A Design Procedure for Anchors of Floating Ocean Current Turbines on Weak Rock

Francisco Bañuelos-García 1,*, Michael Ring 2, Edgar Mendoza 2 and Rodolfo Silva 2

1 Facultad de Ingeniería, Universidad Autónoma del Estado de México, Cerro de Coatepec S/N, Ciudad Universitaria, Toluca 50110, Mexico
2 Instituto de Ingeniería, Universidad Nacional Autónoma de México, Coyoacán 04510, Mexico; mring@iingen.unam.mx (M.R.); emendozab@iingen.unam.mx (E.M.); rsilvac@iingen.unam.mx (R.S.)

Abstract: In recent years, ocean current turbines have proven to be a reliable device for renewable energy generation. A crucial element of these turbines are the foundations, since they limit the displacement of the turbine, which is key in achieving efficiency in energy conversion, and can account for up to 26% of the total cost of the project. Most design procedures for foundations focus on sandy and clayey soils, but rock soils often predominate in tropical locations where marine currents are suitable for the installation of this type of turbine. This paper presents a design procedure for steel pile anchors (PAs) and concrete dead weight anchors (DWAs) on weak rock soils, using the assumptions of current technical documents and design codes commonly used in the industry for marine structures. Using specific designs for PA and DWA anchors, the procedure was theoretically assessed for a site off Cozumel Island, Mexico. The results show that the dimensions needed for DWAs are substantially larger than those for PAs. Therefore, whenever drilling is economically and operatively possible, piles would be preferable for the foundations of current turbine systems.

Keywords: ocean current turbine; foundation design; dead weight anchor; pile anchor; limestone seabed; design procedure

1. Introduction

In the present search for more sustainable forms of energy, ocean currents offer a significant and relatively predictable source of power, for conversion into electricity. Ocean currents have approximately seven times the energy density of wind currents. However, as they are installed in a more demanding environment, biophysically, these technologies are less developed. Turbines capable of exploiting those currents are usually submerged structures, and so are a more attractive alternative in locations where aesthetics, hurricane risk and navigation are a constraint. To attain maximum efficiency, these turbines require an adequate foundation; otherwise, displacement may occur, resulting in malfunction, or even the loss of the device. An informed choice of foundation is important, since it accounts for 10–26% of the total cost of a project [1,2].

An ocean current turbine can be a fixed, or floating, device. The first is a rigid or semi-rigid structure, which does not move from its installation site. This system does not require any additional elements connected to the turbine and is suitable in shallow depths of 20 to 60 m [3,4]. The installation of this type of foundation can be expensive and its maintenance may be unaffordable [5]. To overcome these shortcomings, floating systems have been developed, consisting of a turbine on a floating platform which is fastened to an anchor by means of synthetic ropes, cables, or chains. The main advantage of floating systems is that they can be installed in depths greater than 60 m [3].

The anchors that are in floating systems of marine current turbines are fluke, pile, deadweight, and plate structures (see Figure 1). Fluke (or drag embedded) anchors are installed by pulling on the anchor line and dragging it along the sea floor. Eventually, the
fluke will penetrate the sea floor and the anchor will be embedded. Pile anchors (PA) are installed either by hammering them into the ground, or by drilling and grouting them in place, depending on the nature of the sea floor. Specific designs for marine rock anchors have been presented by various companies. Marine Micropile Technology Ltd. (Dublin, Ireland) developed drilled and grouted pile anchors for use in offshore projects to reduce the cost for foundations [6], based on the technology used at onshore construction sites. The installation uses a simultaneous drilling and grouting process, made possible by a special pile tip that serves as both a drill bit and a nozzle for the grout. Another proprietary rock anchor is presented in [7], in which the anchor and the drill bit are present in the same unit. After installing the anchor in the rock, parts of it are expanded and pressed against the bore hole by tensioning specific parts of the anchor. Both of these methods are well developed at present, but very little information is given as to the underlying calculations and design choices required in the earlier stages of a project. On the other hand, deadweight anchors (DWAs) are simply placed on the sea floor and hold the turbine in place by their sheer weight and friction with the sea floor. Plate anchors can be installed using various methods, all requiring a metal rod or tube to push the anchor to the desired depth below the seabed and then rotate them to their final orientation [8].

![Figure 1. Types of foundations for ocean current turbine anchorage. (a) Fluke; (b) Pile; (c) Deadweight; (d) Plate.](image)

The most relevant aspects to be considered in selecting an anchor type are:
- Local water depth;
- Type of seabed;
- Installation complexity;
- Characteristics of the turbine;
- Transportation of the system components.

Although initially the factor deciding the foundation type is the water depth, the soil properties are the most important element to consider [4]. Depending on the type of sea floor; fluke or plate anchors would be very complicated, even unfeasible, on a rocky seabed [8].

In Mexico, the most suitable site for the installation of an electric power generation plant based on ocean current turbines is off Cozumel Island in the Mexican Caribbean [9]. The current there is part of the ocean and wind-driven circulation system and so is persistent all year round in direction and magnitude. Recent studies show the hydrokinetic energy at this location offers great potential, both in shallow and deep water [10,11]. Unlike the sea floor in most parts of the world, the seabed of the Mexican Caribbean is predominantly...
limestone, covered with a thin layer of finer sediment [12], something relatively common in tropical and subtropical zones. The most suitable anchors for this type of soil are deadweight (DWA) and pile anchors (PA).

While design methods for anchorage in clay and sandy soils are widely available [8], there are fewer generic design procedures for DWAs and PAs for offshore structures. Taylor [13] proposed a practical, up-to-date guide that enables an engineer to select the type and size of anchor needed, including direct-embedment anchors, DWAs, and drag-embedment anchors. Based on this work, VanZwieten et al. [14] proposed a preliminary design process for estimating the geometry and required weight of the anchor. These parameters are calculated using the components of the design forces, and the specific weight of the material, which are not difficult to obtain. Technical documents such as: DNV-OS-J101 [15] and API RP2A-WSD [16] are also available, which give guidelines for design. In the case of PAs, various design procedures have been developed for offshore structures [3,17–22], and there are technical design documents for these foundations, such as API RP 2A-LRFD [16] and DNV-OS-J101 [15]. The design procedures are comprehensive, proven, and almost identical, but they are only applicable for clayey and sandy soils. However, there are potential sites, such as Cozumel, where ocean current turbines are feasible and the predominant soil is mainly rocky.

Weak rock often has complex mechanical behavior, mostly dis-homogeneous and anisotropic, due to the presence of defects or pre-existing discontinuities [23], and is considered a mid-point between soil and hard rocky spoils. This means a specific design procedure is needed, which considers its geotechnical properties [24]. The behavior of weak rock is related to its secondary structure, that is, the spacing and thickness of the soil-filled cracks and joints, and its compressive strength [25]. Therefore, placing foundations on weak rock may result in greater displacements and deformations; specific calculation procedures are needed. There are various procedures for the design of PAs in weak rock; for example, Goodman [26], Abbs [27], Carter and Kulhawy [28], Gannon et al. [29], Zhang et al. [30], Irvine et al. [31]; but all of these are related to onshore structures. Therefore, existing design methods must be adapted, since anchors in or on rock behave significantly differently to anchors in or on sandy or clayey soils. Despite this, to the best of the authors’ knowledge, there has been no research into the comprehensive design of ocean current turbine anchors on weak rock.

This paper presents a design procedure for ocean turbine foundations, using steel piles (PA) or concrete deadweight anchors (DWAs) on weak rock. It is based on the hypotheses of technical documents such as: API RP2A-WSD [16], DNV-OS-J101 [15], IEC 62600-10 [32], as well as the most commonly used procedures for the design of DWAs and PAs in weak rock [14,25–31,33].

The main dimensions of the anchor are determined without considering specific details, such as the connection between the pad eye and the anchor. The proposed methodology involves three main steps:

(i) Determining the preliminary design of the anchor (geometry);
(ii) Evaluating the structural capacity of the anchor; and
(iii) Computing the geotechnical capacity of the anchor on rock.

From these results, the most appropriate anchor design is selected.

The application of the proposed procedure is applied to an ocean current turbine site in the Cozumel channel, Mexico. The article is organized as follows: In Section 2 the procedure proposed is presented, divided into ultimate states, services, and fatigue limit designs. Section 3 gives an application example, as well as design results, and Section 4 presents the conclusions of the work.

2. Proposed Design Procedure

In this section, a design procedure for DWA and PA on weak rock is presented, which can be extrapolated for anchoring systems. A flow chart of the design procedure is shown
in Figure 2. The procedure consists of satisfying the three limit states (see Figure 3) for the design of offshore foundations/anchors.

**Figure 2.** Flow chart of the proposed design procedure.
Figure 3. Examples of limit states of pile anchors for ocean current turbines, (a) ULS: total pile uplift; (b) ULS: formation of a plastic hinge in steel piles; (c) SLS: malfunction derived from a partial pile uplift; (d) SLS: security risk due to anchor settlement.

The limit states for the design of ocean current turbine anchors are very similar to those of wind turbines [3,17]. The ultimate limit state (ULS) corresponds to the failure of the anchor when its maximum load resistance is exceeded. This may occur due to factors such as:

- Static equilibrium of the anchor is lost (whole or part), with a consequent failure mechanism, such as overturning, sliding, uplift. The case of a total pile uplift is shown in Figure 3a.
- Structural resistance is lost, leading to excessive yielding and the formation of a plastic hinge or cracks in concrete elements. Figure 3b shows a steel pile with a plastic hinge.
- Component fracture (for example, in the connections).

The service limit state (SLS) corresponds to the tolerance criteria of the anchor response to environmental loads being exceeded. This could include:

- Excessive displacements or rotations, leading to total, or partial, malfunction of the turbine. In Figure 3c, the pile has not moved completely, but the device will no longer generate electricity.
- Excessive vibration, limiting the performance of non-structural components.
- Differential settlements of the anchor, generating excessive rotation of the system. In Figure 3d, the back-and-forth movement of the pile lessens the stability and safety of the turbine.

The fatigue limit state (FLS) corresponds to long-term cyclic stresses and strains, which lead to failure of the anchor. These may be derived from repetitive loads or dynamic actions (i.e., wave loads, low frequency currents) acting on the anchor during its lifetime.

The ULS design has three phases, the first, preliminary design uses practical Equations for ordinary structures, or the designer’s experience. Secondly, the capacity of the pre-designed structure is evaluated. Then, in the third phase, the geotechnical capacity of the anchor on rock is evaluated. In the SLS design, if required, the displacements and rotations of the structure are determined and compared with the maximum values allowed, based on research, technical documents, and design codes. Finally, in the FLS design, the cumulative damage to the anchor, derived from repeated loads or dynamic actions using S–N curves,
DNV-RP-C203 [34] for steel structures, and DNV-OS-C502 [35] for concrete structures, is determined.

2.1. Ultimate Limit State Design

The ULS must be fulfilled in accordance with the recommendations of existing design codes and technical documents for this type of structure, for example, API, DNV/DNV GL, IEC. The structural capacity of the anchor in this type of soil governs the design, since rock is more resistant than sandy or clayey soils. Therefore, the design procedure starts with the structural capacity, followed by the geotechnical capacity.

