 m, Overload 10 kN/m). Results were then compared with specialized software (SPW 2006 and PROSHEET).

Table 13: *Comparative table of the results obtained*

Method / Software	***Embedment Depth D (m)***	***Total Sheet Pile Length (m)***	***Anchor Force T (kN/m)***	***Max Bending Moment M_{max}***
Simple Thrust *(with $F_s = 2$)*	*3.60*	*7.50*	*67.00*	*Not calculated*
Sheet pile embedded with *(with $F_s = 2$)*	*1.07*	*4.97*	*Not given*	*Not calculated*
Equivalent Beam	*1.73*	*5.63*	*42.68*	*Not calculated*
SPW 2006 *(P-Y curve method)*	*Not explicitly stated*	*6.3*	*40.05*	*40*
PROSHEET Free Earth Support	*1.70*	*5.6*	*41.44*	*-41.62*

IV.9. CONCLUSION

To size the sheet pile wall at Quay G2, we applied three classical analytical methods (Simple Thrust, Sheet Pile Embedded, and Equivalent Beam) for a retained water depth of 3.00 m and a Overload of 10 kN/m², complemented by preliminary checks with SPW 2006 and PROSHEET. The exercise revealed several important insights.

- Wide analytical scatter: Embedment depths vary from 1.07 m (Embedded) to 3.60 m (Simple Thrust) over a factor of 3 showing strong method dependency.
- Simple Thrust is conservative (3.60 m, 67 kN/m) due to safety factor applied to passive pressure, useful for safe preliminary sizing.
- Embedded method gives unrealistic results (1.07 m) for flexible GU 6N piles in loose/medium sand, avoid in this context.
- Equivalent Beam (1.73 m, 42.68 kN/m) matches PROSHEET and GEO5 closely, reliable for predimensioning in sand.
- Bending moment demand (SPW 2006: 40 kN·m/m) is far below capacity (271.6 kN·m/m), design will likely be governed by displacement (~28 mm), not strength.

CHAPTER V: CALCULATION METHODOLOGY

V.1. INTRODUCTION

The design of a sheet pile wall requires a thorough analysis of soil-structure interaction to ensure stability and durability under various loading conditions. Several calculation methods exist, ranging from classical analytical approaches (Blum method) to advanced numerical simulations.

In this chapter, we present the calculation methodology adopted for the detailed design of the sheet pile quay wall of the Marina Bay of Algiers, focusing on the most heavily loaded section: **Quay G2** (water depth **3.00 m**, Overload **10 kN/m²**). Two numerical software packages are used:

- **GEO5** : a comprehensive geotechnical suite integrating both a Finite Element Method (FEM) module and a dedicated sheet pile wall module based on the subgrade reaction method (P-Y curves).
- **PLAXIS 2D** : a reference software for finite element modeling, allowing simulation of non-linear soil behavior, staged construction, and soil-structure interaction.

V.2. GENERAL MODELING FRAMEWORK

V.2.1. REFERENCE SECTION: QUAY G2

All calculations are performed for **Quay G2**, which presents the most severe conditions. The following geometric settings are adopted (after BTP Infraestructuras, 2018):

Table 14: *Geomertic settings of Quay G2*

Parameter	***Value***
Wall height (from sheet pile head to the dredge line)	$H_{tot} = 3.90\ m$
Height from soil surface to dredge line	$H = 3.00\ m$
Overload on backfill	$q = 10kN/m^3$
Anchor level	$0.7\ m$
Sheet pile profile	*GU 6N (steel S240 GP,* $f_y = 240\ MPA,\ w = 1245\ cm^3/m$*)*
Tie rods	*M56/45 (ASDO 355 steel), spacing 3.60 m*

V.2.2. GEOTECHNICAL PARAMETERS (SUMMARY)

From the geotechnical study (Chapter III), the following design values are adopted:

Table 15: *Geotechnical settings of the soil*

Parameter	***Symbol***	***Value***
Soil type	–	*Fine to medium sand, slightly silty*
Design friction angle	φ'	*37.5°*
Effective cohesion	c'	*0 kPa*
seismic inertia angle	ψ	*12.5°*
Unit weight (above water table)	γ	*16 kN/m³*
Saturated unit weight	γ_{sat}	*16 kN/m³*
Submerged unit weight	γ'	*10 kN/m³*
Water level (basin side)	–	*–3.00 m (dredge line)*

V.3. BEHAVIOR OF A SHEET PILE WALL USING GEO5

V.3.1. PRESENTATION OF THE GEO5 SOFTWARE

GEO5 is an integrated geotechnical software suite that combines traditional analytical design with finite element modelling in a modular, user-friendly environment. It offers specialized programs for slopes, retaining walls, foundations, and more, all sharing a common geological model. Built-in support for international design codes, automatic verification, and one-click professional reporting make it highly efficient.

V.3.2.CALCULATION METHODOLOGY ON GEO5

GEO5 is a geotechnical software that offer a lot of models and cases of study, For our case of study we choose “Sheeting Design”.

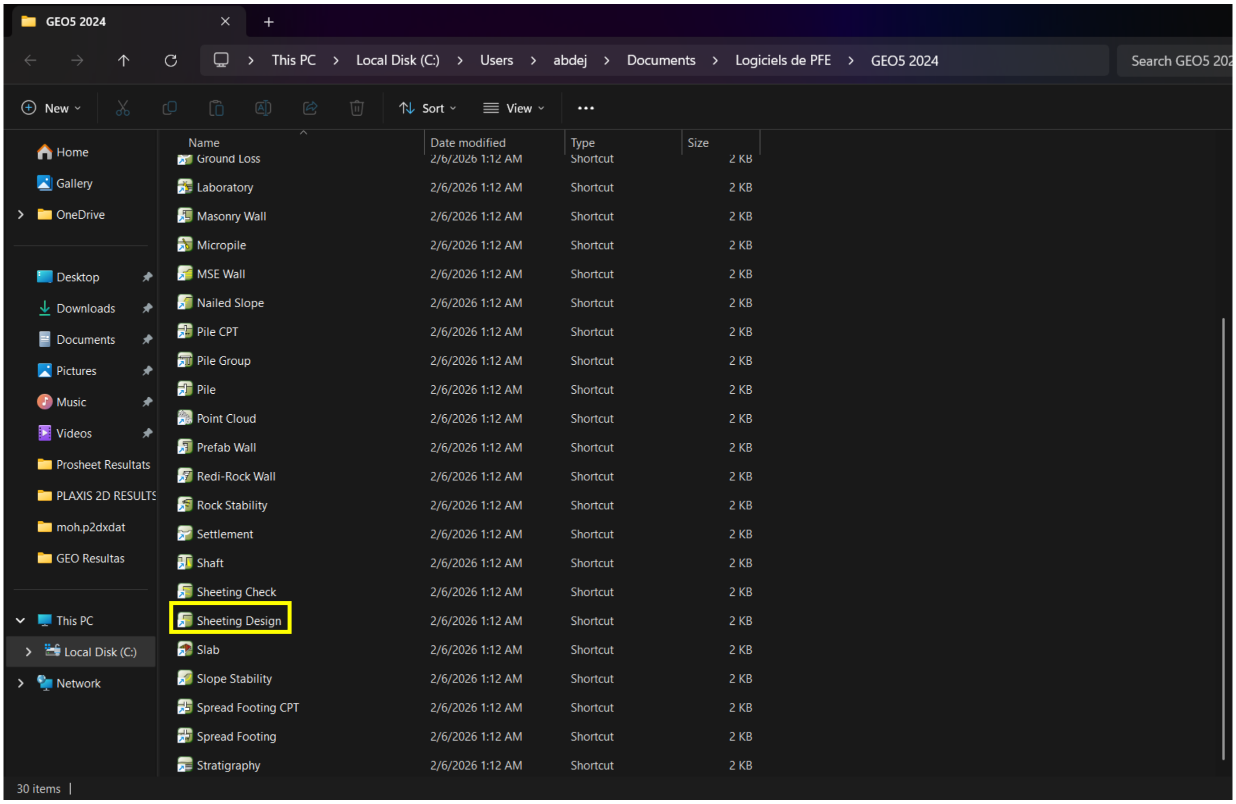

Figure 32: *The GEO5 models*

V.3.2.1.MAIN FEATURES

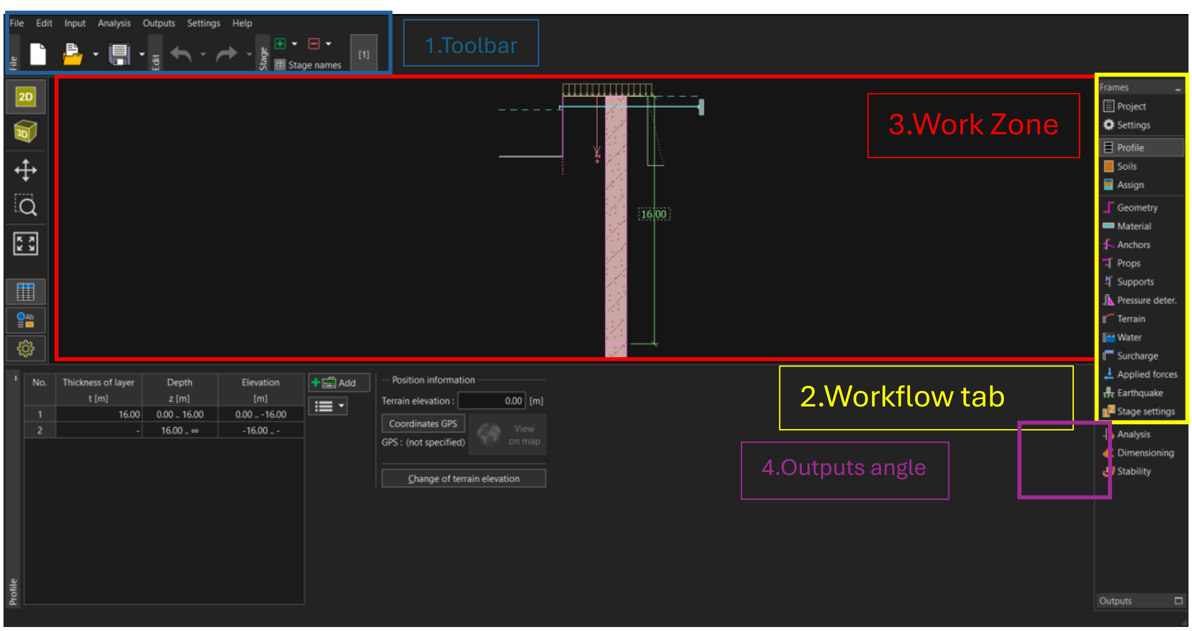

Figure 33: *GEO5 interface*

V.3.2.2. SOIL LAYER

I used the GEO5 software to input a ***16 m*** thick soil layer classified as **Silty sand (SM)** with an effective stress state, a unit weight of ***10.00 kN/m³***, a saturated unit weight of ***16.00 kN/m³***, an angle of internal friction of 37***.5°***, active and passive friction angles of ***12.5°***, and a cohesion of ***0.00 kPa***.

