A numerical assessment of jetty side support system

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Abstract. Steel sheet piles are amongst the most common kinds of quay walls used in the construction of port and harbor facilities. These are used most often for the birth of small crafts that have limited dimensions and capacity. An increasing number of berths require upgrades because of the growing market in marine traffic around the world, as well as continuously increasing dimensions and vessel capacities. A number of methods can be used to increase the steel sheet pile wall's load-carrying capacity, including the use of double sheets with intermediate tie rods. This case study aims to evaluate the use of PLAXIS 3D to check system safety and to ensure sufficient capacity to withstand seismic design forces.

1. Introduction
Quay walls play crucial roles in the operational capacity of ports, marinas, shipyards, and other waterside facilities. In this context, steel sheet piles are among the most common quay wall types used in port construction. These structures are widely utilized in the construction of container and dry-bulk terminals and sea walls and reclamation projects. These structures are particularly useful in situations where the infill is needed seaward of the existing shore as well as for marinas and other structures where deep water is needed directly at the shore.

The design of waterfront-retaining structures to resist earthquakes also remains in flux. This is because the soils behind these structures often lack cohesion and are saturated due to relatively high-water tables. It is also possible that pore pressure build-up and associated liquefaction phenomena can occur as the firm ground shakes. Numerous examples of failure and unsatisfactory performance have, therefore, been reported.

One jetty located in Musaffah, Abu Dhabi, United Arab Emirates, has been expanded to meet owner requirements (Figure 1). An extensive study was conducted to understand this structure and check the compliance of existing sheet pile walls with Eurocode. Simultaneously, three-dimensional (3D) numerical modeling was used to simulate the side support system based on real site conditions.

Numerous previous authors have attempted to analyze and predict the load-carrying capacity of different steel sheet pile walls. Earlier research has addressed the behavior of steel sheet pile walls using grouted anchors and has predicted their carrying capacity taking into account the fact that they are flexible structures ([1]; [2]; [3]; [4]; [5]). Finite element (FE) techniques have also been applied to investigate structural behaviors and failure mechanisms ([6]; [7]; [8]; [9]; [10]).
Full-scale field tests have been used to investigate the behavior of sheet pile quay walls [11]. In one earlier study, [12] provided significant results to assess sheet pile walls’ failure mechanisms based on field observations. In contrast, additional anchored tie-rod grouting in the backfill and along walls were used by [13] and [14] to increase height.

A FE method was applied using the software PLAXIS to investigate the behavior of sheet pile walls and predict movement [15]. In another example, [16] performed a parametric study on enhancing steel-piling quay walls using grouted anchors. This study shows that steel sheet pile walls’ rehabilitation using additional grouting tie rods is significant in anchored quay wall performance. Anchored walls and surrounding soils also exhibit more stabilized behaviors when grouted anchors are used.

A range of projects ([17]; [18]; [19]; [20]) have assessed the earthquake performance of more than 100 anchored quay walls in 26 harbors in Japan, Alaska, the West Indies, and Chile. These studies indicate that waterfront structures and, in particular, anchored quay walls are highly susceptible to earthquake-induced damage. The bulk of anchored quay wall failures are also known to have occurred in locations where liquefaction has not been observed visually ([21]; [22]).

An advanced seismic assessment of anchored quay walls was performed by [23] using numerical analysis. This study highlighted a range of numerical modeling techniques based on a performance-based design approach.

## 2. 3D numerical modeling

A range of 3D numerical modeling approaches has been developed to check jetty side support system safety and to ensure sufficient capacity to withstand seismic design force. In this context, the software PLAXIS 3D was used to simulate real conditions in the case study and simulate different soil-structural interactions.

The retaining structure assessed here is composed of three different sections (Figure 2). Specifically, Section A within this structure extends for 48.4 m and has a depth of 14 m; the sheet pile section used here is denoted PU32. In contrast, Section B is the first length extension of this structure, 35.21 m in length, extending to a depth of 14 m; the sheet pile section used here is denoted GU16-400. The second extension of this structure is Section C; this section is 27.02 m in length, extends to a depth of 12 m, and utilizes the SX18 sheet pile section.