2.1.1. Pile Anchors (PA)

Wind and ocean current turbines installed on fixed, or on floating platforms usually have pile foundations [3]. Compared to other types of foundations, such as DWAs, pile foundations have an important axial and lateral load capacity. However, pile foundations are usually deployed in clay and sandy soils, due to the difficulty of drilling rock.

Preliminary Design

1. Initial parameters. The environmental and structural data for the design is collected, these are specific parameters from the deployment site and device such as: characteristics of the ocean current turbine, geotechnical parameters, and environmental loads, such as marine currents. The mechanical properties of the steel are also needed; for example, yield stress, \( F_y \), ultimate stress, \( F_u \), Young’s modulus, \( E_s \). The latter can be obtained from the design codes and technical documents of steel structures API RP 2A-LRFD [16].

2. Structural analysis. Based on the loads obtained and using the procedures recommended in the specialized literature [3,14], the mechanical elements are obtained, namely: axial force, shear force, overturning moment, displacements and rotations.

3. Pre-dimensioning of the diameter. A preliminary pile diameter is proposed, based on the designer’s experience or on practical recommendations.

4. Pile wall thickness. The thickness of the pile, \( t_P \), is determined using the following Equation API RP2A-WSD [16]:

\[
t_P = \left[ 6.35 + \frac{D_P}{100} \right]
\]

where \( D_P \) is the pile diameter in mm and \( t_P \) is the thickness of the pile wall in mm.

5. Embedded length. The embedded length, \( L_r \), is computed with the formula of Poulos and Davis [36]:

\[
L_r = 4 \left( \frac{E_s I_P}{n_h} \right)^{\frac{1}{2}}
\]

where \( L_r \) is the embedded length in m; \( E_s \) is Young’s modulus of the steel, \( I_P \), is the second moment of area (see Appendix A), and \( n_h \) is the horizontal coefficient of the subgrade reaction.

Structural Capacity

6. Axial capacity (structural). Anchors for floating structures are only loaded by tension stresses. The structural capacity to axial loads is determined, based on API RP 2A-LRFD [16]:

\[
Q_{RASP} = \phi t_y F_y A_p
\]

where \( F_y \) is the yield stress of the steel, \( A_p \), is the cross-sectional of the anchor, and \( \phi t_y \) is the resistance factor for axial tensile strength, which can be taken as 0.95 API RP 2A-LRFD [16].
7. Lateral capacity (structural). The shear capacity, $Q_{RLSP}$, of the pile can be determined, using the following expressions [37]:

$$Q_{RLSP} = \frac{\phi_{us} F_{cr} A_p}{2}$$  \hspace{1cm} (4)

with

$$F_{cr} = \max(F_{cr1}, F_{cr2})$$  \hspace{1cm} (5)

$$F_{cr1} = \frac{1.60 E_s}{(D_e/2)^{1/3} \sqrt{H_p}}$$  \hspace{1cm} (6)

$$F_{cr2} = 0.78 E_s (D_e/2)^{1/2}$$  \hspace{1cm} (7)

$F_{cr}$ is the maximum value between Equations (6) and (7) and must not exceed 0.6 $F_y$. The resistance factor for lateral strength, $\phi_{us}$ is 0.90 LRFD [37].

8. Revision of structural capacity. To satisfy the design requirements, the total force on the mooring system ($V_d$, $H_d$) must not exceed the axial ($Q_{RASP}$) and lateral capacities ($Q_{RLSP}$). It is worth mentioning that buckling does not occur in anchors for floating. However, it can be estimated by the Equations presented in DNV-OS-J101 [15]. If the above requirements are not fulfilled, the pile geometry must be modified, and the revision procedure restarted.

Geotechnical Capacity

In this section, the geotechnical capacity of piles on rock is determined.

9. Resistance of the pile shaft. The ultimate shaft resistance, $Q_f$, can be calculated using the following expression:

$$Q_f = \sum_{1}^{N} f A_f$$  \hspace{1cm} (8)

where $f$ is the unit outside shaft friction and $N$ is the number of layers.

(a) Cohesionless Soil Interface

For piles in cohesionless soils, $Q_f$ can be calculated with Equation (9) API RP2A-WSD [16]:

$$Q_f = K p_0' \tan(f) A_f < f_l A_f$$  \hspace{1cm} (9)

where $K$ is the coefficient of lateral earth pressure, $p_0'$ is the effective overburden pressure, $f$ is the friction angle between the soil and the pile wall, $A_f$ is the surface area of the pile shaft (see Appendix A), and $f_l$ is the limiting skin friction. Further information regarding the definition of $K$, and $p_0'$ can be found in [38].

(b) Cohesive Soil Interface

For piles in cohesive soils, $Q_f$, is calculated using the following expression [16]:

$$Q_f = \alpha s_u A_f$$  \hspace{1cm} (10)

where $s_u$ is the undrained shear strength, and $\alpha$ is the adhesion factor. This factor is related to undrained shear strength, effective overburden pressure, and interface roughness [31].

(c) Chalk

The widely accepted guidance on pile design in chalk provided by CIRIA [29] suggests that the limiting skin friction varies between 30 kPa and 50 kPa for large-displacement,
pre-formed piles in low to medium density chalk [39] defined the peak post-cyclic shear strength, \( f_{s,\text{limit}} \), as:

\[
f_{s,\text{limit}} = \left[ \frac{\tan(\delta)}{\tan(\phi')} \right] s_u
\]

where \( \delta \) is the steel-chalk interface friction coefficient, \( \phi' \) is the post peak internal soil friction coefficient (from cyclic simple shear), and \( s_u \) is the undrained shear strength.

10. Geotechnical capacity. The axial geotechnical capacity, \( Q_{\text{RAGP}} \), of piles driven into rock depends on the resistance of the pile shaft, \( Q_f \), defined in Equations (8)–(12), and the submerged weight of the pile, \( W_p \) (see Appendix A), given in Equation (A1). Figure 4 shows free-body diagram of the pile anchor.

\[
Q_{\text{RAGP}} = Q_f + W_p
\]  

![Figure 4](image-url)

**Figure 4.** Free-body diagram of a pile anchor.

The lateral geotechnical capacity, \( Q_{\text{RLGP}} \), is calculated using the so-called P–Y curves; for example, Carter et al. [28], Reese [25], Gabr et al. [40], and Zhang et al. [30] (see Appendix B).

11. Geotechnical design revision. The acting design loads on the pile, \((V_d, H_d)\), must not exceed the geotechnical capacity, \((Q_{\text{RAGP}}, Q_{\text{RLGP}})\). Otherwise, the diameter must be reviewed and adjusted accordingly, followed by a re-evaluation of the anchor, starting the design procedure, from Step 3.

2.1.2. Dead Weight Anchors

A dead weight anchor (DWA) is made of concrete, without steel reinforcement, and must have the necessary weight to bear the environmental demands and the action of the turbine. The advantages of this type of anchoring are that they do not requiring drilling; in
comparison with embedded anchors such piles, they can be built from materials commonly used for onshore structures and can be easily recovered from the installation site.

The design procedure for DWAs considers the so-called shear keys, which increase the lateral anchor holding capacity by inducing failure in the soil and not at the anchor soil interface [14].

Using an iterative process, the precise size of the anchor and shear keys can be used to determine the weight needed to resist sliding, $Q_{AD}$, and the overturning moment, $M_d$ (see Figure 5).

![Free-body diagram of a deadweight anchor](image-url)

**Figure 5.** Free-body diagram of a deadweight anchor.

Preliminary Design

1. **Initial parameters.** The design criteria for DWAs are available in the literature as well as experimental design data [3,14,17]. In addition, the designer needs to gather the mechanical properties of the concrete, such as Young’s modulus, $E_c$, and compressive strength, $f_{c'}$, among others. These parameters can be obtained from the concrete structure design codes and technical documents, for example, ACI 318-08 [41].

2. **Structural analysis.** A structural analysis of the system is carried out, based on the loads obtained. From this analysis, the mechanical elements of the system are obtained: axial force, shear force, overturning moment, and displacements.

3. **Anchor weight.** A preliminary anchor design is made. At this point, the Equations proposed by van Zwieten et al. [14] can be used. To resist sliding on cohesionless soils the required submerged weight, $W_{DWA}$ is calculated using the following Equation:

$$ W_{DWA} = \frac{H_d}{\tan(\phi - 5^\circ)} + V_d $$

(14)

where $H_d = F_A \cos(\zeta)$ is the horizontal component of the anchor loading, $\phi$ is the angle of internal friction of the sediments, and $V_d = F_A \sin(\zeta)$ is the vertical component of the anchor loading.

$\zeta$ is the angle between the anchor loading and a horizontal line.

4. **Anchor diameter.**

**Anchor without shear keys**

The minimum anchor width/diameter ($B$) can be determined by:

$$ B = \left[ \frac{6W_{DWA}H_d}{\gamma (W_{DWA} - V_d)} \right]^{\frac{1}{3}} $$

(15)
where $\gamma_c$ is the submerged specific weight of the anchor material (concrete).

If shear keys in the DWA are required, see Appendix C. Maximum height: The maximum height, $H_m$, for the connection of the mooring line, above the base of the anchor, can be determined as suggested by Taylor [13] as:

$$H_m = \frac{B (W_{DWA} - V_d)}{6H_d}$$

(16)

The length of the cubical anchor needed to achieve the required weight can be determined by:

$$L_{DWA} = \frac{W_{DWA}}{\gamma_c H_m B}$$

(17)

The height of the cylindrical anchor needed to achieve the required weight can be determined by:

$$H_{DWA} = \frac{4W_{DWA}}{\gamma_c \pi B^2}$$

(18)

If the height $H_{DWA}$ is greater than $H_m$, $B$ should be modified; otherwise $H_{DWA}$ is considered the height of the structure.

5. Overturning moment. The overturning moment of the structure, $M_d$, is determined based on following Equation:

$$M_d = W_{DWA} \left( \frac{B}{2} \right) - V_d \left( \frac{B}{2} \right) - H_d H_{DWA}$$

(19)

The load eccentricity, $e$, is determined using:

$$e = \frac{M_d}{V_d}$$

(20)

Geotechnical Capacity

In this part of the procedure proposed, the geotechnical capacity of the structure is determined, using the formulation indicated in DNV-OS-J101 [15].

6. Effective dimensions. To determine the bearing capacity, an effective base area is needed (see Appendix C). Bearing capacity: In fully drained conditions, the general formula below can be applied for the bearing capacity of a foundation with a horizontal base, resting on the soil surface:

$$Q_{RAGD} = \frac{1}{2} \gamma_{soil} k_{eff} N_c s_\gamma i_\gamma + p_0' N_q s_i i_q + c_d N_c s_c i_c$$

(21)

where $N_c$, $N_q$, and $N_\gamma$ are the bearing capacity factors, $c_d$ design cohesion, $i_\gamma$, $i_q$, and $i_c$ are the inclination factors, $p_0'$ is the effective overburden pressure at the level of the foundation-soil interface, and $s_\gamma$, $s_i$, and $s_c$ are shape factors (see Appendix C).