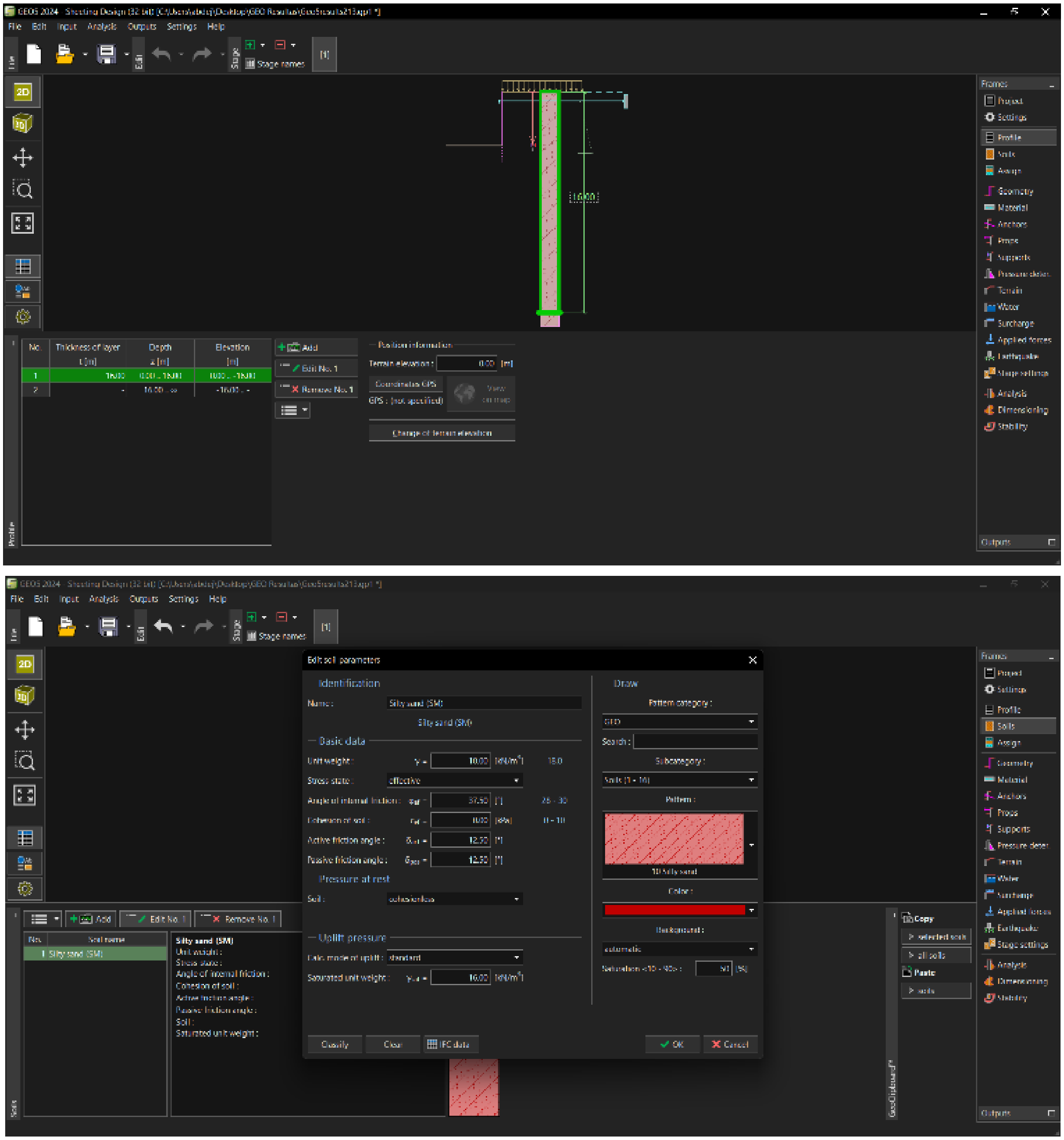

Figure 34: *Soil layer inputs*

V.3.2.3. STRUCTURAL ELEMENTS

- **Sheet pile**: plate element with GU 6N properties In the geometry stage, I selected the GU 6N EN 10248-1:S240 GP sheet pile profile from the software's built in catalog, and I set the excavation depth to **3.90 m**, measured from the top of the sheet pile down to the dredge line.

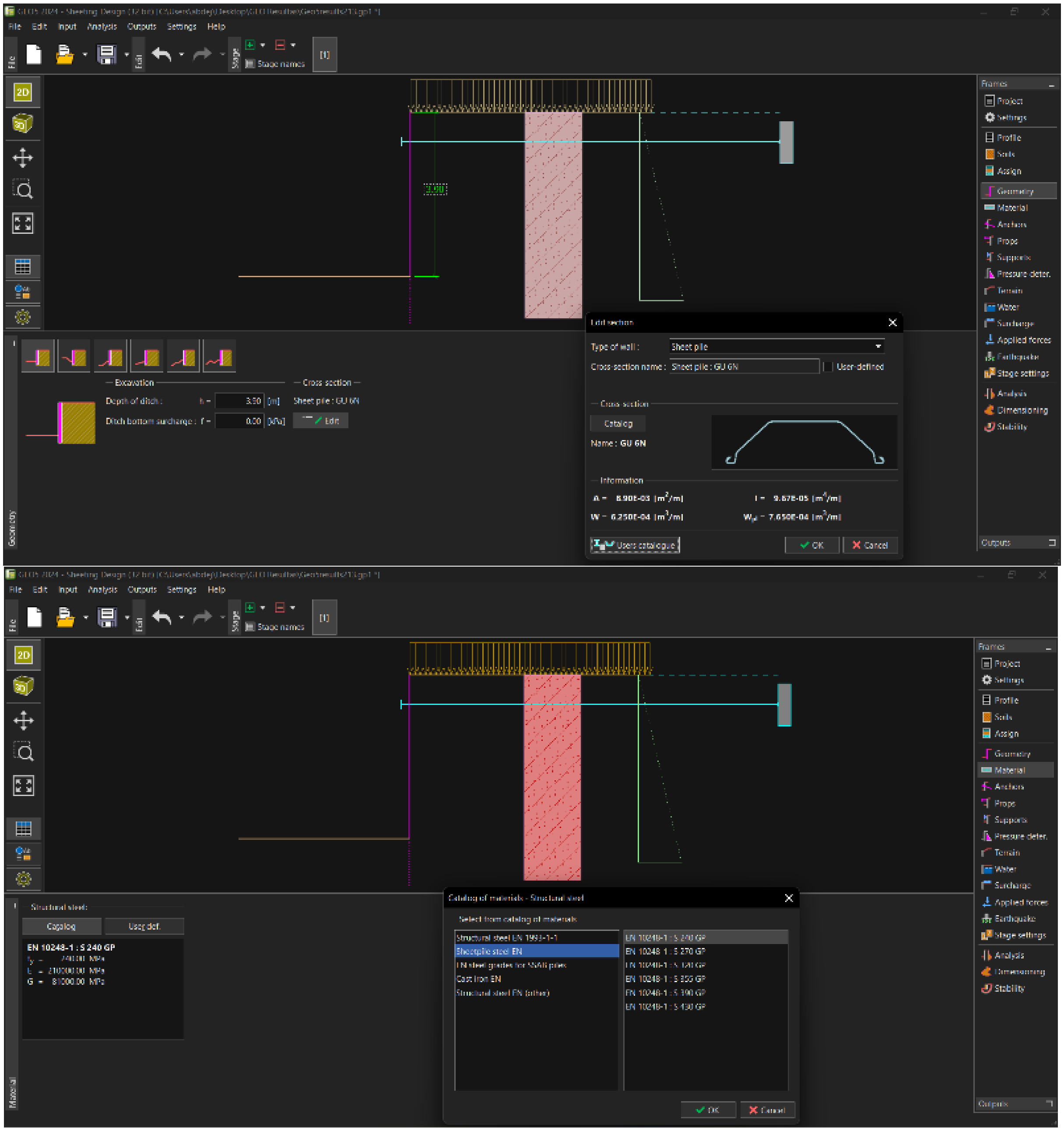

Figure 35: *Sheet pile selection*

- **Anchor**:

I configured the deadman anchor with a depth of ***0.70 m***, an overall length of ***8.00 m***, a spacing of ***3.60 m***, a tension strength of ***210.00 kN***, a cross-sectional area of ***330.000 mm²***, an elasticity modulus of ***210,000.00 MPa***, and anchor element dimensions of ***0.50 m*** in width and ***1.00 m*** in height.

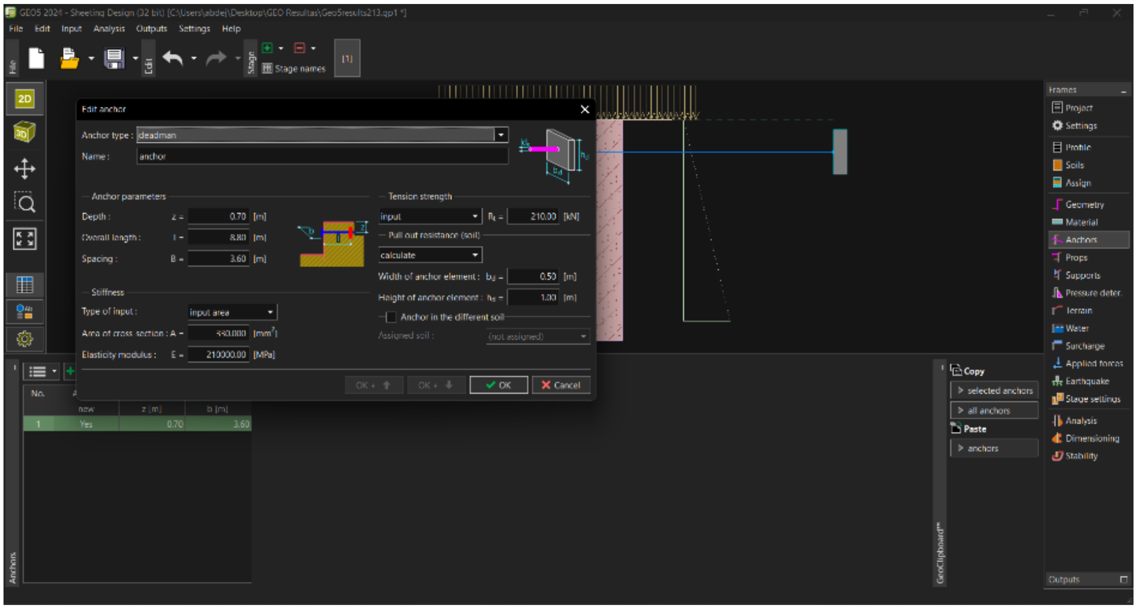

Figure 36: *Anchor settings*

- **Terrain:**

The "Terrain" dialog box allows you to specify the terrain shape. The shape is chosen using the graphical symbol. The terrain shape can be validated either within the dialog box by entering the input values. The last option to select is a general terrain shape. In this case, the frame contains a table with a list of terrain points.