### 2.1 Model geometry

The general configuration of the model applied here is shown in Figure 3, while the tie rods that connect the front sheet pile wall to the back-sheet one and the bollard are shown in Figure 4. In this case, the soil mass is hidden in the model and configuration; thus, GWT is encountered at 2 m depth from the natural ground surface, as shown in the ground profile (Figure 5).
2.2. Model material properties
Based on on-site ground investigations and laboratory and field test results, interpreted parameters for different soil and rock layers are listed in Table 1. The sequence of ground layers is shown in Figure 5.
Figure 4. The connections between the front and back-sheet pile walls; (a) model simulation; (b) sheet pile wall sections

Figure 5. Ground profile summary
Table 1. Soil and rock properties

| Soil type | Medium dense sand | Mudstone | Gypsum |
|-----------|-------------------|----------|--------|
| Depth (m) | (0.0) to (-9.0)   | (-9.0) to (-15.3) | (-15.3) to (-40.0) |
| Constitutive model | Hardening soil | Hardening soil | Hardening soil |
| \( \gamma_{\text{sat}} \) (kN/m\(^3\)) | 16.00 | 22.00 | 22.00 |
| \( E_{\text{ref}} \) (kPa) | 35,000 | 440,000 | 1,690,000 |
| \( E_{\text{ref}} \) (kPa) | 35,000 | 440,000 | 1,690,000 |
| \( E_{\text{ref}} \) (Pa) | 105,000 | 1,320,000 | 5,070,000 |
| Stress power (m) | 0.50 | 0.50 | 0.50 |
| \( C \) (kPa) | 10 | 180 | 350 |
| \( \varphi \) (degree) | 35 | 34 | 46 |
| \( \Psi \) (degree) | 5 | 4 | 16 |
| \( \nu_{\text{ur}} \) | 0.25 | 0.20 | 0.20 |

Table 2. Interpreted and assigned sheet pile wall properties

| Sheet pile section | PU32 | GU16-400 | SX18 |
|--------------------|------|----------|------|
| Section            | A    | B        | C    |
| Equivalent height (m) | 0.452 | 0.29 | 0.36 |
| \( \gamma \) (kN/m\(^3\)) | 4.18 | 5.31 | 3.74 |
| \( E_1 \) (KN/m\(^2\)) | 1.88 x 10\(^7\) | 2.22 x 10\(^7\) | 1.67 x 10\(^7\) |
| \( E_2 \) (KN/m\(^2\)) | 9.39 x 10\(^5\) | 1.11 x 10\(^6\) | 8.33 x 10\(^5\) |
| \( \nu_{12} \) | 0 | 0 | 0 |
| \( G_{12} \) (KN/m\(^2\)) | 9.39 x 10\(^5\) | 1.11 x 10\(^6\) | 8.33 x 10\(^5\) |
| \( G_{13} \) (KN/m\(^2\)) | 1.79 x 10\(^6\) | 2.27 x 10\(^6\) | 1.6 x 10\(^6\) |
| \( G_{23} \) (KN/m\(^2\)) | 5.36 x 10\(^5\) | 6.8 x 10\(^5\) | 4.8 x 10\(^5\) |

The properties of medium dense sand soils are based on SPT values. Mudstone and gypsum rocks were simulated as equivalent masses by applying Hock-Brown failure criteria. Simultaneously, the software Roclab was used to interpret equivalent properties based on unconfined compression strength values.

Sheet pile walls with three different sections were simulated as plate elements. The basic properties of each sheet pile wall are presented in Table 2, while the basic properties of utilized sheet pile walls are presented in Table 3. Figure 6 shows the properties assigned to different pile walls in the PLAXIS model to simulate their anisotropic behavior.
Table 3. Sheet pile wall basic properties

| Sheet pile type | PU32 | GU16–400 | SX18 |
|-----------------|------|----------|------|
| Section         | A    | B        | C    |
| Length (m)      | 48.40| 35.21    | 27.02|
| Depth (m)       | 14.00| 14.00    | 12.00|
| $\gamma_{steel}$ (kN/m$^3$) | 78.00| 78.00    | 78.00|
| $E_{steel}$ (kPa) | $2 \times 10^8$ | $2 \times 10^8$ | $2 \times 10^8$ |

| Sectional area of wall meter length (m$^2$) | 242.3 x 10$^{-4}$ | 197.3 x 10$^{-4}$ | 173 x 10$^{-4}$ |
| Moment of inertia for wall meter length (m$^4$) | 72,320 x 10$^{-8}$ | 22,580 x 10$^{-8}$ | 32,400 x 10$^{-8}$ |
| Section height (m) | 0.452 | 0.290    | 0.360 |

A total of 12 bollards were simulated as embedded beams. The cross-section of these bollards comprises hollow circular sections, 0.3556 m in outer diameter, and a thickness of 0.00952 m. The Young’s modulus of steel is $2 \times 10^8$ kN/m$^2$, while the unit weight is 78 kN/m$^2$. Inertial properties were automatically calculated using the software PLAXIS based on hollow circular tube sectional properties. Two different cross-sections of tie rods were used in the PLAXIS 3D model according to the proposed design; these were measured as tie rods 90 mm and 50 mm in diameter. These steel rods have Young’s modulus values of $2 \times 10^8$ kN/m$^2$; The EA of the pile 90 mm in diameter is $1.272 \times 10^6$ kN, while that of the 50 mm diameter rod is $392.7 \times 10^3$ kN.