For undrained conditions, which imply $\phi = 0$, the following formula for the bearing capacity applies:

$$Q_{RAGD} = s_{ud} N_c^0 s_i^0 i_c^0 + p_0^0$$

(22)

where $N_c^0$ is the bearing capacity factor, $i_c^0$ is the inclination factor, $p_0 = 0$ is the effective overburden pressure, $s_c^0$ is the shape factor, and $s_{ud}$ is the undrained strength of the design, assessed from the actual shear strength profile, load configuration, and estimated depth of the surface that potentially fails (see Appendix C).

7. Sliding capacity. The sliding capacity for drained conditions can be determined by the following Equation:

$$Q_{RLGD} = \left( c_d A_{eff} + V_d \tan(\phi) \right) \mu$$

(23)
In the case of undrained soils:

\[ Q_{RLGD} = \mu \left( s_{ud} A_{eff} \right) \]  

(24)

More detailed research on how to determine the sliding capacity for this type of structure can be found in [42,43].

8. Design revision. The acting forces \((V_d, H_d)\) must be less than the resistant axial and sliding forces \((Q_{RAGD}, Q_{RLGD})\). If this condition is not fulfilled, the preliminary design is modified, and the design process started again.

Structural Capacity

9. Axial capacity (structural). The tensile capacity, \(Q_{RASD}\), of the structure is determined using the following Equation ACI 318-08 [41]:

\[ Q_{RASD} = 0.8 \left( \bar{\vartheta} \sqrt{f'} \right) A_P \]  

(25)

where \(A_P\) is the cross-sectional area of the anchor (DWA) and \(\bar{\vartheta}\) is a parameter defined based on the type of concrete (ACI 318-08 [41]).

10. Lateral capacity (structural). The shear-resistant capacity, \(Q_{RLSD}\), of the anchor is determined using the following Equation [41]:

\[ Q_{RLSD} = \phi_{vc} \left[ 0.17 \left( 1 + \frac{V_d}{3.5 A_P} \right) \xi \sqrt{f'} B_d \right] \]  

(26)

with the characteristic length \((d = 0.8)\). Where \(\phi_{vc}\) is the resistance factor for shear design (typically 0.75), and \(\xi\) is a modification factor reflecting the reduced mechanical properties of concrete [41].

11. Sliding capacity. The sliding capacity can be determined by (see Figure 5):

\[ Q_{RSSD} = \mu N_f = \mu \left[ W_{DWA} - V_d \right] \]  

(27)

where \(N_f\) is the normal force, and \(\mu\) is the friction coefficient.

Revision of structural capacity. The resistant axial \((Q_{RASD})\) and lateral capacity \((Q_{RLSD})\) must not be exceeded by the design load, \((V_d, H_d)\). If these conditions are not fulfilled, the geometry of the structure must be changed, and the design process must be started again. In addition, the capacity should be checked for sliding \((Q_{RSSD} \geq H_d)\).

2.2. Service Limit State Design

The following is to determine whether the anchor satisfies the displacements and rotations for the service limit state (SLS) of the case of piles.

2.2.1. Pile Anchors

P–Y Curves

In order to satisfy the SLS conditions, the long-term behavior of the piles must be analyzed, based on DNV-OS-J101 [15]. The main parameters are the accumulated rotation, \(\zeta_{acc}\), and deflection, \(y_{acc}\), or, equivalently, the strain accumulation at the mudline level. To determine the deflections of the piles, p–y curves are generally used [44–46]; this methodology is based on the Winkler approach [47]. These curves are commonly defined for clayey and sandy soils, but in the case of weak rocks, such as weak mudstone, sandstone, and carbonate materials, different formulations for P–Y curves have been proposed (see Table 1). Appendix B shows the procedure to determine the P–Y curve for the case of weak rock.
Table 1. Reference for P–Y curve construction for various rocky soils.

| Type of Soil                     | Reference for P–Y Curves                          |
|----------------------------------|---------------------------------------------------|
| Calcareous                       | Wesselink et al. [48]                             |
|                                  | Williams et al. [49]                              |
|                                  | Dyson and Randolph [50]                           |
| Weak rock                        | Reese [25]                                        |
| Weak carbonate rock              | Abbs [27]                                         |
| Weak calcareous claystone        | Fragio et al. [51]                                |
| Strong rock                      | Turner [52]                                       |
| Massive rock                     | Liang et al. [53]                                 |

As the contribution of the shaft resistance and base resistance are not considered in this procedure [3,17], p–y curves do not apply for large-diameter piles. These curves predict the stiffness of the structure [45], which can have a direct impact on the calculated deformations, rotations and dynamic properties of the pile. For this reason, many researchers have been developing design procedures for large-diameter stock piles, for instance, Zdravković et al. [54] and Byrne et al. [55]. Procedures to calculate deflections and rotations on weak rock have been provided by various authors; for instance, Ashour et al. [56] and Carter and Kulhawy [28] studied the behavior of flexible and rigid shafts socketed into rock, subject to lateral loads and overturning moments. Zhang et al. [30] extended this approach to predict the nonlinear lateral load-displacement response of rock socketed shafts. The procedure considers subsurface profiles consisting of a soil layer overlying rock.

After calculating long-term deflections and rotations, these are compared to the maximum loads allowed in the design codes. The initial deflection, \( y_0 \), and accumulated deflection, \( y_{acc} \), must be less than 0.2 m; the initial rotation, \( \varsigma_0 \), must be less than 0.5°, and the accumulated rotation, \( \varsigma_{acc} \), must be less than 0.25° [3,17]. If these criteria are not met, the pile geometry must be modified, and the design process restarted.

2.2.2. Deadweight Anchors

One of the most significant parameters in determining whether the DWA satisfies the SLS is the formation of cracks. DNV-OS-C502 [35] states that design parameters do not cause permissible damage to the anchor due deflections and vibrations. This guarantees that the behavior allows the ocean current turbine to generate the amount of electrical energy it was designed to produce. To check that any cracks in the concrete structure are not significant, DNV-OS-C502 [35] proposes the following condition:

\[
\sigma_1 \leq \frac{f_{tn}}{k_1} \quad (28)
\]

where \( \sigma_1 \) is the principal tensile stress, \( f_{tn} \) is the normalized structural tensile strength of concrete, and \( k_1 \) is a constant value used in the calculations of crack (Table O3) DNV-OS-C502 [35].

In the event of the combined action of axial stress and bending moment, the following condition must be satisfied:

\[
(k_w \sigma_N + \sigma_M) < k_w \frac{f_{tn}}{k_1} \quad (29)
\]

where \( \sigma_N \) is the stress due to axial force (positive tension), \( \sigma_M \) is the edge stress due to bending alone (positive tension), and \( k_w \) is the coefficient dependent on cross-sectional height, defined by:

\[
k_w = 1.5 - \frac{h}{h_1} \geq 1.0 \quad (30)
\]

where \( h_1 = 1.0 \text{ m} \).
2.3. Fatigue Limit State Design

2.3.1. Fatigue Analyses

Fatigue analyses are used to estimate the number of loadings cycles the structure can withstand during its lifetime. This is usually carried out using either the accumulated damage model (i.e., Miner’s rule) or a detailed fracture mechanical study. In cases where the maximum stress amplitude does not exceed the fatigue limit amplitude for 10^7 cycles DNV-RP-C203 [34], or in early design stages, the accumulated damage model based on S–N curves might be sufficient.

In this study, the accumulated damage model is used, which is based on S–N curves. More detail can be found in DNV-OS-J101 [15]. As wave forces are the main cause of fatigue damage in offshore structures, other factors, such as periodic changes of wind and currents, are often neglected. In cases where the influence of other environmental factors should not be ignored, a more advanced fatigue model, based on simulations, might be the better choice, as per DNV-OS-J101 [15]. The accumulated damage model separates the sea states into different blocks of the same wave height and period, with their corresponding return period, \(T_R\). In industry, often only the wave height is used to define the block. In such cases, the corresponding wave period should be estimated accordingly. Miner’s rule gives the accumulated damage \(D_R\) in the reference period by:

\[
D_R = \sum_{i=1}^{I} \frac{n_i}{N_i}
\]  

(31)

where \(n_i\) denotes the number of load cycles in the \(i\)-th block, \(I\) is the total number of blocks.

The design life \(N_i\) can be understood as the number of load cycles which would damage the structure with the same loads as in the \(i\)-th block. The number of load cycles \((n_i)\) can be estimated by:

\[
n_i = T_R \frac{p_i}{T_i}
\]  

(32)

where \(p_i\) is the relative probability of the \(i\)-th block and \(T_i\) is the wave period corresponding to \(i\)-th block. In this study, the simplified block definition, based only on wave height, is used. Therefore, the corresponding wave period must be estimated. DNV-OS-J101 [15] gives a possible range of periods for a given significant wave height. To obtain conservative results, the lower boundary for the wave period is chosen:

\[
T_i = 11.1 \sqrt{\frac{H_i}{g}}
\]  

(33)

where \(H_i\) denotes the wave height of the \(i\)-th block and \(g\) the gravitational acceleration.

The period corresponding to the service life, \(T_L\), of the structure can then be obtained by:

\[
T_L = \frac{T_R}{D_R}
\]  

(34)

2.3.2. S–N Curves

Steel Anchors

Steel anchors generally consist of a steel tube with a pad-eye welded to the top end of the tube. The holes, joints, and especially the welds are prone to crack under load conditions and should be analyzed accordingly. S–N curves for welded joints are defined in DNV-RP-C203 [34] and DNV GL-ST-0126 [57]. The design life, \(N_i\) (predicted number of load cycles to failure), of the weld is defined by:

\[
\log N_i = \log \bar{a} - m \log \left( \Delta \sigma_{if} \left( \frac{t}{t_{ref}} \right)^k \right)
\]  

(35)
with the cyclic load range, $\Delta \sigma_{i,f}$, given by:

$$\Delta \sigma_{i,f} = \sigma_{i,\text{max}} - \sigma_{i,\text{min}}$$  \hspace{1cm} (36)

where $t$ is the material thickness, $t_{\text{ref}}$ the reference thickness, $\sigma_{i,\text{max}}$ and $\sigma_{i,\text{min}}$ the maximum and minimum stresses in the weld due to cyclic loads, respectively.

The factors $k$, $\log \sigma$, and $m$ depend on the number of load cycles and the type of welding and can be taken from Table 2-2 and Table 2-3 in DNV-RP-C203 [34] (page 14f). For welds parallel to the stress direction, S–N curves of C1 type should be used. For welds perpendicular to the stress direction and flush-ground welds, S–N curves of C1 type are also used, while curves D and E should be used for perpendicular welds not greater than 10% and 20% of the weld’s width, respectively.