For the terrain input, I selected a plain straight line with a slope angle $\beta = 0°$.

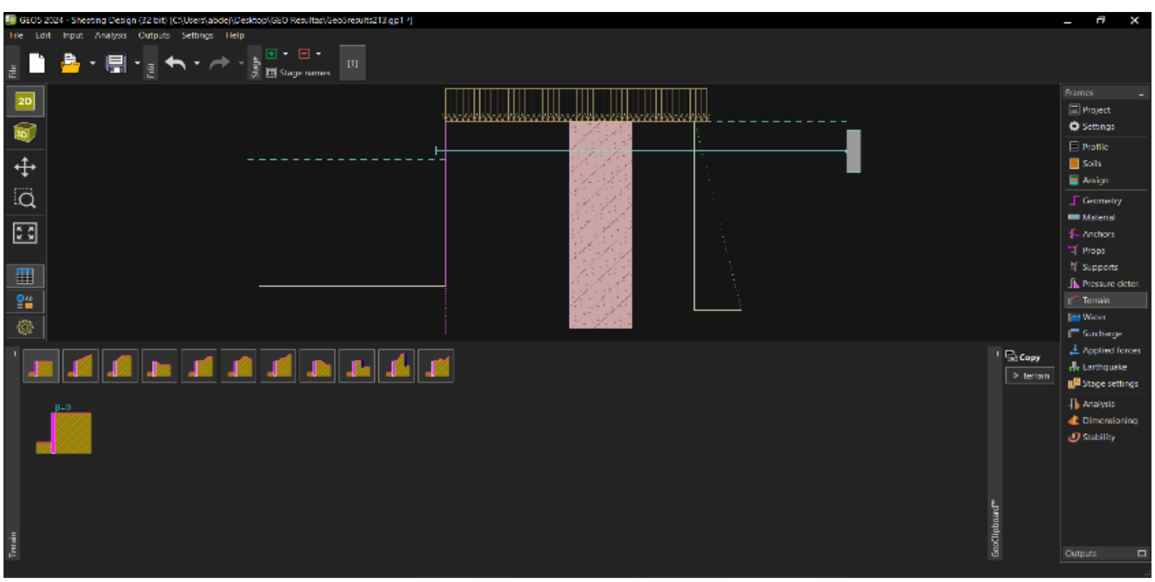

Figure 37: *Terrain selection*

V.3.2.4. WATER FLOW CONDITIONS

I set the water level parameters by defining the groundwater table (GWT) in front of the construction to 0.90 m.

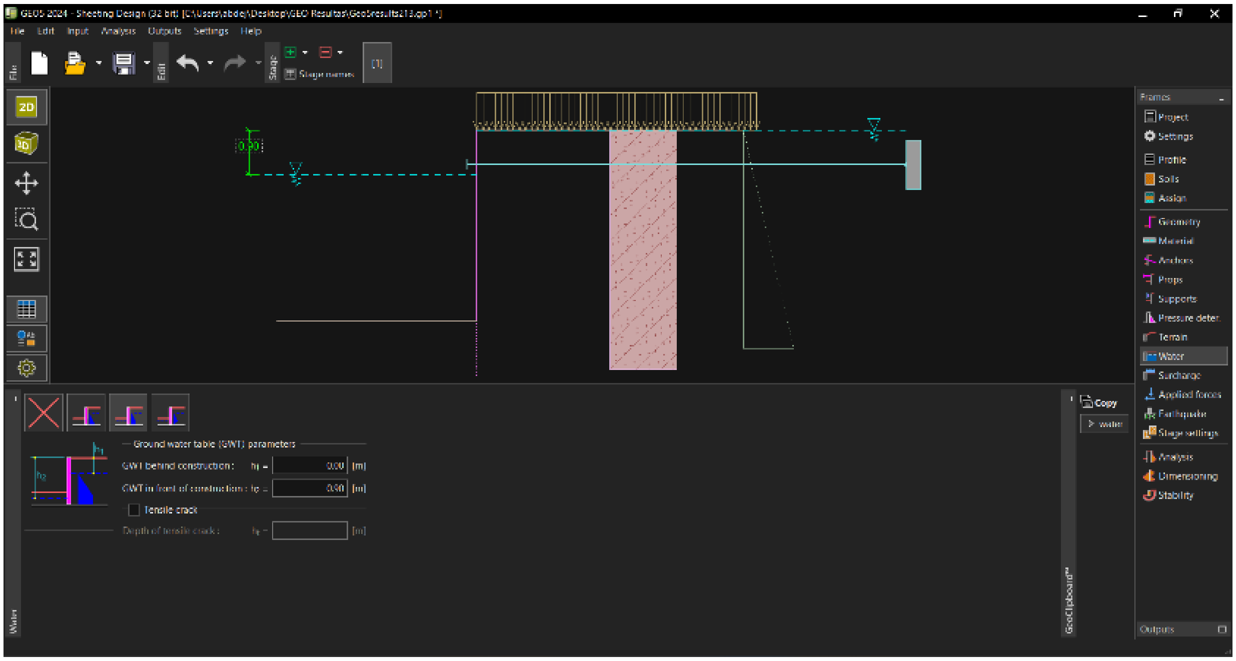

Figure 38: *Water Flow conditions*

V.3.2.5. OVERLOADS (LOADS)

The "Overload" dialog box contains a table with a list of the Overloads entered.

Entering the Overload type: linear, surface, point, and its magnitude on the upstream side of the profile.

For the Overload, I selected a surface type with a permanent action and set the magnitude to ***q = 10.00 kN/m².***

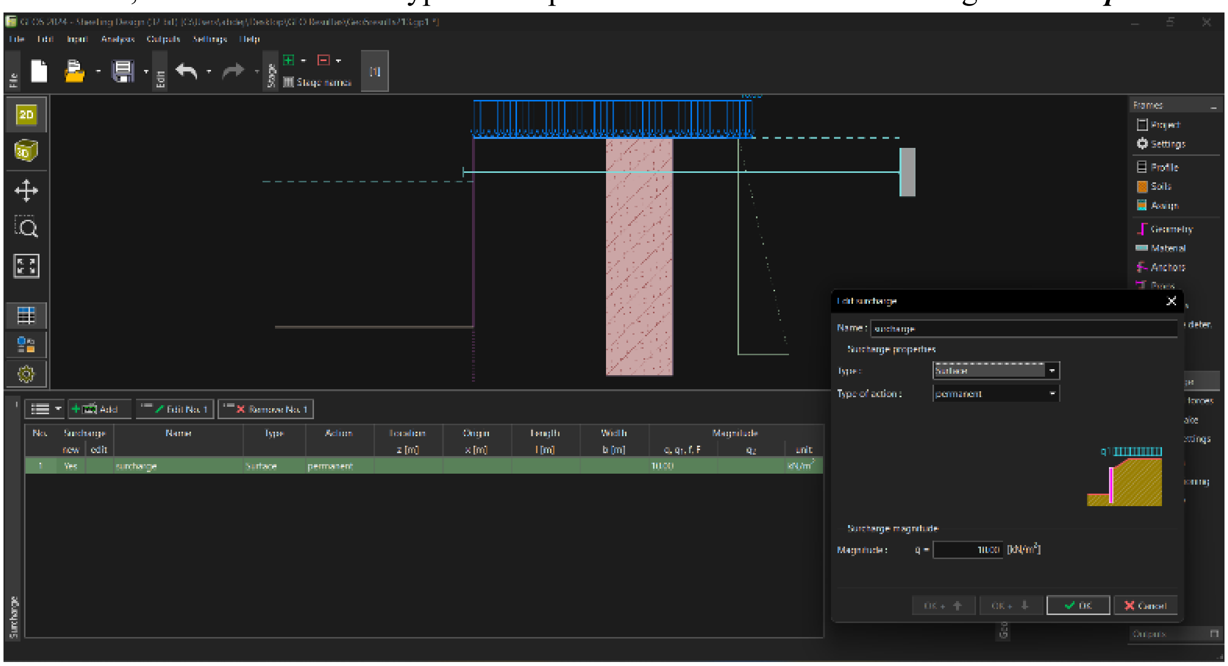

Figure 39: *Surcharge applied*

V.3.2.6. RESULTS OBTAINED

The GEO5 FEM module provides:

- Required embedment depth D
- Anchor force T (kN/m)
- Bending moment, shear force and displacement diagrams
- Global safety factors (c-ϕ reduction analysis)

Based on the GEO5 outputs, the analysis resulted in an anchor force of ***43.8 kN.m/m***, a maximum bending moment of ***43.35 kN.m/m***, a structure length of 5.66 m, and a depth in soil of ***1.72 m***. For the slope stability verification using the Bishop method for a slip surface depth of ***1.72 m***, the software calculated a safety factor of ***1.54***. Since this value is greater than the threshold of ***1.5***, the slope stability is considered acceptable.