Two types of loads were assigned in this model to account for proposed jetty loadings. The first type of load assessed here is the area (surface load) represented by the dead and live load on the jetty and was assigned a value of 50 kPa. Similarly, a 150 kPa of surface (area load) accounts for loads at crane locations. The second type of load assessed is exerted onto the bollard due to ship berthing; this was assigned as a point load at each bollard and equal to 100 kN in each direction, $x$, $y$, and $z$.

Surface load simulations are summarized in Figure 7. In this depiction, the red area with small vertical arrows represents the 50 kPa surface load, while the big vertical arrow represents the 150 kPa surface load of the crane acting on a 3.0 m x 8.0 m area. The blue arrows in Figure 8 express the resultant point loads of 100 kN acting in each direction on the bollards.

![Configuration of surface loads](image)
2.3. Stages in model construction

- The initial phase: Initial stress was generated using the $K_0$ procedure.
- Phase 1 (excavation and activating all structures): Soil was excavated from the beach, and all structures were activated, including sheet pile walls, tie rods, and embedded bollard beams.
- Phase 2 (activating loads): All loads were activated in this phase, including two surface loads of 50 kPa for dead and live loads, 150 kPa at crane loads, and bollard forces, which were defined as point loads on all bollards as 100 kN longitudinal, 100 kN transverse, and 100 kN pulling up forces.
- Phase 3 (safety calculation factors): The C-Phi reduction for calculating safety factors was activated in this phase.
- The different model construction phases are shown in Figure 9, including the initial stage (Figure 9a), phase 1 (Figure 9b), and phase 3 (Figure 9c).

2.4. Model analysis and results

The global safety factor of this model was 4.90. The maximum settlement value of 3.4 cm was observed underneath the crane, while the maximum lateral deflection of sheet piles was 2.9 cm. Figure 10 and Figure 11 show that the maximum total displacement of 3.4 cm occurs underneath the crane. Similarly, Figure 12 shows that the maximum value of vertical displacement ($U_z$) in the model is 3.3 cm.
Figure 9. Model construction stages

Figure 10. Total deformation of geometry and meshing

Figure 11. Colored contours of total displacement \([U]\) in the model depicted for 3D mass
The side support system comprises three walls with different cross-sections. The summary in Figure 13 summarizes wall vertical displacements: 1.9 cm, 3.2 cm, and 2.1 cm for wall 1, wall 2, and wall 3. The summary in Figure 14 depicts total wall displacements; these are 2.1 cm, 3.3 cm, and 2.9 cm for wall 1, wall 2, and wall 3. The summary in Figure 15 depicts lateral wall displacements: 2.09 cm, 1.80 cm, and 2.90 cm for wall 1, wall 2, and wall 3.
3. Seismic analysis

Evaluation of structural stability concerning earthquakes must be considered, in particular excessive movements. The damaging effects of earthquakes are essentially, but not exclusively, the result of horizontal oscillatory ground accelerations being transferred to structures above ground level through foundations, bases, and pile support. The response of a structure to these accelerations depends upon the type, mass, dimensions, and failure modes. It is, therefore, crucial to select appropriate structures in seismically active areas.

[25] (Section 1653) and [26] categorized Abu Dhabi as Zone 0; this means that no seismic effects are considered in the structures' design. However, according to the Abu Dhabi Municipality, structural engineering submissions should be based on seismic loading, corresponding to a peak ground acceleration of 0.15 g for Zone 2A defined in [26] for a 50-year design life. This code states that there is a 10% excess probability during the life of the facility. A ground acceleration of 0.20 g has also been estimated in reclaimed soils to account for amplification.

An earthquake was simulated based on the acceleration time-history as the base of this model. Thus, typical seismic load values were expressed in terms of peak ground acceleration (PGA). The ground motion was normalized to a target peak acceleration of 0.15 g, and the earthquake duration was set at 31.2 seconds (Figure 16). The response of this system due to seismic excitation was then studied.
The data in Figure 17 illustrates the maximum model vertical displacement considering seismic loads. The maximum value of Uz in this model equals 3.3 cm; this analysis shows that the seismic loads caused by earthquakes do not cause any structural damage.

4. Conclusion

A 3D numerical analysis was conducted using the software PLAXIS 3D to check this system's global safety and to ensure that deformations and straining actions of walls fell within an acceptable range. Structural stability was also evaluated when subjected to seismic loads.