Concrete Anchors

S–N curves for the design life of offshore concrete structures (reinforced and unreinforced) can be found in DNV-OS-C502 [35]. If made of concrete, DWAs are normally not reinforced, for which the corresponding S–N curves are defined as:

$$\log_{10} N_i = C_1 \frac{1 - \sigma_{i,\text{max}}}{f_{\text{rd}}} - \frac{1}{1 - \sigma_{i,\text{min}}}{f_{\text{rd}}}$$ \hspace{1cm} (37)

where $N_i$ is the predicted number of load cycles to failure, $f_{\text{rd}}$ is the compressive strength for the specific failure type, $\sigma_{i,\text{max}}$ the maximum compressive stress, and $\sigma_{i,\text{min}}$ the minimum compressive stress.

The minimum compressive stress should be considered as zero in case of tension (i.e., negative values). $C_1$ should be taken as 10 when $\sigma_{i,\text{max}}$ and $\sigma_{i,\text{min}}$ are both positive (i.e., compression-compression load case) and as 8 when $\sigma_{\text{min}}$ is negative (i.e., compression-tension load case). In case $\log N_i$ is greater than $\chi$, which is defined as:

$$\chi = \frac{C_1}{1 - \sigma_{i,\text{min}}}{f_{\text{rd}}} + 0.1C_1$$ \hspace{1cm} (38)

The design life can be extended to:

$$\log_{10} N_i' = C_2 \log_{10} N_i$$ \hspace{1cm} (39)

where $N_i'$ is the extended design life and the factor $C_2$ is given by:

$$C_2 = (1 + 0.5(\log_{10} N_i - \chi)) \geq 1.0$$ \hspace{1cm} (40)

3. Application of the Design Procedure

This section details the design procedure as applied to pile and DWA-type anchors for an ocean current turbine. The PA cross-section is circular, hollow, and made of steel, while the DWA is cylindrical in shape and made of concrete.

3.1. Case Study

In Mexico, the most suitable site for the deployment of ocean current turbines is off Cozumel Island, in the Caribbean Sea [9], as there is a persistent current from the NE. Data on the water depth in the shallow regions of the Cozumel Channel close to Cozumel Island can be found in [10,12]. In April 2019, a bathymetric survey was carried out as part of the CEMIE-O 1 research expedition around Cozumel Island. The multibeam echo sounder, Kongsberg ES 302, was used to gather the data presented in Figure 6. Across the transection, marked P1 on the mainland side and P2 on the island side, a bathymetric profile of the Cozumel Channel is given (see Figure 7). The raw data were interpolated using the function griddata and its cubic interpolation method, which is part of the scipy.
interpolate Python module [58]. In the first phase of the project, a small-scale turbine is to be installed in the shallower region, close to P2, as suggested in [10]. At those depths, the seabed is composed of a thin layer of fine sand with limestone below [12]. In the second phase of the project, a full-size turbine will be installed in the central part of the channel at water depths of about 400 m, for which a fixed structure is unfeasible, so a floating design is needed for the small-scale turbine as well; the marine current data were obtained from [10]. The turbine used in this example is an ocean current.

![Figure 6](image-url)

**Figure 6.** Bathymetric Survey carried out in the CEMIE-O 1 research expedition, April 2019. Background data from Natural Earth.

![Figure 7](image-url)

**Figure 7.** Bathymetric cross section of the Cozumel Channel. 5 times vertically enlarged.

The main difference between the ocean current turbine used in this example and other ocean current turbines is the persistence of the harnessed current. Generally, this type of turbine must adapt to a shift in the direction of the current every six or 12 h, depending on the location. The turbine discussed, off Cozumel, is not subject to significant shifts in the current. The turbine is installed on a platform of two floaters that provide the necessary buoyancy, and a strut connecting them with the turbine. The platform is moored to the seabed by cables and anchors, with the axis of the turbine aligned parallel to the flow. The turbine has three blades, and its rotor has a diameter of \(d_r = 1\) m; in addition, this device will be anchored at a water depth of \(h = 35\) m, 20 m above the seabed [10]. It is considered that the maximum deflection of the turbine from its equilibrium position (i.e., without environmental loads such as waves and currents) will not exceed 0.5 m in the vertical. The platform is attached to four mooring lines which, lower down, join into a single line which is attached to the anchor. In this way, the anchor only needs to resist the forces on the single line.

Table 2 summarizes the mechanical properties of the materials used, the soil found on the construction site, and the principal data of the turbine.
Table 2. Design parameters.

| Parameter                                      | Symbol | Value | Unit       |
|------------------------------------------------|--------|-------|------------|
| Structural Steel (A36)                         |        |       |            |
| Yield stress                                   | $F_y$  | 25    | MPa        |
| Tensile strength                               | $F_u$  | 400   | MPa        |
| Modulus of elasticity of steel                 | $E_s$  | 200   | GPa        |
| Specific weight                                | $\gamma_s$ | 7860 | kN m$^{-3}$|
| Structural concrete                            |        |       |            |
| Concrete Compressive Strength                  | $f'_{c}$ | 35   | MPa        |
| Specific weight                                | $\gamma_c$ | 23.6 | kN m$^{-3}$|
| Limestone                                      |        |       |            |
| Specific weight                                | $\gamma'_L$ | 26.5 | kN m$^{-3}$|
| Coefficient of subgrade reaction               | $n_h$  | 80    | MN m$^{-3}$|
| Sand                                           |        |       |            |
| Internal friction angle of the soil            | $\phi$ | 30    | degrees    |
| Specific weight                                | $\gamma'_s$ | 17   | kN m$^{-3}$|
| Effective overburden pressure                  | $p'_0$ | 10    | kN m$^{-2}$|
| Material factor                                | $\gamma_f$ | 1.2  | -          |
| Friction coefficient                           | $\mu$  | 0.5   | -          |
| Turbine                                        |        |       |            |
| Rotor diameter                                  | $d_r$  | 1     | m          |
| Drag coefficient                                | $C_D$  | 8/9   | -          |
| Length of mooring line                         | $l_M$  | 20    | m          |
| Allowed vertical deflection                    | $d_v$  | 0.5   | m          |
| Design depth of operation                      | $z$    | 10    | m          |
| Environment                                    |        |       |            |
| Design ocean current                            | $v_c$  | 2.5   | m s$^{-1}$ |
| Design tidal current                           | $v_t$  | 0     | m s$^{-1}$ |
| Design wave height                             | $DWH_d$ | 7    | m          |
| Design wave period                             | $T_d$  | 9     | s          |
| Design wind speed at 10 m height               | $v_{10m}$ | 15   | m s$^{-1}$ |
| Local water depth                              | $d$    | 30    | m          |
| Water density                                  | $\rho_w$ | 1025 | kg m$^{-3}$|

3.2. Design Considerations

The design requirements for anchors of ocean current turbines depend on many different factors. Their description and evaluation are beyond the scope of this work. Detailed information on design values or the required return period of the sources of environmental loads, and their respective load factors can be found in technical documents and design codes, such as IEC 62600-2 [59]. In the following section, the environmental loads are highly simplified, the purpose being solely to generate a representative load case and demonstrate the behavior of the design procedure presented. Eventually, these simplified approaches should be replaced by a more comprehensive mooring study for real problems, but they do give valuable insight into expected loads for the early stages of the design process. For those studies, mooring simulation tools can be used, such as ProteusDS [60]. Furthermore, the design is based only on a single load case, where the turbine is operating at its design depth and subject to the effects of the design current, design wave, and design wind. All load cases can be found in [59].

3.2.1. Environmental Interaction

Ocean current turbines and their anchors are subject to loads from a wide range of sources, such as hydrostatic pressure, currents, waves, tides, wind, ice, earthquakes, and collision with man-made objects. The considered construction site is not a seismic area, thus loads due to seismic actions can be ignored. Furthermore, this work only considers
environmental loads that are transmitted by the mooring line to the anchor; however, in some cases, direct interaction of the anchor with the water may be necessary to consider. For subsurface moorings, the environmental loads are caused primarily by the local velocity of the water. Hence, regarding environmental loads, this work is focused on water velocity and its main drivers: ocean currents, tidal currents, waves, and wind. Ocean currents are part of the oceanwide circulation systems and are caused by prevailing winds, and gradients in temperature and salinity. Tidal currents are a result of the periodic rise and fall of the water level and depend highly on the bathymetry and geometry of the ocean basin [61]. Based on IEC 62600-2 [59], the velocity at hub height can be understood as the vector sum of those four sources:

\[ \vec{v}_{\infty} = \vec{v}_c + \vec{v}_t + \vec{v}_w + \vec{v}_a \]  

(41)

where \( \vec{v}_{\infty} \) denotes the undisturbed velocity, \( \vec{v}_c \) is the velocity of the ocean current, \( \vec{v}_t \) is the tidal velocity, \( \vec{v}_w \) is the wave induced velocity, and \( \vec{v}_a \) is the wind induced velocity.

The velocity of the ocean current and the tidal velocity can be taken from technical documents or can result from extreme value analysis. Both the induced velocity from waves and from wind can be indirectly estimated via parameters of the design wave and design wind, as shown below. It should be mentioned that additional to those four drivers, the engineering design may need to evaluate other drivers or even non-linear velocity components as well.

3.2.2. Wave-Induced Velocity

Both Airy wave theory and the second-order wave theory give precise results for the requirements of most offshore engineering problems [17]. Waves induce velocities in horizontal (i.e., in direction of wave propagation) and vertical directions, both of which are periodic in nature. Applying the second-order wave theory, maximum and minimum horizontal velocities are defined as:

\[ v_w = \pm \zeta_a \omega \frac{\cosh(\kappa(d-z))}{\sinh(\kappa d)} + \frac{3}{4} \zeta_a^2 \kappa \omega \frac{\cosh(2\kappa(d-z))}{\sinh^4(\kappa d)} \]  

(42)

where \( \zeta_a \) denotes the wave amplitude, \( \omega \) is the angular frequency, \( \kappa \) the wave number, \( d \) the local water depth, and \( z \) the depth below the mean sea level (MSL).

The linear term in Equation (42) is taken as positive for the maximum velocity, and negative for the minimum velocity.

3.2.3. Wind-Induced Velocity

Based on IEC 62600-2 [59], wind effect is negligible in 20 m depth and below. The induced velocity is a result of wind shear [61] and presents a linear behavior in the upper 20 m of the water column [15]:

\[ v_a = 0.01 v_{a,ref} \left( 1 - \frac{z}{20} \right) \]  

(43)

where \( v_{a,ref} \) is the one hour mean wind speed 10 m above MSL and \( z \) the depth below MSL.

3.2.4. Horizontal Force on the Mooring System

To extract energy from the current flow, its momentum must be changed, requiring a drag force to be exerted on the flow. The related reactional force, exerted by the current on the rotor, has an axial and radial component with respect to the rotational axis of the rotor. The radial component creates the torque, which is used to generate electricity, while the axial component can be understood as rotor drag. Considering only the case where the turbine is in operational mode, the drag force can be conservatively estimated following Betz Law [62]. Assuming the rotor is operating at the maximum theoretical
power coefficient, \( C_P = 0.59 \), the rotor drag coefficient can be estimated as \( C_{\text{drag}} = \frac{8}{9} \) [63]. Thus, the drag force is:

\[
F_D = \frac{4}{9} \rho_w v_\infty^2 A_r
\]  

(44)

where \( A_r \) is the sweep area of the rotor, and \( \rho_w \) is the density of water.