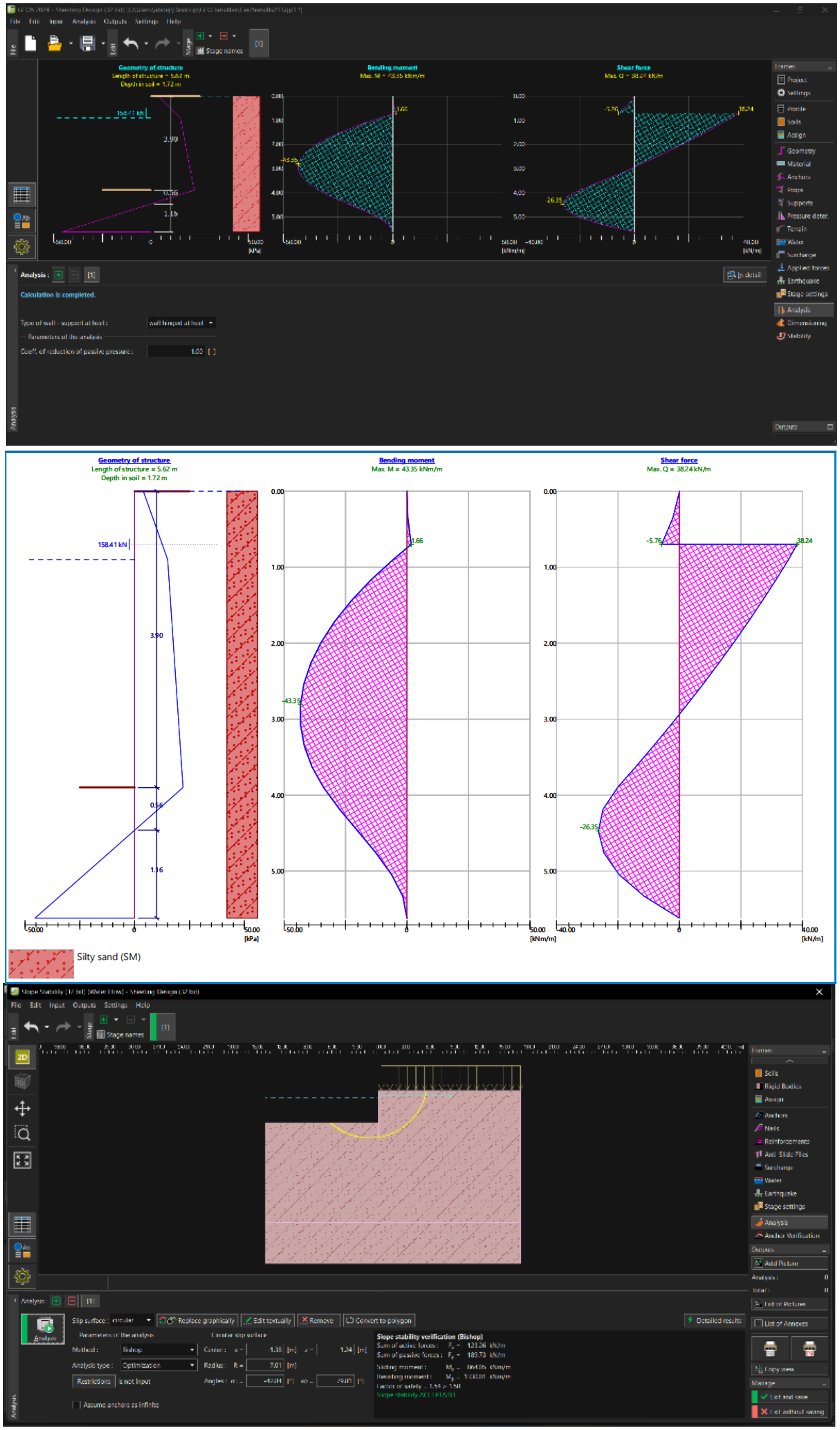

Figure 40: *GEO5 Results (outputs)*

V.4.BEHAVIOR OF A SHEET PILE WALL USING PLAXIS 2D

V.4.1 PRESENTATION OF THE PLAXIS 2D SOFTWARE

PLAXIS 2D is a leading finite element software for geotechnical analysis, modelling soil–structure interaction under both drained and undrained conditions, It simulates complex geotechnical problems like excavations, embankments, tunnels, and foundations with advanced soil models and staged construction. The user friendly interface combines robust mesh generation, groundwater flow, and safety analysis, giving engineers reliable predictions of deformation and stability.

V.4.2 METHODOLOGY USING PLAXIS 2D

V.4.2.1 MAIN FEATURES

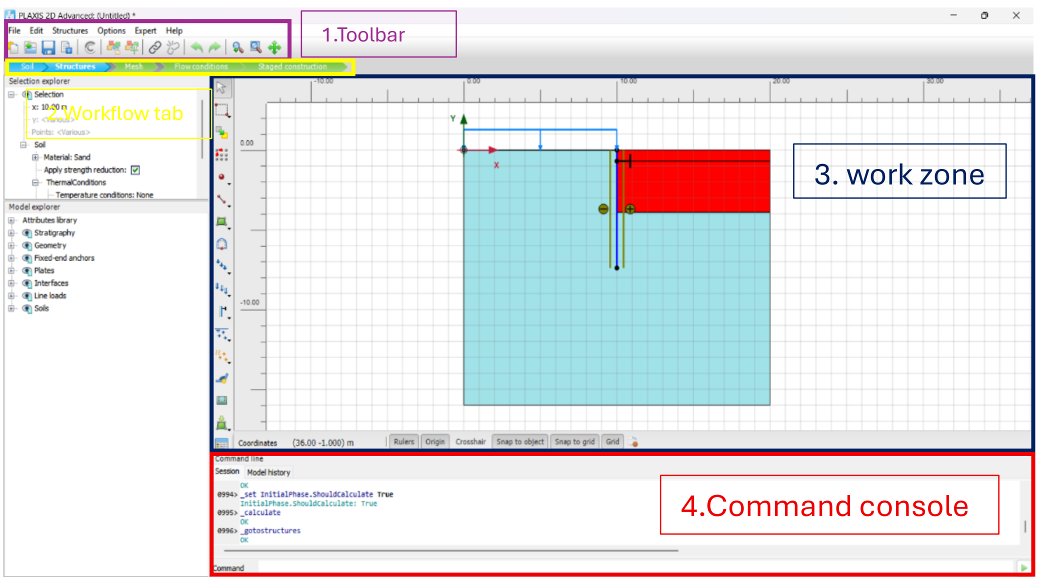

Figure 41: *PLAXIS 2D interface*

V.4.2.2. GEOMETRY, MESH AND BOUNDARY CONDITIONS

a) **Geometry**: Initial phase of constructing the simplified two-dimensional geometric model, including the definition of the study domain and structural elements.

b) **Mesh**: Generation of the finite element mesh, a critical step that determines the accuracy of the numerical results.

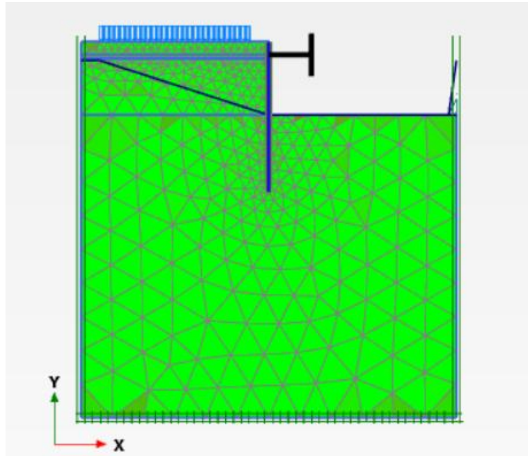

Figure 42: *Finite element mesh of the soil–sheet pile structure*

c) **Flow Conditions**: Definition of hydraulic conditions for coupled hydro-mechanical analyses.

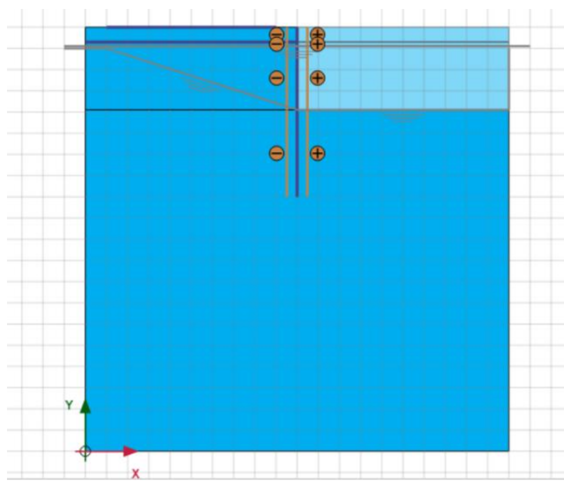

Figure 43: *Flow conditions of our structure*

V.4.3. SOIL CONSTITUTIVE MODEL

1. **Importing Soil Data**

   We integrated the geotechnical characteristics determined during the site investigation campaigns, based on SPT tests (borehole SC05) and CPT tests (CPT-1 and CPT-2), and we introduced the parameters mentioned previously: (internal friction angle φ', effective cohesion c', relative density, … etc.).

2. **Sheet pile Parameters**

   We entered all the pile characteristics for our case study.

3. **Loads**

   We complied with all normal operating conditions (CT1) together with the permanent loads of our study.

V.4.4. RESULTS OBTAINED

PLAXIS 2D provides:

- Internal force diagrams (bending moments, shear forces) in the wall
- Horizontal displacement profiles

Based on the PLAXIS 2D results, the analysis yielded an anchor force of **43.34 kN/m**, a bending moment of **47.71 kNm/m**, and a total displacement of **0.01608 m.**

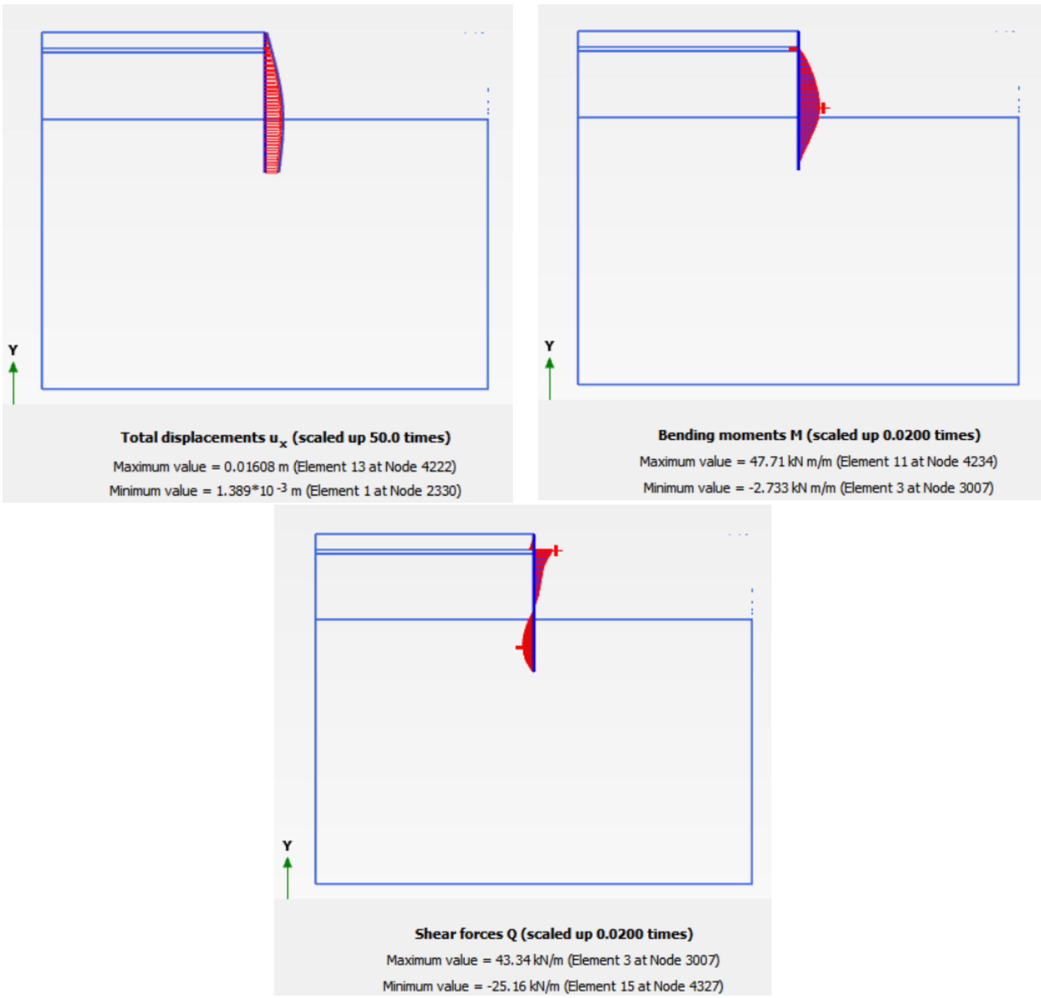

Figure 44: *PLAXIS 2D Results (Outputs)*

V.5. COMPARISON OF THE METHODS

V.5.1. COMPARISION OF METHODS ACCORDING TO ANALYSIS CRITERIA

Table 16: *Comparative table based on the technical characteristics*

Criterion	PLAXIS 2D (FEM)	GEO5 – Sheet Pile
Method	Finite Element	Subgrade reaction (Winkler)
Soil model	Hardening Soil / Mohr-Coulomb	P-Y curves (O'Neill / Reese)
Soil continuity	Yes (continuum)	No (independent springs)
Primary use	Final design, complex interaction	Preliminary design, parametric studies
Computation time	Moderate (minutes)	Very low (seconds)
Output richness	Bending moment, Stresses, plastic points, etc.	Bending moments, shear, wall length, embedment depth