The key numerical model findings of this study are:

- The global safety factor in this model is 4.9, and so the system is safe;
- The total value of wall displacements at the most critical section ranges between 2.1 cm and 3.3 cm, within an acceptable range, and;
- The seismic analysis shows that the loads caused by earthquakes do not cause any structural damage.
References

[1] Cherubini C (2000) Probabilistic approach to the design of anchored sheet pile walls. *Computers and Geotechnics* 26: 309–330.

[2] Endley S, Dunlap W, Knuckey D and Sreerama K (2000) Performance of an anchored sheet-pile wall. Geotechnical Measurements: Lab and Field xx: 179–197.

[3] Macnab A (2002) Earth Retention Systems Handbook, McGraw Hill Professional.

[4] Rym郑za B and Sahajda K (2008) Static analysis of restrained sheet-pile walls. *11th Baltic Sea Geotechnical Conference*.

[5] Schriver A and Valsangkar A (1996) Anchor rod forces, and maximum bending moments in sheet pile walls using the factored strength approach. *Canadian Geotechnical Journal* 33(5): 815–821.

[6] Briaud J and Lim Y (1999) Tieback walls in sand: numerical simulation and design implications, *Journal of Geotechnical and Geoenvironmental Engineering* 125(2): 101–110.

[7] Don C and Warrington P (2007) Anchored sheet pile wall analysis using a fixed end method without estimation of point of contra flexure. *Vulcanhammer Info* xx: 1–27.

[8] Krabbenhoft K, Damkilde L, and Krabbenhoft S (2003) A general nonlinear optimization algorithm for lower bound limit analysis. *International Journal for Numerical Methods in Engineering* 56(2): 165–184.

[9] Krabbenhoft K, Damkilde L, and Krabbenhoft S (2005) Ultimate limit state design of sheet pile walls by finite elements and nonlinear programming. *Computers & Structures* 83(4–5): 383–393.

[10] Lyamin A and Sloan S (2002) Lower bound limit analysis using nonlinear programming. *International Journal for Numerical Methods in Engineering* 55(5): 573–611.

[11] Briaud J, Nicholson P, and Lee J (2000) Behavior of full-scale VERT wall in sand. *Journal of Geotechnical and Geoenvironmental Engineering* 126(9): 808–818.

[12] Barley A (1997) Failure of a 21-year-old anchored sheet pile quay wall. *Thames Ground Engineering* 30: 42–45.

[13] Ebeling R, Azene M, and Strom R (2002) Simplified procedures for the design of tall, flexible anchored tieback walls. *US Army Corps of Engineers*, Engineer Research, and Development Center.

[14] Strom R W and Ebeling R M (2001) State of the practice in the design of tall, stiff, and flexible tieback retaining walls (No. ERDC/ITL–TR-01-1), Engineer Research and Development Center, Vicksburg MS Information Technology Lab.

[15] Brinkgreve R, Swolfs W and Engine E (2002) Plaxis user’s manual, Balkema, Rotterdam, Netherlands.

[16] El-Naggar M (2010) Enhancement of steel sheet pile quay walls using grouted anchors. *Journal of Soil Science and Environmental Management* 1(4): 69–76.

[17] Duke C and Leeds D (1963) Responses of soils, foundations, and Earth structures to the Chilean earthquakes of 1960. *Bulletin of the Seismological Society of America* 53(2): 309–357.

[18] Hayashi S and Katayama T (1970) Damage to harbor structures by the Tokachioki earthquake. *Soils and Foundations* 10(2): 83–102.

[19] Hung S and Werner S (1982) An assessment of earthquake response characteristics and design procedures for port and harbor facilities. *3rd International Earthquake Microzonation Conference*
[20] Kitajima S (1978) Analysis of seismic damage in anchored sheet pile bulkheads. *Port and Harbor Research Institute* **18**(1): 67–130.

[21] McCullough N (1998) The seismic vulnerability of sheet pile walls. Department Of Civil, Construction, and Environmental Engineering, Oregon State University.

[22] McCullough N, Dickenson S and Pizzimenti P (2001) The seismic modeling of sheet pile bulkheads for waterfront applications, 2nd FLAC Symposium, Lyon, France.

[23] Ebrahimian B (2009) Seismic performance of anchored quay walls and numerical simulation techniques, *Performance-Based Design in Earthquake Geotechnical Engineering*, Kokusho, Tsukamoto & Yoshimine.

[24] PLAXIS material models manual 2018.

[25] Uniform Building Code (1997) International Conference of Building Officials, Whittier, CA.

[26] Frank R (2004) Designers’ Guide to EN 1997-1 Eurocode 7: Geotechnical design-General rules (Vol. 17). Thomas Telford.