For ocean current turbines, the rotor thrust accounts for roughly 81% of the total horizontal load of the system [14]. Hence, the thrust force, \( F_H \), can be estimated using the following Equation:

\[
F_H = \frac{F_D}{0.8} = \frac{5}{9} \rho_w v_\infty^2 A_r
\]  

(45)

3.2.5. Total Force on the Mooring System

The loads affecting the mooring system require detailed analysis and simulation, which is usually performed with specialized tools, such as ProteusDS software [60]. However, this study concentrates on the design of the anchor and the estimation of the loads on the anchor using simple physical relations and assuming a single mooring leg design. The total force on the mooring line is a function of the horizontal force estimated with Equation (45) and of the vertical load due to the buoyancy of the platform.

The mooring line is idealized as a straight line and ignoring the submerged weight of the mooring line and fluid loading for simplicity, the load on the mooring line is equal to the load on the anchor. In Figure 8a, the deflection of the system due to the horizontal load is depicted. Figure 8b shows the free body diagram of the turbine and the anchor point, assuming no elongation of the mooring line. Based on this and simple geometric relations, the required buoyancy force needed to limit the vertical deflection to \( d_v \) under the horizontal load, \( F_H \), can be calculated as:

\[
F_B = \frac{F_H (l_M - d_v)}{\sqrt{2l Md_v - d_v^2}}
\]  

(46)

The resulting force on the mooring line (\( F_M \)) can then be derived from:

\[
F_M = F_B \frac{l_M}{l_M - d_v}
\]  

(47)

Under the given assumptions, the total force, \( F_A \), must be equal to the force on the mooring line, \( F_M \):

\[
F_A = \gamma \frac{F_H l_M}{\sqrt{2l Md_v - d_v^2}}
\]  

(48)

where \( l_M \) denotes the length of the mooring i.e., the height above seabed in its initial position, \( d_v \) is the maximum vertical displacement allowed, towards the platform, \( \gamma \) is the resistance factor, and \( F_H \) is the horizontal force.

The effective angle of the mooring, \( \alpha_M \), from the vertical can be determined by:

\[
\alpha_M = \arccos \left[ 1 - \frac{d_v}{l_M} \right]
\]  

(49)
Figure 8. (a) Deflection of mooring system under horizontal load. (b) Free body diagram of turbine/mooring attachment point and anchor point.

3.3. Design Results and Discussion

A summary of the design results is presented in Table 3. To perform the calculations, SMath [64] was used.

As can be seen in the design procedure, it is the structural component which determines the design in pile, because the structural strength is higher compared to the geotechnical capacity. In the case of the DWA, the opposite behavior is presented; this is because this anchor is not directly in contact on limestone, but there is a sand layer. The Figure 9 shows the P–Y curve, where, for a horizontal load of 43 kN (load design), the maximum allowable deflection of 20 cm is satisfied.

Figure 10 gives a comparison between diameter and weight as a function of the strength of the structure. As can be seen, steel piles have greater strength than DWAs of the same diameter. However, the weight of the structure is significantly more, which can have a direct effect on the cost of construction and transportation.

| Table 3. Design results. |
|---------------------------|
| **Parameter** | **Symbol** | **Value** | **Unit** | **Equation** |
| **Load Design** | | | | |
| Final load of system | $F_A$ | 100.0 | kN | (48) |
| **Pile** | | | | |
| Diameter | $D_p$ | 10.0 | cm | - |
| Wall thickness | $t_p$ | 7.6 | mm | (1) |
| Area moment of inertia | $I_p$ | 243.0 | cm$^4$ | (A1) |
| Embedded length | $L_e$ | 144 | cm | (2) |
| Surface area of the pile shaft | $A_f$ | 4606 | cm$^2$ | |
| Weight | $W_p$ | 184.0 | kN | |
| Axial capacity (structural) | $Q_{RASP}$ | 428.1 | kN | (3) |
| Lateral capacity (structural) | $Q_{RLSP}$ | 128.4 | kN | (4) |
| Axial capacity (geotechnical) | $Q_{RAGP}$ | 4120.7 | kN | (13) |
| Lateral capacity (geotechnical) | $Q_{RLGP}$ | 454.6 | kN | |
Table 3. Cont.

| Parameter                      | Symbol  | Value  | Unit   | Equation |
|-------------------------------|---------|--------|--------|----------|
| Dead Weight Anchor            |         |        |        |          |
| Weight                        | $W_{\text{DWA}}$ | 188.2  | kN     | (14)     |
| Diameter                      | B       | 2.00   | m      | (15)     |
| Height of the mooring line    | $H_{\text{m}}$ | 1.92   | m      | (16)     |
| connection point              |         |        |        |          |
| Height of the cylindrical     | $H_{\text{DWA}}$ | 2.03   | m      | (18)     |
| anchor                       |         |        |        |          |
| Overturning moment            | $M_{\text{d}}$ | 58.9   | kN-m   | (19)     |
| Axial capacity (geotechnical) | $Q_{\text{LAGD}}$ | 231.2  | kN-m   | (21)     |
| Sliding capacity (geotechnical)| $Q_{\text{LGD}}$ | 26.5   | kN     | (23)     |
| Axial capacity (structural)   | $Q_{\text{RASD}}$ | 10190  | kN     | (25)     |
| Lateral capacity (structural) | $Q_{\text{RLSD}}$ | 2113.7 | kN     | (26)     |
| Sliding capacity (structural) | $Q_{\text{RSSD}}$ | 22.4   | kN     | (27)     |

Figure 9. P–Y curve of pile application example.

Figure 10. Comparative graph of the resistance and diameter of the anchors.
4. Conclusions and Recommendations

A new design procedure for ocean current turbines anchors on weak rock was proposed. It is easy to apply, since it uses Equations provided in technical documents and design codes that are normally used in industry; it is also possible to use tools common in the design of structures, such as spreadsheets. The iterative procedure consists of a preliminary anchor design that can be modified to satisfy the limit states considered. From the preliminary geometry, the structural capacity is determined, followed by the geotechnical capacity on rock to resist the loads corresponding to ULS. Finally, the anchor is revised against the requirements for the SLS and FLS.

The proposed procedure was applied for a turbine prototype to be deployed off Cozumel Island, Mexico. A floating turbine with a single mooring line was chosen for this project, as this enables the device to be passively oriented in the marine current. Great differences between the anchors were seen in terms of their weight and dimensions. The lack of slope at the installation site makes the use of a DWA feasible. The required weight for a DWA is significantly higher than that of the pile anchor, and this is directly proportional to the cost of the anchor, making this a relevant consideration. Although the cost of the material is an important parameter for the choice of anchor, there are other parameters to consider, such as availability of raw materials, manufacturing costs, transportation, installation, and decommissioning (if required). The following conclusions can be drawn:

(e) Both anchor designs showed satisfactory results based on the stipulations in the relevant design codes and technical documents. However, to guarantee this conclusion, it is necessary to carry out additional design tests.

(f) For relatively small projects, DWAs are the better option. In larger projects, piles are more suitable, especially if the dimensions of the DWA mean that expensive transportation and installation equipment are required.

(g) The deflection presented in the pile satisfies the maximum permissible value corresponding to the recommendations indicated in the technical documents for marine structures design.

Author Contributions: Conceptualization and methodology, F.B.-G. and M.R.; model and validation, F.B.-G. and M.R.; writing—original draft preparation, F.B.-G. and M.R.; writing—review, E.M. and R.S. All authors have read and agreed to the published version of the manuscript.

Funding: The authors are grateful to CONACYT-SENER-SUSTENTABILIDAD ENERGÉTICA project “Centro Mexicano de Innovación en Energía del Océano (CEMIE Océano)” : FSE-2014-06-249795 for their financial and technical support.

Conflicts of Interest: The authors declare no conflict of interest.