V.5.2. ADVANTAGES AND LIMITATIONS OF EACH METHOD

Table 17: *Advantages and limitations of each method table*

Software	Advantages	Limitations
PLAXIS 2D	1. Very accurate modeling of real soil deformations and displacements (beyond simple stiffness checks) 2. Excellent soil-structure interaction (arching effect, multiple anchors) 3. Handles complex geometries (staged excavations, groundwater conditions)	1. High computational cost (requires a fine mesh resolution) 2. Requires advanced parameters (E'_{ref}, etc.) often missing from standard site reports 3. Does not allow for full 3D spatial analysis (asymmetrical layouts)
GEO5	1. Extremely fast (near-instant calculations) 2. Simple to use, ideal for parametric studies - Designed for geotechnical standards (Eurocodes, etc.) 3. Less demanding on input data	1. Does not calculate real wall displacements (only serviceability checks) 2. Less suitable for complex geometries or group effects from multi-level anchors 3. Analysis based on analytical simplifications (Blum method, etc.) rather than actual soil mechanics

V.5.3. SUMMARY OF SOFTWARE PACKAGES USED FOR SHEET PILE WALL

***Table 18**: Softwares packages used.*

Software	Developer	Model / Method	Primary Design Application / Stage	Open Source	How It Works	Mathematical Approach
PLAXIS 2D	Bentley Systems	**Finite Element Method (FEM)**	**Final Design & Detailed Verification** – Used for realistic serviceability (displacement) checks, seismic/dynamic analysis, and validating preliminary designs. In this study, it governed the final embedment depth (6.30 m) and provided the most reliable displacement estimate (16 mm).	**No** (Commercial)	The soil and structure are divided into a finite element mesh. Simulates staged construction, non-linear soil behavior (arching, stress redistribution), and fully coupled deformation.	Solves partial differential equations (stress-strain) using continuum mechanics with advanced constitutive models (e.g., Hardening Soil, Mohr-Coulomb).
GEO5	Fine Software	**Subgrade Reaction / FEM**	**Detailed Design & Integrated Code Compliance** – Used for rapid ultimate limit state (ULS) checks, global stability (Bishop method), and parametric studies. In this study, it confirmed the anchor force (43.8 kN/m) and global safety factor (1.54) per Eurocode 7.	**No** (Commercial)	Modular tools; the "Sheeting Design" module uses independent soil springs (Winkler) with P-Y curves; other modules use FEM for global stability and advanced analysis.	For sheeting: beam-on-elastic-foundation with non-linear springs. For global stability: limit equilibrium (Bishop) or FEM. Provides automatic code-based partial factors (Eurocode 7).
SPW 2006	Delft University	**Beam on Elastic Foundation (Subgrade Reaction)**	**Advanced Preliminary Design** – Used to evaluate wall flexibility, embedment depth, and realistic displacement profiles using non-linear soil springs. In this study, it provided a conservative upper-bound displacement (28 mm), justifying the need for 6.30 m piles.	**No** (Proprietary / Freeware)	Models the wall as a vertical beam supported by non-linear P-Y springs representing the soil. Accounts for wall-soil relative displacement to generate passive/active resistance.	Uses the subgrade reaction method with P-Y curves to solve differential equations for beam deflection, yielding shear force, bending moment, and horizontal displacement profiles.
ProSheet	ArcelorMittal	**Limit Equilibrium Method (LEM)**	**Rapid Preliminary Design & Profile Optimization** – Used for feasibility studies, initial sizing of sheet piles and anchors, and quick assessment of load combinations. In this study, it gave initial estimates (5.60 m length, 41.4 kN/m anchor force) that matched the Equivalent Beam method.	**No** (Proprietary / Freeware)	Uses classical analytical methods (e.g., Blum's free earth support method) with built-in manufacturer profile databases to automate profile selection and optimization.	Based on force and moment equilibrium (limit state) using earth pressure theories (Rankine, Coulomb, Caquot/Kérisel) to calculate active/passive pressures and anchor forces.

V.6. CONCLUSION

In this chapter, we built a detailed calculation framework for the sheet pile quay wall by pairing two very different software tools GEO5 and PLAXIS 2D and applying them to Quay G2. The two tools are wonderfully complementary: GEO5's sheet pile module uses a fast, code-friendly spring and P-Y-curve approach, ideal for rapid parametric studies and standard safety checks (Eurocode 7, French norms), while PLAXIS 2D is a full finite element package that simulates real nonlinear soil behaviour, staged construction, arching, and load transfer much richer physically but slower and more demanding on input data.

We fed both models the same geotechnical picture from the site investigation: a friction angle of 37.5°, no cohesion, unit weights of 16 kN/m³ above and 10 kN/m³ below water, and the same GU 6N sheet pile and M56/45 tie-rod layout. One practical difference emerged: to keep displacements within acceptable limits and maintain numerical stability, the PLAXIS model needed a slightly longer sheet pile (6.30 m instead of the initial 5.62 m) a point we'll revisit later.

For global stability, GEO5's Bishop-type slope analysis gave a safety factor of 1.54 for a slip surface passing below the pile toe, comfortably above the Eurocode 7 requirement of 1.5 for permanent works. So Quay G2 is safe against overall failure.

The real strength of this dual approach is that GEO5 lets us race through load combinations and construction stages to spot the critical ones, and PLAXIS then validates those findings with a far more realistic simulation. The two models agreed remarkably well: the anchor force was 43.8 kN/m in GEO5 and 43.9 kN/m in PLAXIS, and the bending moments were 43.35 kN·m/m versus 47.71 kN·m/m. That closeness gives us a lot of confidence in the design.

CHAPTER VI: RESULTS AND DISCUSSION

VI.1. INTRODUCTION

This chapter presents a comprehensive analysis and discussion of the results obtained from the various design methods and software applied to the sheet pile quay wall of the Algiers Marina Bay project. The objective is to compare, interpret, and critically evaluate the outcomes from:

- **Three analytical methods:** Simple Thrust (Blum), Sheet Pile Embedded, and Equivalent Beam methods
- **Two specialized software packages:** SPW 2006 (P-Y curve method) and PROSHEET (Free Earth Support)
- **Two finite element programs:** PLAXIS 2D and GEO5

The comparison focuses on four key design parameters: embedment depth, anchor force, maximum bending moment, and wall displacements. By examining the consistencies and discrepancies between these approaches, this chapter aims to provide practical recommendations for sheet pile wall design in similar geotechnical contexts.

VI.2. COMPARATIVE ANALYSIS OF DESIGN METHODS

VI.2.1. EMBEDMENT DEPTH AND TOTAL SHEET PILE LENGTH

Table 19: *Comparison of embedment depth and total length results*

Method / Software	Embedment Depth D (m)	Total Length (m)
Simple Thrust (Fs=2)	3.60	7.50
Sheet Pile Embedded (Fs=2)	1.07	4.97
Equivalent Beam	1.73	5.63
PROSHEET Free Earth	1.70	5.6
SPW 2006 (P-Y)	Not explicitly stated	6.3
GEO5 FEM	1.72	5.62
PLAXIS 2D	N/A (modeled)	6.3

VI.2.1.1. INTERPRETATION AND ANALYSIS

When comparing the results for total sheet pile length, two distinct groups emerge. The Equivalent Beam method, together with PROSHEET Free Earth and GEO5, converges on a value of approximately 5.60 m. These three approaches despite their different theoretical bases (limit equilibrium, Rowe's reduction, and subgrade reaction) agree remarkably well, with a maximum spread of only 0.03 m. From a purely rotational stability standpoint, this suggests that 5.60 m of total length is sufficient to prevent overturning about the anchor point.

In contrast, SPW 2006 and PLAXIS 2D both failed to reach a stable solution at 5.60 m. Iterative increases in length were required until convergence was achieved at 6.30 m. This discrepancy is not a sign that one group is wrong rather, it reflects a fundamental difference in what each method accounts for. SPW and PLAXIS explicitly model wall flexibility, soil continuity, and most importantly serviceability limit state (SLS) displacements.

So where does this leave the designer? For preliminary design tenders, feasibility studies, or initial cost estimates the 5.60 m result is perfectly acceptable. It satisfies ultimate limit state requirements and has been validated by three independent calculation methods. However, for final design the wall that will be built the additional 0.70 m of length (≈12.5% increase) is a prudent investment. It brings displacements within comfortable limits, avoids local soil failure at the toe, and provides a genuine safety margin against uncertainties like scour or construction tolerances.

The economic impact is modest. Assuming a GU 6N sheet pile cost of €150 to 200 per meter length per meter of wall, the extra 0.70 m adds roughly €105 to140 per linear meter. For the entire 800 m quay, this translates to €84,000 to €112,000 a small premium (10 to 15% of the sheet pile material cost) for the assurance that comes from advanced numerical modelling and satisfactory serviceability performance. **(ArcelorMittal, 2020; Global Market Share and Ranking, n.d.)**

VI.2.2. ANCHOR FORCE COMPARISON

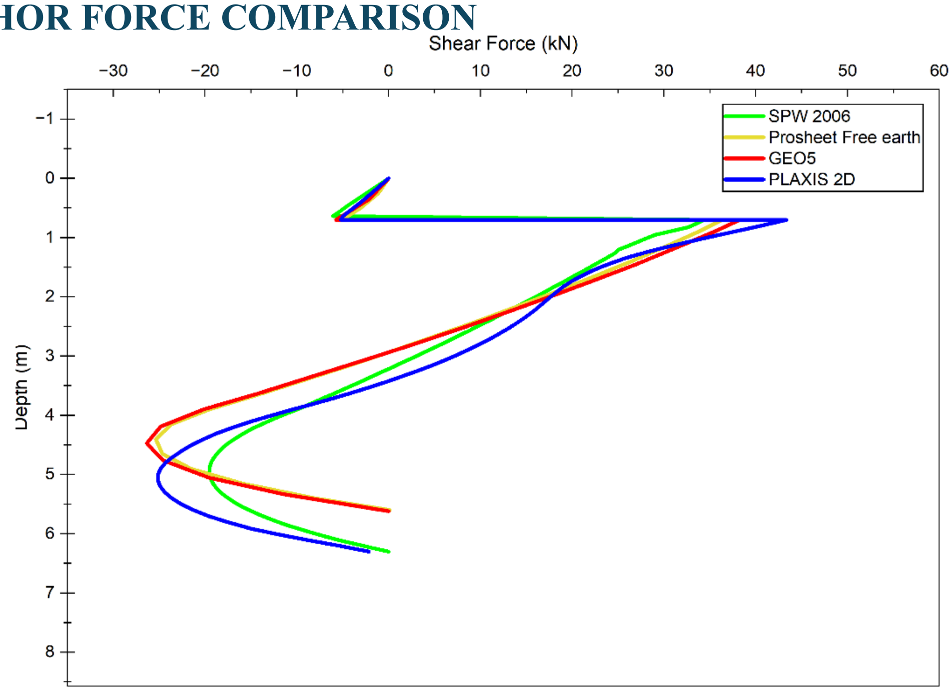

Figure 45: *Shear Force comparison (All Softwares)*

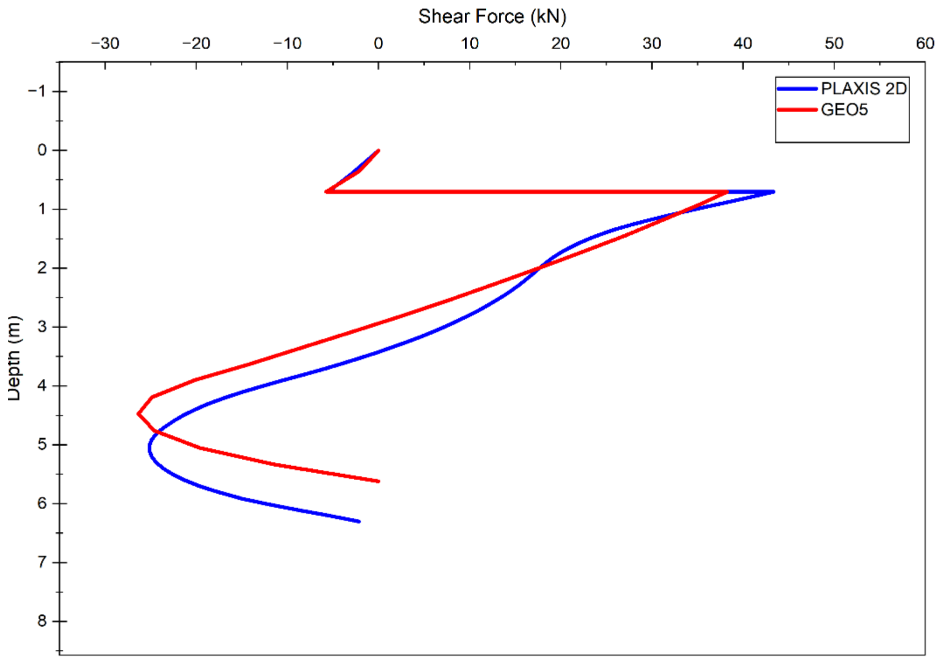

Figure 46: *Shear Force comparison (PLAXIS 2D and GEO5)*

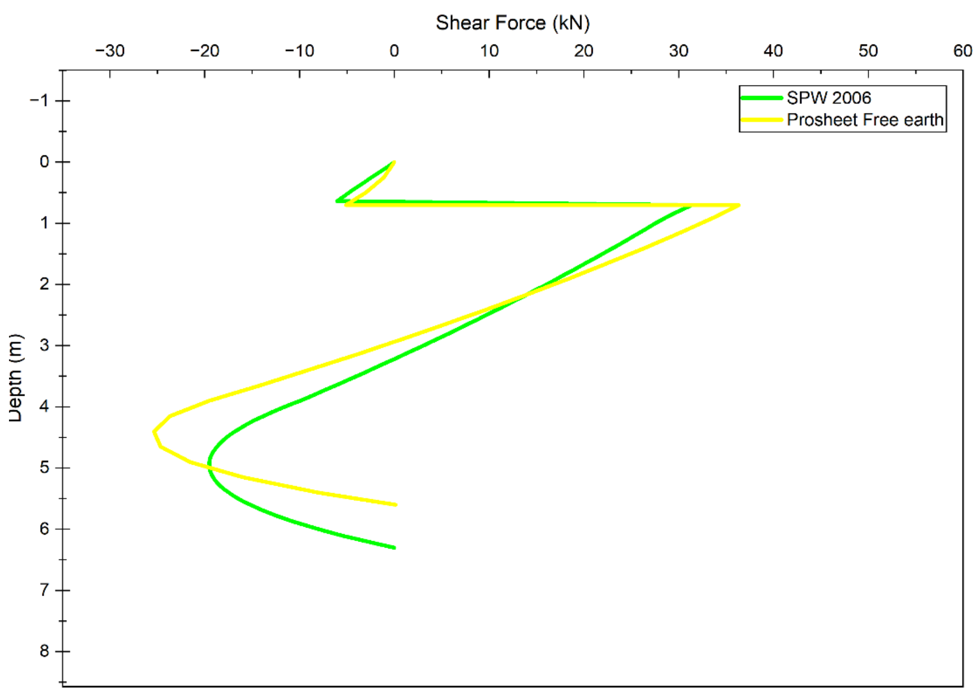

Figure 47: *Shear Force comparison (SPW and Prosheet)*

__Table 20:__ Comparison of anchor force results.

Method / Software	Anchor Force (kN/m)	Force per Tie Rod at 3.6 m spacing (kN)	Capacity M56/45 (kN)
Simple Thrust (Fs=2)	67.00	241.2	500
Equivalent Beam	42.68	153.6	500
PROSHEET Free Earth	41.4	149.04	500
SPW 2006	40.05	144.18	500
GEO5	43.80	157.7	500
PLAXIS 2D	43.34	156.02	500

VI.2.2.1.INTERPRETATION AND ANALYSIS

Figure 45: All software comparison

This figure presents a side by side bar chart of anchor forces. Two clear subgroups appear:

- **Lower group (40 to 41.5 kN/m):** PROSHEET Free Earth (41.4) and SPW 2006 (40.05). Both are based on the subgrade reaction (beam on elastic foundation) approach with nonlinear P-Y springs. Their close agreement suggests that the Winkler type modelling, which treats the soil as independent springs, tends to produce slightly lower anchor demands.
- **Higher group (43.3 to 43.8 kN/m):** GEO5 (43.80) and PLAXIS 2D (43.34). These two are nearly identical, despite their different mechanical formulations GEO5's sheet pile module also uses a subgrade reaction, while PLAXIS 2D employs a full finite element continuum (Hardening Soil model). The difference between the two groups is about **2.5 to 3.5 kN/m**, or roughly 6 to 8% of the mean. This is well within typical geotechnical uncertainty.

Figure 46: PLAXIS 2D vs GEO5

The almost perfect overlap of the two bars (43.8 vs 43.3 kN/m) is striking. It indicates that, for this particular geometry and soil profile (medium to dense sand, ϕ' = 37.5°, no cohesion), the anchor force is not sensitive to whether one models the soil as independent springs (GEO5) or as a continuous deformable medium (PLAXIS). The slight difference (0.5 kN/m) can be attributed to minor variations in the implementation of wall-soil friction and the way the anchor is tied to the wall.

Figure 47: SPW 2006 vs PROSHEET Free Earth (paired bar chart)

Both bars are low (40.05 and 41.4 kN/m). SPW gives the lowest of all four. PROSHEET is about 1.35 kN/m higher. This small discrepancy likely comes from differences in:

- the P-Y curve formulations
- the treatment of the water pressure distribution behind and in front of the wall.
- and the numerical tolerance for convergence.

Nevertheless, the values are close enough to be considered equivalent for practical design.

Summary for anchor force

All four software packages give anchor forces in the narrow range **40 to 44 kN/m**. The mean of the four is **42.15 kN/m**. For design purposes, a value of **42 kN/m** (rounded from the GEO5/PLAXIS average) is recommended. The tie rods (M56/45, 497 kN capacity) are safe with a minimum safety factor of **497 / (42 × 3.6) = 3.2**.