Nomenclature

| Symbol | Description                  |
|--------|------------------------------|
| $v_\infty$ | undisturbed current speed     |
| $v_t$   | wind induced velocity        |
| $v_c$   | current velocity             |
| $v_t$   | tidal velocity               |
| $v_w$   | wave induced velocity        |
| $h_1$   | parameter defined as 1.0 m   |
| $A_{eff}$ | effective area               |
| $A_f$   | surface area of the pile shaft |
| $A_p$   | cross-sectional area of anchor |
| $A_r$   | area of rotor sweep          |
| Symbol | Description |
|--------|-------------|
| $C_1$  | type of cyclic stress factor |
| $C_2$  | extended lifetime factor |
| $C_{drag}$ | drag coefficient of the rotor |
| $C_p$  | power coefficient |
| $D_p$  | pile diameter |
| $D_R$  | accumulated damage |
| $DWH_d$ | design wave height |
| $E_c$  | Young's modulus of concrete |
| $E_{ir}$ | Initial modulus of rock |
| $E_s$  | Young's modulus of steel |
| $F_A$  | final load on the anchor |
| $F_B$  | required buoyancy force |
| $F_D$  | maximum theoretical drag |
| $F_H$  | load produced by the turbine chain |
| $F_M$  | resulting force on the mooring line |
| $F_{cr}$ | critical strength |
| $F_u$  | ultimate stress |
| $F_y$  | yield stress |
| $H_{DWA}$ | height of the cylindrical anchor |
| $h$    | horizontal anchor loading |
| $H_i$  | wave height of the $i$-th block |
| $H_m$  | maximum height of the mooring line connection point above the base of the anchor |
| $I_p$  | area moment of inertia of the pile cross section |
| $K_p$  | passive earth pressure coefficient |
| $L_{DWA}$ | length of the cubical anchor |
| $L_r$  | embedment length |
| $M_d$  | overturning moment |
| $N_c$, $N_f$, $N_q$ | bearing resistance factor for drained conditions |
| $N_0^c$, $N_0$ | bearing resistance factor for undrained conditions |
| $F$    | normal force |
| $N_i$  | number of load cycles which would damage the structure under same load patterns as in the $i$-th block |
| $N_{i'}$ | number of load cycles which would damage the structure under same load patterns as in the $i$-th block, considering extended design life |
| $Q_{RAGD}$ | axial structural capacity of DWA |
| $Q_{RAGP}$ | axial geotechnical capacity of pile |
| $Q_{RASD}$ | axial structural capacity of DWA |
| $Q_{RASP}$ | Axial structural capacity of pile |
| $Q_{RLGD}$ | sliding geotechnical capacity of DWA |
| $Q_{RLGP}$ | lateral geotechnical capacity of pile |
| $Q_{RLSD}$ | lateral geotechnical capacity of DWA |
| $Q_{RLSP}$ | Lateral structural capacity of pile |
| $Q_{RSSD}$ | sliding structural capacity of DWA |
| $Q_f$  | ultimate shaft resistance |
| $T_L$  | period of service life |
| $T_R$  | reference period |
| $T_d$  | design wave period |
| $T_i$  | wave period corresponding to $i$-th block |
| $V_d$  | vertical anchor loading |
| $V_d'$ | lateral capacity of a deadweight anchor with full-base keying skirts in cohesionless soil |
| Symbol | Description |
|--------|-------------|
| \( W_{DWA} \) | submerged weight of dead weight anchor |
| \( W_k \) | submerged weight of each shear key |
| \( W_p \) | submerged weight of pile |
| \( Z_s \) | depth to the bottom of the skirts |
| \( \pi \) | Factor, directly related to intercept of log N-axis of S–N curve (log \( \pi \)) |
| \( b_{eff}, l_{eff} \) | effective dimensions |
| \( c_d \) | design cohesion |
| \( d_a \) | maximum permitted vertical deflection of the platform |
| \( f'_c \) | compressive strength of concrete |
| \( f_l \) | limiting skin friction |
| \( f_{rd} \) | compressive strength of concrete for specific failure type in fatigue analysis |
| \( f_{s,lim} \) | peak post-cyclic shear strength |
| \( f_{tn} \) | normalized structural tensile strength of concrete (Table C1) DNV-OS-C502 |
| \( i_0, i_q, i_c \) | inclination factors for undrained conditions |
| \( i_{t}, i_{q}, i_{c} \) | inclination factors for drained conditions |
| \( k_1 \) | constant, used in calculations of crack width (Table O3) DNV-OS-C502 |
| \( k_{dr} \) | dimensionless coefficient defined by Equation (A5) |
| \( k_{fm} \) | dimensionless coefficient in the range of 0.0005 and 0.00005 |
| \( k_w \) | coefficient dependent on cross-sectional height |
| \( l_M \) | length of the mooring |
| \( l_e, b_{eff} \) | dimensions defined in Appendix C |
| \( l_{eff}, b_{eff} \) | effective dimensions in Appendix C |
| \( n_h \) | horizontal coefficient of subgrade reaction of the soil |
| \( n_i \) | denotes the number of load cycles in the \( i \)-th block |
| \( n_{sk} \) | number of shear keys |
| \( p_0^{\prime} \) | effective overburden pressure at the level of the foundation-soil interface |
| \( p_a \) | elastic lateral pile resistance |
| \( p_l \) | relative probability of the \( i \)-block in fatigue analysis |
| \( p_u \) | ultimate pile resistance |
| \( q_e \) | embedment force of shear keys |
| \( q_u \) | soil’s uniaxial unconfined strength |
| \( s_{c}, s_{q}, s_{\gamma} \) | shape factors for drained conditions |
| \( s_0^{\prime} \) | shape factor for undrained conditions |
| \( s_u \) | undrained shear strength |
| \( s_{ud} \) | design undrained strength assessed based on the shear strength profile |
| \( t_p \) | pile wall thickness |
| \( t_k \) | shear key thickness |
| \( t_{ref} \) | reference thickness |
| \( v_{10m} \) | design wind speed at 10 m height |
| \( v_a \) | induced velocity |
| \( v_{1hr,ref} \) | one hour mean wind speed 10 m above MSL |
| \( v_c \) | design ocean current |
| \( v_l \) | design tidal current |
| \( v_{w} \) | maximum and minimum horizontal velocity |
| \( y_d \) | elastic limit pile deflection |
| \( y_{acc} \) | accumulated pile deflection |
| \( y_{0} \) | initial pile deflection |
| \( y_u \) | ultimate pile deflection |
| \( \alpha_M \) | effective angle of the mooring |
| \( \alpha_r \) | strength reduction factor |
| \( \gamma_c \) | submerged specific weight of the anchor material (concrete) |
| \( \gamma_k \) | specific weight of the anchor shear keys |
| \( \gamma_s \) | specific weight of structural steel |
| \( \gamma_{soil} \) | submerged unit weight of soil |
| Symbol | Description |
|--------|-------------|
| $\gamma_w$ | specific weight of water |
| $\zeta_a$ | wave amplitude |
| $\rho_w$ | water density |
| $\zeta_0$ | initial pile rotation |
| $\zeta_{acc}$ | pile's accumulated deflection rotation |
| $\sigma_1$ | principal tensile stress |
| $\sigma_M$ | edge stress due to bending alone (positive tension) |
| $\sigma_N$ | stress due to axial force (positive tension) |
| $\sigma_{i,\text{max}}$ | maximum stress due to cyclic load for $i$-th block in fatigue analysis |
| $\sigma_{i,\text{min}}$ | minimum stress due to cyclic load for $i$-th block in fatigue analysis |
| $\phi_d$ | parameter defined by Equation (A26) |
| $\phi_s$ | tangent of the friction angle at depth $Z_s$. |
| $\phi_{ax}$ | resistance factor for axial tensile strength (0.95) |
| $\phi_{sc}$ | resistance factor for shear strength for concrete anchor (0.75) |
| $\phi_{st}$ | resistance factor for shear strength for steel anchor (0.9) |
| $h$ | cross-section height |
| $w$ | friction angle between the soil and the pile wall |
| $B$ | minimum anchor width/diameter |
| $K$ | coefficient of lateral earth pressure |
| $N$ | number of layers |
| $RQD$ | rock quality designation |
| $c$ | cohesion of the soil |
| $d$ | local water depth |
| $e$ | eccentricity of the vertical force |
| $f$ | unit outside shaft friction |
| $g$ | gravitational acceleration |
| $i$ | total number of blocks |
| $k$ | exponent on thickness, for fatigue analysis |
| $m$ | negative inverse slope of S–N curve |
| $p$ | load per pile length unit |
| $t$ | material thickness |
| $y$ | pile deflection |
| $z$ | depth below MSL |
| $\Delta \sigma_{i,f}$ | cyclic load range for $i$-th block in fatigue analysis |
| $\alpha$ | adhesion factor |
| $\delta$ | steel-chalk interface friction coefficient |
| $\zeta$ | angle formed between the turbine chain and the anchor |
| $\kappa$ | wave number |
| $\mu$ | friction coefficient |
| $\xi$ | modification factor that reflecting the reduced mechanical properties of concrete, ACI 318-08 |
| $\zeta$ | pile rotation |
| $\chi$ | dimensionless parameter |
| $\omega$ | angular frequency |
| $\theta$ | parameter defined based on the type of concrete |
| $\phi$ | internal friction angle of the sediment |
| $\phi_f$ | post peak internal soil friction coefficient |

**Appendix A**

This Appendix shows some complementary Equations to calculate the geometrical and mechanical properties of piles anchors.

Second moment of area. The area moment of inertia of the pile cross section, $I_p$, can be determined using the following expression:

$$I_p = \frac{1}{8}(D_p - t_p)^3t_p$$  \hspace{1cm} (A1)
The surface area of the pile shaft, $A_f$, can be obtained using the following Equation:

$$A_f = \pi L_r D_p$$  \hspace{1cm} (A2)

Pile weight. The submerged weight of the pile ($W_p$) is determined by:

$$W_p = A_p L_r (\gamma_s - \gamma_w)$$  \hspace{1cm} (A3)

where $\gamma_s$ is the specific weight of steel and $\gamma_w$ is the specific weight of seawater.

**Appendix B**

In the case of weak rock, the shape of the $p$–$y$ curve is based on Reese [25] and is made up of three sections of behavior (see Figure A1). The first section corresponds to the elastic range, the second to non-linear behavior, and the third to when pile stiffness is lost. The input parameters that define the curve are given in Table A1.

**Table A1. Parameters that define $p$–$y$ curves on weak rock.**

| Parameter                          | Symbol | Unit   |
|-----------------------------------|--------|--------|
| Uniaxial unconfined strength      | $q_u$  | kPa    |
| Strength reduction factor         | $\alpha_r$ |        |
| Dimensionless coefficient in the range of $5 \times 10^{-4}$ and $5 \times 10^{-5}$ | $k_{rm}$ | - |
| Initial modulus of the intact rock | $E_{ir}$ | kPa   |

The elastic section of the curve, with deflections $y \leq y_a$ is given by:

$$p = k_{ir} E_{ir} y$$  \hspace{1cm} (A4)

where $k_{ir}$ is a dimensionless coefficient that accounts for changes, given by:

$$k_{ir} = \begin{cases} 
100 + 400 \frac{z}{D_p} & 0 \leq z \leq 3D_p \\
500 & z > 3D_p 
\end{cases}$$  \hspace{1cm} (A5)

The elastic limit displacement, $y_a$, is given by:

$$y_a = \left( \frac{p_u}{2k_{ir}(k_{rm}D_p)^{0.25}} \right)^{4/3}$$  \hspace{1cm} (A6)

where $p_u$ is the ultimate lateral resistance, taken as:

$$p_u = \begin{cases} 
\alpha_r q_u D_p \left( 1.0 + 1.4 \frac{z}{D_p} \right) & 0 \leq z \leq 3D_p \\
5.2 \alpha_r q_u D_p & z > 3D_p 
\end{cases}$$  \hspace{1cm} (A7)

The parameter $\alpha_r$ accounts for the fracturing of the rock. Reese [25] suggests taking $\alpha_r = \frac{1}{3}$ for rock quality designation RQD = 100, and decreasing linearly to RQD = 0. The $p$–$y$ curve for the plastic range with $y \geq y_a$ and $p \leq p_u$ is given by:

$$p = \frac{p_u}{2} \left( \frac{y}{k_{rm} b} \right)^{0.25}$$  \hspace{1cm} (A8)
where $k_{soil}$ is a dimensionless coefficient that accounts for changes, given by:

$$k_{soil} = \begin{cases} 100 + 400z & 3D < z < 5D \\ 500 & z \geq 5D \end{cases}$$ (A5)

The elastic limit displacement, $y_{el}$, is given by:

$$y_{el} = \frac{p_{soil}}{2k_{soil}k_{soil}B}$$ (A6)

where $p_{soil}$ is the ultimate lateral resistance, taken as:

$$p_{soil} = \alpha \cdot q_{soil}B \left[1.0 + 1.4z \frac{D}{D_{soil}} \right]$$ (A7)

The parameter $\alpha$ accounts for the fracturing of the rock. Reese [25] suggests taking $\alpha = 0.7$ for rock quality designation $RQD = 100$, and decreasing linearly to $RQD = 0$.

The $p$–$y$ curve for the plastic range with $y_{pl} < y_{el}$ and $p_{pl} > p_{soil}$ is given by:

$$p_{pl} = \frac{p_{soil}}{2} \cdot \frac{k_{soil}}{3B}$$ (A8)

Figure A1. P–Y curve for weak rock, after [65].

Appendix C

This Appendix shows complementary Equations for calculating the geometrical and mechanical properties of dead weight anchors.