VI.2.3. BENDING MOMENT ANALYSIS

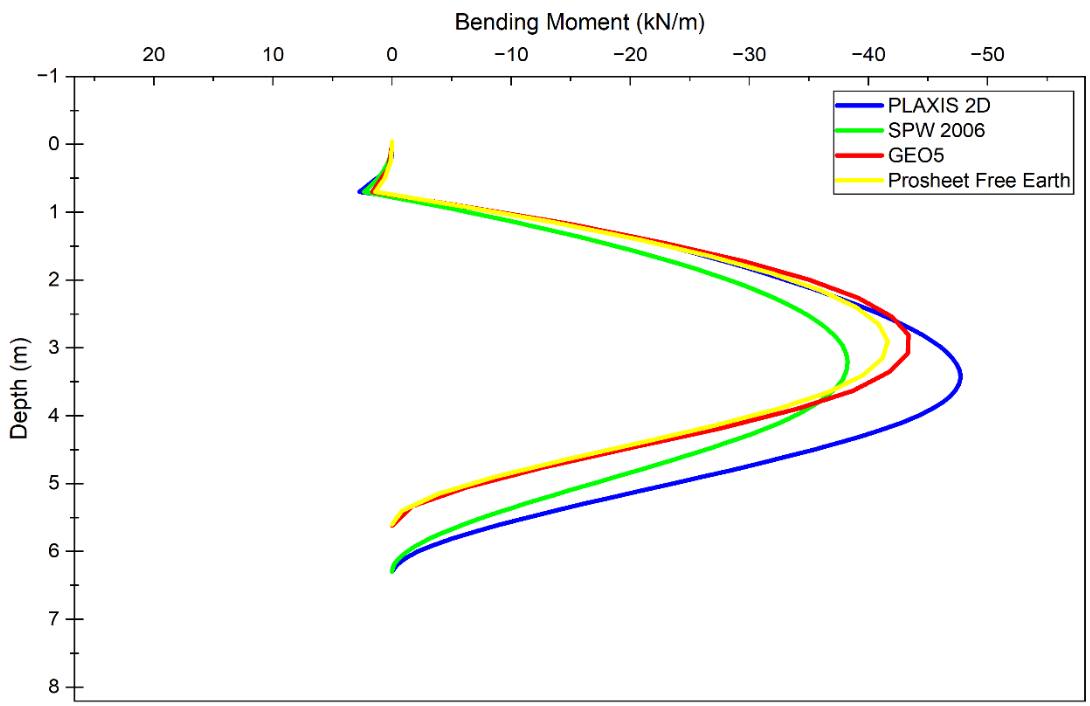

Figure 48: *Bending Moment Comparison (All Softwares)*

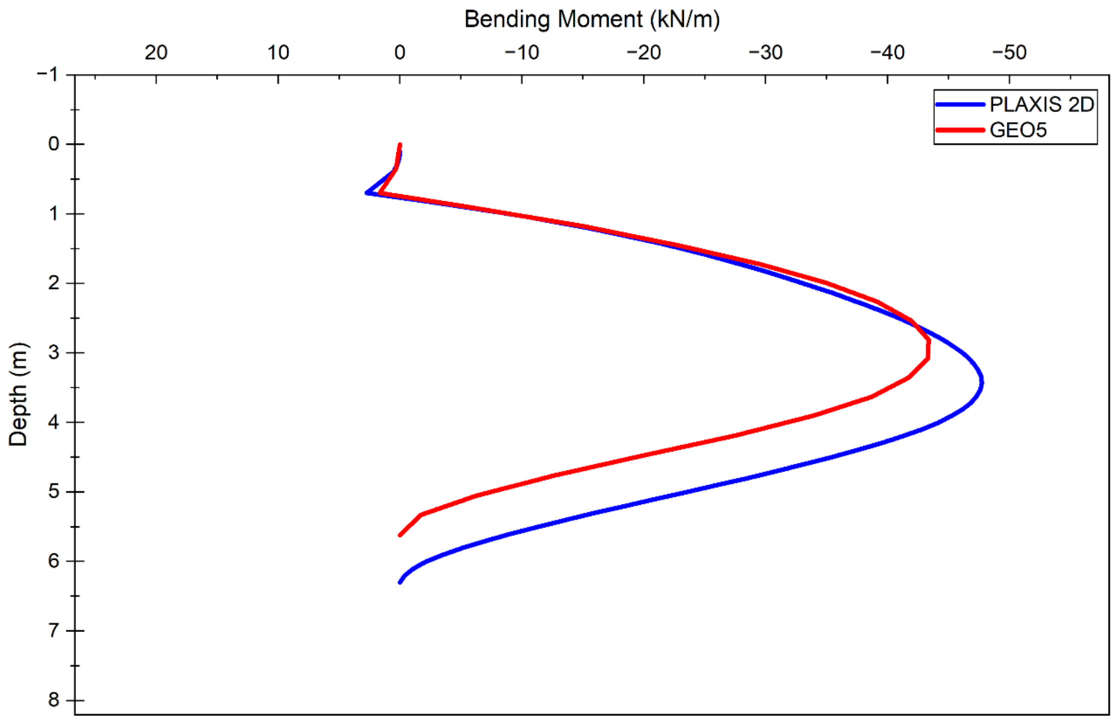

Figure 49: *Bending Moment Comparison (PLAXIS 2D and GEO5)*

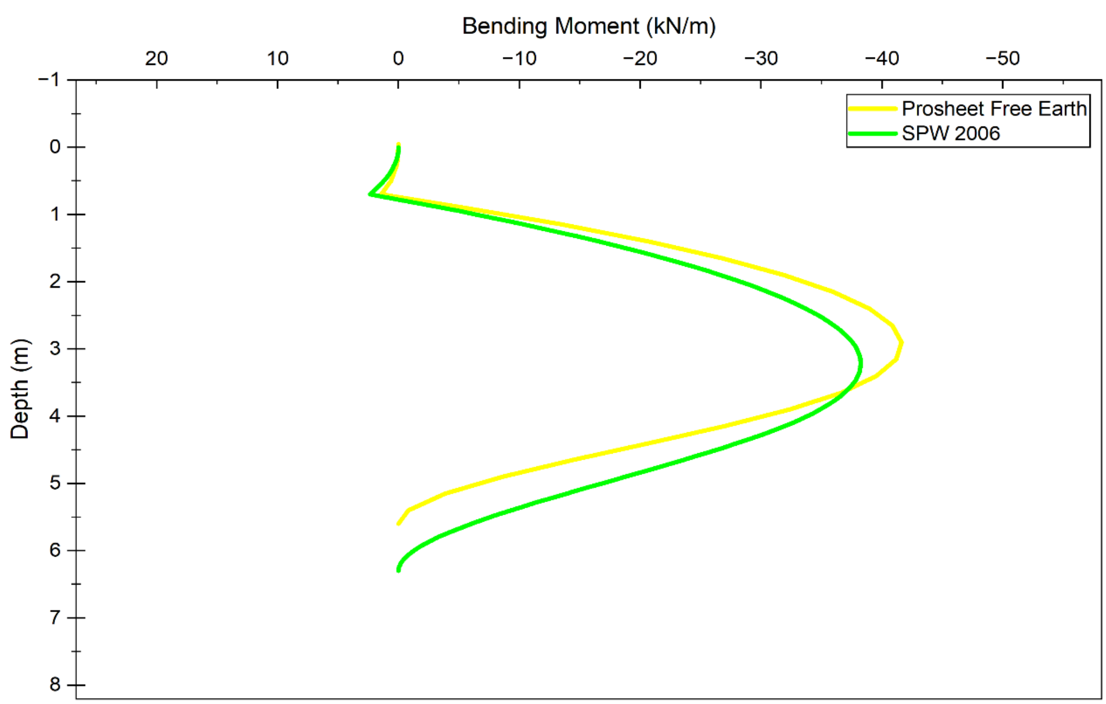

Figure 50: *Bending Moment Comparison (SPW and Prosheet)*

VI.2.3.1. INTERPRETATION AND ANALYSIS

The bending moment along the sheet pile wall is a critical design parameter. Figures VI.5 to VI.8 present the moment diagrams (bending moment vs. depth) computed by the four software packages.

Figure 47 – All software comparison (overlaid moment curves)

This figure superimposes the bending moment profiles from PROSHEET, SPW 2006, GEO5, and PLAXIS 2D. The following observations can be made:

- **Shape of the curves:** All four show a similar characteristic shape for an anchored sheet pile wall: a positive (hogging) moment in the upper part (above the dredge line), then a transition to negative (sagging) moment below the anchor, reaching a maximum negative value somewhere between the dredge line and the toe, and finally approaching zero at the toe.
- **Maximum absolute moment values:**

  PROSHEET Free Earth: = **41.62 kN·m/m**

  SPW 2006: = **40.05 kN·m/m**

  GEO5: = **43.35 kN·m/m**

  PLAXIS 2D: = **47.7 kN·m/m**

PLAXIS 2D gives the highest peak moment, about 20% higher than SPW and 12% higher than GEO5. This reflects the continuum model's ability to develop larger bending strains due to the full interaction between the wall and the surrounding soil mass.

- **Location of the maximum moment:** The depth of the maximum negative moment is consistently around **2.5 to 3.5 m below the top of the wall**.

Figure 48: PLAXIS 2D vs GEO5 (moment curves)

This pair of curves is particularly instructive. Despite using different mechanical formulations (continuum FEM vs. subgrade reaction), the two curves are very close in shape and magnitude. The maximum moments differ by only about 4 kN·m/m (47.7 vs 43.3 kN·m/m). The GEO5 curve is slightly smoother, while PLAXIS shows a more pronounced local curvature near the anchor level, which is typical of FEM due to the localised stress concentration at the anchor connection.

Figure 49: SPW 2006 vs PROSHEET Free Earth (moment curves)

Both curves are very similar, with SPW showing a marginally lower peak (40 vs 42 kN·m/m). The shapes are almost identical, indicating that the two beams on elastic foundation codes produce practically the same bending moment distribution when given the same input parameters. This is reassuring for engineers who may use either tool.

All computed moments (40 to 48 kN·m/m) are far below the design bending resistance of the GU 6N sheet pile, which was calculated in Chapter IV as **271.6 kN·m/m**. Even the highest value from PLAXIS (47.7 kN·m/m) represents only **18%** of the capacity. Therefore, the wall is **over designed in bending**. The governing design criterion will be **serviceability (displacements)**, not ultimate bending strength.

Nevertheless, if a more economical sheet pile section were to be considered (e.g., a smaller profile), the difference between SPW and PLAXIS would become critical. In such a case, the finite element result (PLAXIS) should be trusted as the more realistic estimate of the peak moment.

All software confirm that the GU 6N section is more than adequate. The choice of software does not affect the structural safety margin, but for refined designs, PLAXIS provides the highest (and likely most accurate) moment estimate.

VI.2.4. DISPLACMENT ANALYSIS

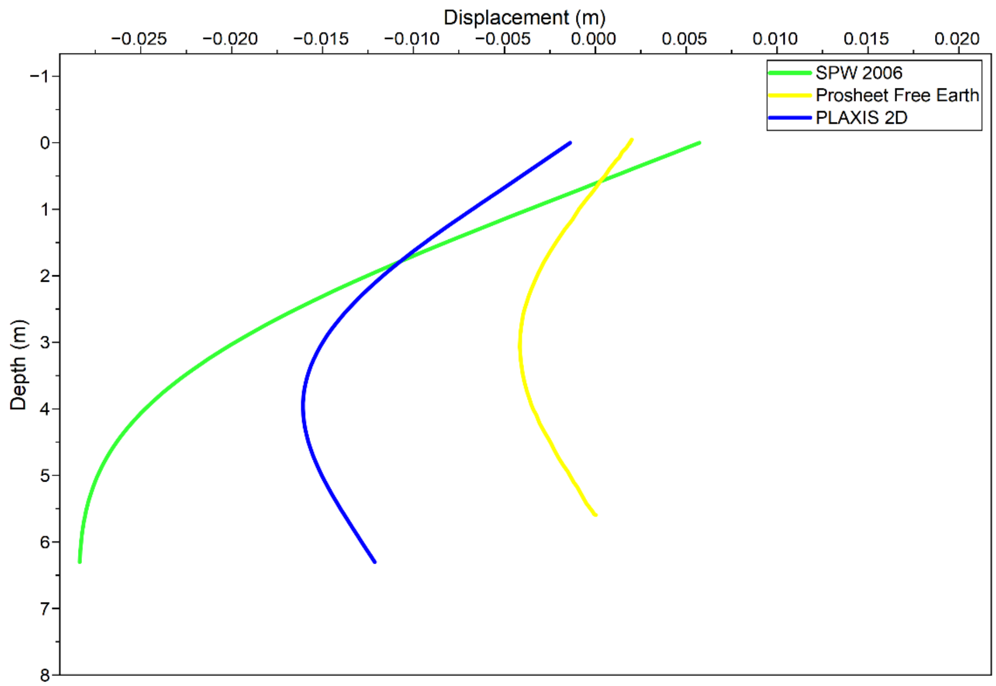

Figure 51: *Displacement Comparison (SPW and Prosheet and PLAXIS 2D)*

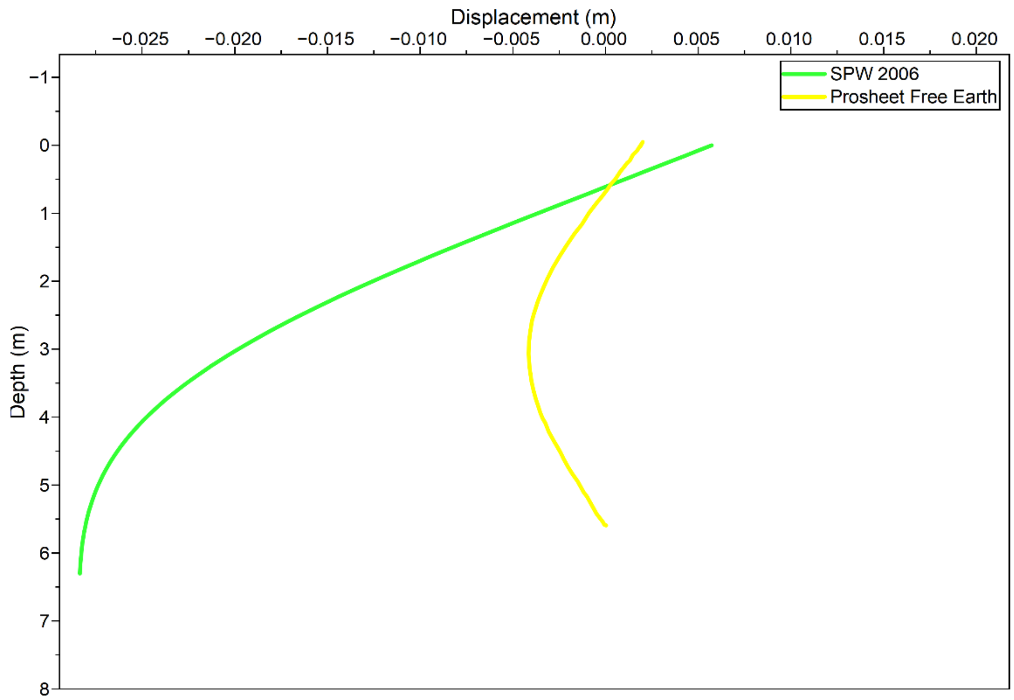

Figure 52: *Displacement Comparison (SPW and Prosheet)*

VI.2.4.1. INTERPRETATION AND ANALYSIS

Key results from software:

Table 21: *Comparison of displacement results.*

Software	Max displacement (mm)	Direction	Reliability
SPW 2006	28	Toward excavation	Conservative upper bound
PLAXIS 2D	16	Toward excavation	Most realistic
PROSHEET	8	Toward excavation	Overly stiff/unconservative
GEO5	-	Not computed	-

When it comes to predicting how much a sheet pile wall will actually move, the different software packages do not see eye to eye. The numbers range from as low as 5 mm to as high as 28 mm a spread that might cause anxiety for a designer. But once we understand what each model is doing, the picture becomes clearer, and we can make a reasoned choice.

- SPW 2006 gave the largest deflection: 28 mm toward the excavation side. Its P-Y springs (Delft University formulation) are relatively soft, and the toe is free to rotate so the wall bends more. Think of this as the cautious engineer's answer: safe, but perhaps a bit pessimistic.
- PLAXIS 2D our finite element model with Hardening Soil, produced 16 mm in the same direction. This is the most realistic estimate because the soil is treated as a continuous medium, not a set of independent springs. Stress redistribution and arching stiffen the response exactly what happens in the ground.
- PROSHEET Free Earth gave only 8 mm less than one third of SPW's value. Its springs (ArcelorMittal's defaults) appear much stiffer than those in SPW. While this is comforting, it is probably too optimistic for a sandy site like Marina Bay.
- GEO5 does not output displacement profiles at all, so it is absent from this part of the comparison.

After looking at the curves and the underlying physics, PLAXIS 2D's 16 mm emerges as the most credible. It respects soil continuity, uses a well-calibrated constitutive model, and avoids the arbitrary stiffness of springs. SPW's 28 mm remains a useful conservative upper bound if the wall can handle 28 mm, it will certainly handle 16 mm.

All predicted displacements sit within or very close to the typical serviceability limit for a marina quay (20 to 30 mm). The wall meets the SLS criteria without difficulty. The GU 6N section is not only strong enough (we saw that in the bending moment analysis), but also stiff enough to keep movements moderate. In short, the wall will not lean, the cranes will not complain, and the boats will berth safely.

If you are doing a quick preliminary design, PROSHEET Free Earth (8 mm) might give you a false sense of security. If you are doing a final check, trust PLAXIS. And if you want a simple conservative bound,

VI.3. CONCLUSION

The comparative study of analytical methods (Simple Thrust, Sheet Pile Embedded, Equivalent Beam) and numerical software (PROSHEET, SPW 2006, GEO5, PLAXIS 2D) applied to the sheet pile quay wall (Quay G2) of the Marina Bay of Algiers project leads to the following conclusions:

- **Embedment depth:** Three methods (Equivalent Beam, PROSHEET Free Earth, GEO5) converged on a total sheet pile length of about 5.60 m, which satisfies rotational (ultimate) stability. However, SPW 2006 and PLAXIS 2D required 6.30 m to achieve numerical stability and acceptable serviceability displacements. The additional 0.70 m is not a sign of error in the simpler methods, but a necessary adjustment to control wall flexibility, soil continuity, and SLS deflections. For preliminary design, 5.60 m is acceptable; for final design, 6.30 m is recommended.
- **Anchor force:** All four software packages gave anchor forces in the narrow band of 40 to 44 kN/m (mean =42 kN/m). The Equivalent Beam analytical method (42.68 kN/m) sits comfortably within this range. The M56/45 tie rods (capacity 497 kN per rod) are safe with a minimum safety factor of about 3.2. Anchor force is a robust parameter, insensitive to modelling details.
- **Bending moment:** The maximum bending moments computed by software range from 40 kN·m/m (SPW) to 47.7 kN·m/m (PLAXIS). These represent only 15 to 18% of the GU 6N section's design resistance

(271.6 kN·m/m). The wall is significantly over-dimensioned in bending; serviceability (displacements) is the governing criterion

- **Displacements:** SPW predicted the largest deflection (=28 mm), PLAXIS gave a more realistic 16 mm, while PROSHEET produced unconservative values (5 to 8 mm). All computed displacements are within typical marina quay limits (20 to 30 mm). PLAXIS 2D (continuum FEM) is considered the most reliable for SLS checks.

Overall recommendation

For the Marina Bay project, the GU 6N sheet pile section is more than adequate. A total length of 6.30 m (embedment =2.40 m below dredge) should be adopted for final design, with an anchor force of 42 kN/m (=151 kN per tie rod at 3.6 m spacing). The design is safe in all limit states, and the additional cost of the longer piles (€105 to 140 per linear metre) is justified by improved serviceability performance and robustness against uncertainties. **(ArcelorMittal, 2020; Global Market Share and Ranking, n.d.)**

GENERAL CONCLUSION

The present study was devoted to the analysis and design of a sheet pile quay wall for the Marina Bay of Algiers project, with a specific focus on the most loaded section, Quay G2 (water depth 3.00 m, Overload 10 kN/m²). The work aimed to model soil-structure interaction and to perform a comparative evaluation of different calculation methods, from classical analytical approaches to advanced numerical software, in order to assess their accuracy, limitations, and practical applicability.

achieve this objective, the thesis was structured into two main parts: a theoretical foundation covering port structures, sheet pile wall behaviour, earth pressure theories, design methods and relevant codes (Eurocode 7, RPA 99/2003, etc.), followed by a practical study comprising predimensioning using three analytical methods (Simple Thrust, Sheet Pile Embedded, Equivalent Beam) and detailed numerical modelling using four software packages (PROSHEET, SPW 2006, GEO5 and PLAXIS 2D). All calculations were based on a thorough geotechnical characterisation of the site (SPT and CPT tests), which led to a design friction angle of 37.5° for the medium dense sandy soil.

The comparison of the results led to the following main conclusions:

1. **Embedment depth:**

   The Equivalent Beam method, PROSHEET Free Earth and GEO5 converged on a total sheet pile length of approximately **5.60 m**, which satisfies rotational (ultimate) stability. However, SPW 2006 and PLAXIS 2D required **6.30 m** to achieve numerical stability and acceptable serviceability displacements. The additional 0.70 m (about 12.5 %) is not an error of the simpler methods but a necessary adjustment to account for wall flexibility, soil continuity and, above all, serviceability limit state (SLS) displacements. For preliminary design, 5.60 m is acceptable; for final design, **6.30 m** is recommended.

2. **Anchor force:**

   All four software packages gave anchor forces in the narrow range of **40 to 44 kN/m** (mean approximately 42 kN/m). The Equivalent Beam analytical method (42.68 kN/m) lies comfortably within this range. The M56/45 tie rods (capacity 497 kN per rod) are safe with a minimum safety factor of about **3.2**. Anchor force is remarkably insensitive to the modelling details, which reassures the designer.

3. **Bending moment:**

   The maximum bending moments from the software range from 40 kN·m/m (SPW) to 47.7 kN·m/m (PLAXIS). These represent only **15 to 18 %** of the GU 6N section's design resistance (271.6 kN·m/m). The wall is amply strong in bending; the governing design criterion is serviceability (displacements), not ultimate strength.

4. **Displacements: the true design driver.**

   SPW predicted the largest deflection (about 28 mm), PLAXIS gave a more realistic 16 mm, while PROSHEET produced unconservative values (5 to 8 mm). All computed displacements lie within typical marina quay limits (20 to 30 mm). PLAXIS 2D (continuum FEM with Hardening Soil) is considered the most reliable for SLS checks.

5. **Complementarity of analytical and numerical methods.**

   The classical Equivalent Beam method remains a valuable tool for rapid predimensioning, especially for anchor force estimation. However, for final design, numerical methods – particularly finite element analysis with PLAXIS 2D – are indispensable to capture wall flexibility, stress redistribution, and realistic serviceability behaviour. The combination of GEO5 (fast, code compliant) and PLAXIS 2D (detailed, physically rich) proved to be a powerful and efficient design strategy.

Recommendations for the Marina Bay project:

- Adopt a **total sheet pile length of 6.30 m** (embedment about 2.40 m below dredge level) for Quay G2.
- Use the **GU 6N** profile (S240 GP steel) with a design anchor force of **42 kN/m** (approximately 151 kN per tie rod at 3.6 m spacing).
- The existing M56/45 tie rods and the anchor wall are more than adequate.
- For similar future projects, it is recommended to start with the Equivalent Beam method for preliminary sizing and then validate the design using PLAXIS 2D (or another FEM code) to check serviceability displacements.

Contribution and perspectives.

This work provides a comparative reference for engineers in Algeria and elsewhere who are confronted with the design of anchored sheet pile quay walls in granular soils. It demonstrates the importance of using multiple methods and understanding their underlying assumptions. Future research could extend the comparison to three dimensional effects, seismic loading (using the Mononobe-Okabe method and RPA 99/2003), and the influence of cyclic berthing forces on long term displacements. A cost optimisation study could also be carried out to explore the possibility of using a lighter sheet pile section, guided by the PLAXIS displacement results.

In fine, the sheet pile quay wall for the Marina Bay of Algiers, as designed and verified through this multi-method approach, satisfies all ultimate and serviceability limit state requirements and represents a safe, durable and economically sound solution.

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