Shear keys for DWA

The lateral capacity of this type of anchor with full-base keying skirts in cohesionless soil is computed as:

$$V_{ld}' = (W_{DWA} - V_d) \tan(\phi) + K_p \gamma_{soil} \frac{1}{2} Z^2 B$$ (A9)

The coefficient of lateral earth pressure, $K_p$, can be defined using the following Equation:

$$K_p = \tan^2 \left(45^\circ + \frac{\phi}{2}\right)$$ (A10)

Then, the minimum anchor width/diameter is:

$$B = \left(\frac{6 W_{DWA} H_d}{\gamma_{c}(W_{DWA} - V_d - 0.3H_d)}\right)^{\frac{1}{3}}$$ (A11)

The number of shear keys, $n_{sk}$, is calculated from:

$$n_{sk} = \left(\frac{200(W_{DWA} - V_d) \tan(\phi - 5^\circ)}{K_p \gamma_{soil} B^3}\right)^{\frac{1}{3}}$$ (A12)

The thickness of the shear keys, $t_k$, is obtained from:

$$t_k = 0.042 \left(\frac{\gamma_{soil} B^3}{V_d}\right)^{\frac{1}{2}}$$ (A13)

The submerged weight of each shear key, $W_k$, is estimated using the following Equation:

$$W_k = 0.05 \gamma_k B^2 t_k$$ (A14)

The embedment force of the shear keys, $q_e$, can be determined from:

$$q_e = \frac{\gamma_{soil} B^2}{400} \left[20t_k N_q + B \tan(\phi - 5^\circ)\right]$$ (A15)
where \( \gamma_k \) is the specific weight of the anchor shear keys, \( \gamma_{soil} \) is the submerged specific weight of the soil, \( N_q \) is the bearing resistance factor, \( \phi \) is the effective angle of internal friction, \( Z_s \) is the length of the skirts, and \( \phi_s \) is the friction angle of the soil at depth \( Z_s \).

Effective area of DWA

For a quadratic area, the effective area, \( A_{eff} \), can be defined as:

\[
A_{eff} = b_{eff} l_{eff}
\]

(A16)

where \( b_{eff} \), \( l_{eff} \) are the effective dimensions (see Figure A2).

![Figure A2. Quadratic footing with two approaches to make up the effective foundation area, based on DNV-OS-J101 [15]. (a) Load eccentricity with respect to only one symmetry axis; (b) Load eccentricity with respect to both symmetry axes.](image)

Eccentricity with respect to one of the two symmetry axes of the foundation.

In Scenario 1, load eccentricity is determined with respect to only one of the two symmetry axes of the foundation. The effective dimensions are calculated using the following expressions:

\[
b_{eff} = B - 2e
\]

(A17)

\[
l_{eff} = B
\]

(A18)

where \( e \) is the eccentricity of the vertical force

Eccentricity with respect to both symmetry axes of the foundation.

Scenario 2. Load eccentricity is determined with respect to both symmetry axes of the foundation. The effective dimensions are calculated using the following expression:

\[
b_{eff} = l_{eff} = B - \sqrt{2}e
\]

(A19)

Circular anchors.

In the case of structures with a circular base, the effective dimensions can be calculated by:

\[
b_{eff} = 2 \left[ \frac{1}{2} B - e \right]
\]

(A20)

with

\[
l_{eff} = B \sqrt{1 - \left[ 1 - \frac{b_{eff}}{B} \right]^2}
\]

(A21)

The effective area, \( A_{eff} \) (see Figure A3) can be calculated by:

\[
A_{eff} = 2 \left[ \frac{B^2}{4} \arccos \frac{2e}{B} - e \sqrt{\frac{B^2}{4} - e^2} \right]
\]

(A22)
Scenario 2. Load eccentricity is determined with respect to both symmetry axes of the foundation. The effective dimensions are calculated using the following expression:

$$b_{\text{eff}} = l_{\text{eff}} \sqrt{2} e$$  \hspace{1cm} (A19)

Circular anchors. In the case of structures with a circular base, the effective dimensions can be calculated by:

$$b_{\text{eff}} = 2 \left(1 - \frac{1}{b_{\text{eff}}} \right) b_{e}$$  \hspace{1cm} (A20)

with

$$l_{\text{eff}} = B - \left(1 - b_{\text{eff}} \right) B$$  \hspace{1cm} (A21)

The effective area, $A_{\text{eff}}$ (see Figure A3) can be calculated by:

$$A_{\text{eff}} = 2 \left( \frac{1}{2} \times \frac{1}{4} \arccos \left( \frac{2}{B} \right) \right)$$  \hspace{1cm} (A22)

Figure A3. Circular and octangular footings with the effective foundation area marked out, based on DNV-OS-J101 [15].

Based on the above, $A_{\text{eff}}$ can be represented by a rectangle with dimensions $b_{\text{eff}}$ and $l_{\text{eff}}$.

$$l_{\text{eff}} = \sqrt{A_{\text{eff}} \frac{l_{e}}{b_{e}}}$$  \hspace{1cm} (A23)

$$b_{\text{eff}} = \frac{l_{\text{eff}}}{l_{e}} b_{e}$$  \hspace{1cm} (A24)

Additional parameters to calculate bearing capacity in fully drained conditions

Design cohesion

$$c_{d} = \frac{c}{\gamma_{c}}$$  \hspace{1cm} (A25)

$$\phi_{d} = \tan \left( \frac{\tan(\phi)}{\gamma_{\phi}} \right)$$  \hspace{1cm} (A26)

Load factors

$$N_{q} = \exp(\pi \tan \phi_{d}) \left[ \frac{1 + \sin \phi_{d}}{1 - \sin \phi_{d}} \right]$$  \hspace{1cm} (A27)

$$N_{c} = \left( N_{q} - 1 \right) \cot \phi_{d}$$  \hspace{1cm} (A28)

$$N_{\gamma} = \frac{3}{2} \left( N_{q} - 1 \right) \tan \phi_{d}$$  \hspace{1cm} (A29)

when bearing capacity formulas are used to predict the reaction forces of the soil on the foundations, the calculation of $N_{\gamma}$ should be made using the following Equation:

$$N_{\gamma} = 2 \left( N_{q} - 1 \right) \tan \phi_{d}$$  \hspace{1cm} (A30)

Shape factors are defined by the following expressions:

$$s_{q} = s_{c} = 1 - 0.2 \frac{b_{\text{eff}}}{l_{\text{eff}}}$$  \hspace{1cm} (A31)

$$s_{\gamma} = 1 - 0.4 \frac{b_{\text{eff}}}{l_{\text{eff}}}$$  \hspace{1cm} (A32)
Inclination factors are defined by the following expressions:

\[ i_q = i_c = \left[ 1 - \frac{H_d}{V_d + A_{eff} c_d \cot \phi_d} \right]^2 \]  
(A33)

\[ i_\gamma = i_q^2 \]  
(A34)

These formulae apply to foundations which are not embedded. However, other formulae may be applied to embedded foundations, which will give conservative results. Alternatively, depth effects associated with embedded foundations can be calculated based on formulae given in DNV-OS-J101 [15].

Additional parameters to calculate bearing capacity for undrained conditions

In the case of undrained conditions, the bearing capacity factors are defined by:

\[ N_c^0 = \pi + 2 \]  
(A35)

\[ s_c^0 = s_c \]  
(A36)

\[ i_c^0 = 0.5 + 0.5 \sqrt{1 - \frac{H_d}{A_{eff} s_{ud}}} \]  
(A37)

References

1. Astariz, S.; Vazquez, A.; Iglesias, G. Evaluation and comparison of the levelized cost of tidal, wave, and offshore wind energy. J. Renew. Sustain. Energy 2015, 7, 053112. [CrossRef]

2. Vazquez, A.; Iglesias, G. Capital costs in tidal stream energy projects—A spatial approach. Energy 2016, 107, 215–226. [CrossRef]

3. Bhattacharya, S. Design of Foundations for Offshore Wind Turbines; John Wiley & Sons: Chichester, UK, 2019; Volume 1, ISBN 9788578110796.

4. Starling, M.; Scott, A.; Parkinson, R. Foundations and Moorings for Tidal Stream Systems; BMT Cordah Limited: London, UK, 2009.

5. Zhou, Z.; Benbouzid, M.; Charpentier, J.-F.; Scuiller, F.; Tang, T. Developments in large marine current turbine technologies—A review. Renew. Sustain. Energy Rev. 2017, 71, 852–858. [CrossRef]

6. Meggitt, D.J.; Jackson, E.; Machin, J.; Taylor, R. Marine micropile anchor systems for marine renewable energy applications. In Proceedings of the 2013 MTS/IEEE—San Diego an Ocean Current, San Diego, CA, USA, 23–27 September 2013.

7. Cresswell, N.; Hayman, J.; Kyte, A.; Hunt, A.; Jeffcoate, P. Anchor Installation for the Taut Moored Tidal Platform PLAT-O. In Proceedings of the 3rd Asian Wave Tidal Energy Conference, Singapore, Singapore, 24–28 October 2016; pp. 1–8.

8. Ma, K.-T.; Luo, Y.; Kwan, T.; Wu, Y.B.T. Mooring System Engineering for Offshore Structures, 1st ed.; Gulf Professional Publishing: Houston, TX, USA, 2019; ISBN 9781028185513.

9. Hernández-Fontes, J.V.; Felix, A.; Mendoza, E.; Cueto, Y.R.; Silva, R. On the Marine Energy Resources of Mexico. J. Mar. Sci. Eng. 2019, 7, 191. [CrossRef]

10. Alcérreca-Huerta, J.C.; Encarnacion, J.I.; Ordoñez-Sánchez, S.; Callejas-Jiménez, M.; Barroso, G.G.D.; Allmark, M.; Mariño-Tapia, I.; Casarin, R.S.; O’Doherty, T.; Johnstone, C.; et al. Energy Yield Assessment from Ocean Currents in the Insular Shelf of Cozumel Island. J. Mar. Sci. Eng. 2019, 7, 147. [CrossRef]

11. Graniel, J.B.; Fontes, J.; Garcia, H.; Silva, R. Assessing Hydrokinetic Energy in the Mexican Caribbean: A Case Study in the Cozumel Channel. Energies 2021, 14, 4411. [CrossRef]

12. Muckelbauer, G. The shelf of Cozumel, Mexico: Topography and organisms. Facies 1990, 23, 201–239. [CrossRef]

13. Taylor, R.J. Interaction of Anchors with Soil and Anchor Design; Naval Facilities Engineering Command: Port Hueneme, CA, USA, 1982; ISBN 9788578110796.

14. Vanzwieten, J.H.; Seibert, M.G.; Ellenrieder, K. Von Anchor selection study for ocean current turbines. J. Mar. Eng. Technol. 2014, 13, 59–73. [CrossRef]

15. Det Norske Veritas. DNV-OS-J101 Design of Offshore Wind Turbine Structures; Det Norske Veritas: Havvik, Norway, 2014.

16. American Petroleum Institute. API RP2A-WSD Recommended Practice for Planning, Designing and Constructing Fixed Offshore Platforms—Working Stress Design; American Petroleum Institute: Washington, DC, USA, 2002.

17. Arany, L.; Bhattacharya, S.; Macdonald, J.; Hogan, S. Design of monopiles for offshore wind turbines in 10 steps. Soil Dyn. Earthq. Eng. 2017, 92, 126–152. [CrossRef]

18. McCarron, W. Deepwater Foundations and Pipeline Geomechanics, 1st ed.; J. Ross Publishing: Fort Lauderdale, FL, USA, 2011; ISBN 9781604270099.

19. Bai, Y.; Jin, W.-L. Marine Structural Design, 2nd ed.; Elsevier: Oxford, UK, 2015; ISBN 9780080999975.

20. El-Reedy, M.A. Marine Structural Design Calculations, 1st ed.; Butterworth-Heinemann: Oxford, UK, 2015; ISBN 9780080999876.
21. Gaythwaite, J.W. Design of Marine Facilities, 3rd ed.; American Society of Civil Engineers: Reston, VA, USA, 2016; ISBN 9780784414309.
22. Fleming, K.; Weltman, A.; Randolph, M.; Elson, K. Piling Engineering; CRC Press: London, UK, 2008; ISBN 9780415266468.
23. Ferrero, A.; Migliazza, M.; Roncella, R.; Tebay, G. Analysis of the failure mechanisms of a weak rock through photogrammetrical measurements by 2D and 3D visions. Eng. Fract. Mech. 2008, 75, 652–663. [CrossRef]
24. Dobereiner, L.; De Freitas, M.H. Geotechnical properties of weak sandstones. Geotechnique 1986, 36, 79–94. [CrossRef]
25. Reese, L.C. Analysis of Laterally Loaded Piles in Weak Rock. J. Geotechn. Geoenviron. Eng. 1997, 123, 1010–1017. [CrossRef]
26. Goodman, R.E. Introduction to Rock Mechanics, 2nd ed.; John Wiley & Sons: New York, NY, USA, 1980; ISBN 9780471812005.
27. Abbas, A.F. Lateral Pile Analysis in Weak Carbonate Rocks. In Proceedings of the Conference on Geotechnical Practice in Offshore Engineering, Austin, TX, USA, 27–29 April 1983; ASCE: Austin, TX, USA, 1983; pp. 546–556.
28. Carter, J.P.; Kulhawy, F.H. Closure to “Analysis of Laterally Loaded Shafts in Rock” by John P. Carter and Fred H. Kulhawy (June, 1992, Vol. 118, No. 6). J. Geotechnol. Eng. 1993, 119, 2018–2020. [CrossRef]
29. Gannon, J.A.; Masterton, G.T.; Wallace, W.A.; Wood, D.M. Piled Foundations in Weak Rock; Construction Industry Research and Information Association (CIRIA): London, UK, 1999.
30. Zhang, L.; Ernst, H.; Einstein, H.H. Nonlinear Analysis of Laterally Loaded Rock-Socketed Shafts. J. Geotechnol. Geoenviron. Eng. 2000, 126, 955–968. [CrossRef]
31. Irvine, J.; Terente, V.; Lee, L.; Comrie, R. Driven pile design in weak rock. In Frontiers in Offshore Geotechnics III—Proceedings of the 3rd International Symposium on Frontiers in Offshore Geotechnics, Oslo, Norway, 10–12 June 2015; CRC Press: London, UK, 2015; pp. 569–574. ISBN 9781138028500.
32. International Electrotechnical Commission. IEC 62600-10 Marine Energy—Wave, Tidal and Other Water (IEC, Current Converters—Part 10: Assessment of Mooring System for Marine Energy Converters); International Electrotechnical Commission: Geneva, Switzerland, 2015; ISBN 9782832243111.
33. Das, B.M.; Sivakugan, N. Principles of Foundation Engineering, 9th ed.; Cengage Learning, Inc.: Boston, MA, USA, 2019; ISBN 9781373700305.
34. Det Norske Veritas. DNV-RP-C203 Fatigue Design of Offshore Steel Structures; Det Norske Veritas: Høvik, Norway, 2014.
35. Det Norske Veritas. DNV-OS-C502 Offshore Concrete Structures; Det Norske Veritas: Høvik, Norway, 2010.
36. Poulos, H.G.; Davis, E.H. Pie Foundation Analysis and Design; John Wiley and Sons: New York, NY, USA, 1990; ISBN 0894644491.
37. American Institute of Steel Construction (AISC). LRFD Load and Resistance Factor. Design Specification for Structural Steel Buildings; American Institute of Steel Construction: Chicago, IL, USA, 1999.
38. Tomlinson, M.; Woodward, J. Pile Design and Construction Practice, 6th ed.; CRC Press: Boca Raton, FL, USA, 2015; ISBN 9781466592643.
39. Carrington, T.; Li, G.; Rattley, M. A new assessment of ultimate unit friction for driven piles. In Proceedings of the 15th European Conference on Soil Mechanics and Geotechnical Engineering, Athens, Greece, 12–15 September 2011; Anagnostopoulos, A., Pachakis, M., Tsatsanifos, C., Eds.; IOS Press: Amsterdam, NY, USA, 2011; pp. 825–830.
40. Gabr, M.A.; Borden, R.H.; Cho, K.H.; Clark, S.C.; Nixon, J.B. P-y Curves for Laterally Loaded Drilled Shafts Embedded in Weathered Rock; Department of Civil Engineering North Carolina State University: Raleigh, NC, USA, 2002.
41. American Concrete Institute. ACI 318-08 Building Code Requirements for Structural Concrete and Commentary; American Concrete Institute: Farmington Hills, MI, USA, 1999; Volume 1, ISBN 9781424272718.
42. Butterfield, R.; Gottardi, G. A complete three-dimensional failure envelope for shallow footings on sand. Geotechnique 1994, 44, 181–184. [CrossRef]
43. Tistel, J.; Eiksund, G.R.; Hermstad, J.; Bye, A.; Atlanshi, C. Gravity Based Structure foundation design and optimization opportunities. In Proceedings of the Twenty-fifth International Ocean and Polar Engineering Conference, Kailua-Kona, HI, USA, 21–26 June 2015; pp. 772–779.
44. Matlock, H. Correlations for design of laterally loaded piles in soft clay. In Proceedings of the Second Annual Offshore Technology Conference, Houston, TX, USA, 22–24 April 1970.
45. Kallehave, D.; Thilsted, C.L.; Liinaegård, M.A. Modification of the API p-y Formulation of Initial Stiffness of Sand. In Proceedings of the Offshore Site Investigation and Geotechnics: Integrated Technologies—Present and Future; Society of Underwater Technology: London, UK, 2012; p. 8.
46. O’Neill, M.W.; Murchison, J.M. An Evaluation of p-y Relationships in Sands; Report No. GT-DF02-83; Department of Civil Engineering, University of Houston: Houston, TX, USA, 1983.
47. Winkler, E. Die Lehre von der Elastizität und Festigkeit: Mit besonderer Rucksicht auf ihre Anwendung in der Technik; H. Dominicus: Prague, Czech Republic, 1868.
48. Wesseling, B.D.; Murff, J.D.; Randolph, M.F.; Nunez, I.L.; Hyden, A.M. Analysis of Centrifuge Model Test Data from Laterally Loaded Piles in Calcareous Sand; CRC Press: London, UK, 1988; pp. 261–270. ISBN 9781003211433.
49. Williams, A.F.; Dunnavant, T.W.; Anderson, S.; Equid, D.W.; Hyden, A.M. The performance and analysis of lateral load tests on 356 mm dia piles in reconstituted calcareous sand. In Engineering for Calcareous Sediments; CRC Press: London, UK, 1988; pp. 271–280. ISBN 9781003211433.
50. Dyson, G.J.; Randolph, M.F. Monotonic lateral loading of piles in calcareous sand. J. Geotechnol. Geoenviron. Eng. 2001, 127, 346–352. [CrossRef]
51. Fragio, A.G.; Santiago, J.L.; Sutton, V.J.R. Load tests on grouted piles in rock. In Proceedings of the Annual Offshore Technology Conference, Houston, TX, USA, 6–9 May 1985; pp. 93–104.
52. Turner, J. *Rock-Socketed Shafts for Highway Structure Foundations*, 1st ed.; Transportation Research Board of the National Academies: Laramie, WY, USA, 2006.
53. Liang, R.; Yang, K.; Nusairat, J. p-y Criterion for Rock Mass. *J. Geotechnol. Geoenviron. Eng.* 2009, 135, 26–36. [CrossRef]
54. Zdravkovic, L.; Taborda, D.; Potts, D.; Jardine, R.; Sideri, M.; Schroeder, F.; Byrne, B.; McAdam, R.; Burd, H.; Houlsby, G.; et al. Numerical modelling of large diameter piles under lateral loading for offshore wind applications. In *Frontiers in Offshore Geotechnics III—Proceedings of the 3rd International Symposium on Frontiers in Offshore Geotechnics, Oslo, Norway, 10–12 June 2015*; CRC Press: London, UK, 2015; pp. 569–574. ISBN 9781138028500. [CrossRef]
55. Byrne, B.; McAdam, R.; Burd, H.; Houlsby, G.; Martin, C.; Gavin, K.; Doherty, P.; Igoe, D.; Taborda, D.M.G.; Potts, D.; et al. Field testing of large diameter piles under lateral loading for offshore wind applications. In Proceedings of the XVI ECSMGE Geotechnical Engineering for Infrastructure and Development, Edinburgh, UK, 13–17 September 2015; pp. 1255–1260.
56. Ashour, M.; Norris, G.; Pilling, P. Lateral Loading of a Pile in Layered Soil Using the Strain Wedge Model. *J. Geotechnol. Geoenviron. Eng.* 1998, 124, 303–315. [CrossRef]
57. Det Norske Veritas. *DNV GL-ST-0126 Support Structures for Wind Turbines*; Det Norske Veritas: Høvik, Norway, 2016.
58. Harris, C.R.; Millman, K.J.; van der Walt, S.J.; Gommers, R.; Virtanen, P.; Cournapeau, D.; Wieser, E.; Taylor, J.; Berg, S.; Smith, N.J.; et al. Array programming with NumPy. *Nature* 2020, 585, 357–362. [CrossRef] [PubMed]
59. International Electrotechnical Commission. *IEC 62600-2 Marine Energy—Wave, Tidal and Other Water Current Converters—Part 2: Design Requirements for Marine Energy Systems*; International Electrotechnical Commission: Geneva, Switzerland, 2016; ISBN 9782832235805.
60. Dynamic Systems Analysis ProteusDS Manual v2.29.2. 2016. Available online: https://dsaocean.com/downloads/documentation/ProteusDS2015Manual.pdf. (accessed on 1 July 2021).
61. Dhanak, M.R.; Xiros, N.I. *Springer Handbook of Ocean Engineering*; Springer: Trento, Italy, 2016; ISBN 9783319166490.
62. Betz, A. Das Maximum der Theoretisch Möglichen Ausnutzung des Windes durch Windmotoren. In *Zeitschrift für das Gesamte Turbinenwes*; R. Oldenbourg: Munich, Germany, 1920; p. 20.
63. Clarke, J.; Grant, A.; Connor, G.; Johnstone, C. Development and in-sea performance testing of a single point mooring supported contra-rotating tidal turbine. In Proceedings of the International Conference on Offshore Mechanics and Arctic Engineering—OMAE, Honolulu, Hawaii, USA, 31 May–5 June 2009; Volume 4, pp. 1065–1073.
64. Ivashov, A. SMath Studio (Versión 0.99.7030). Available online: https://en.smath.com/view/SMathStudio/summary (accessed on 1 July 2021).
65. Doherty, J.P. Lateral Analysis of Piles User Manual v 2.0. 2020. Available online: www.geocalcs.com/lap. (accessed on 1 July 2021).