Proceedings of the 2nd Marine Energy Technology Symposium
Global Marine Renewable Energy Conference
April 15-17, 2014, Seattle, WA

MOORING ANCHORS FOR MARINE RENEWABLE ENERGY FOUNDATIONS

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ABSTRACT

With the increasing use of offshore wind turbines, it has become necessary to explore deep-water sites for locating wind farms. Floating turbines are an ideal choice for these locations. Such turbines are anchored with mooring chains to the sea floor using suction anchors, driven piles or gravity foundations. This paper presents design methods for these types of foundations. Moored gravity foundations have been used for the much larger floating oil and gas installations. These concrete foundations resist the applied wind and wave loads through the dead weight of the concrete base combined with a short skirt along the periphery. A competent bearing stratum at seabed level is necessary to facilitate a gravity base.

INTRODUCTION

Houlahan et al. [1] discuss the options currently available to support offshore wind turbine generators, such as gravity bases, monopoles, jackets and, more recently, floating turbines. Although offshore fixed bottom foundations for oil industry can go up to 300m water depth, existing fixed foundations wind turbine technology had been limited to maximum water depths of 35 m. Worldwide deep-water wind resources are extremely abundant in subsea areas with depths up to 600 meters.

A floating wind turbine in deepwater is usually positioned on a submersible floating structure that is anchored by mooring lines to the seabed.

Such mooring systems are more practical where fixed foundations or jacket legs are not feasible. Furthermore, deep water locations are usually far from the shorelines thus reducing visual pollution as well as preventing interference with fishing and shipping lanes. Moreover, wind patterns are more favorable in offshore locations that are further away from land.

The mooring chains of a floating wind turbine can connected to either anchors embedded in the seafloor or through use of gravity base foundations. See Figure 1.

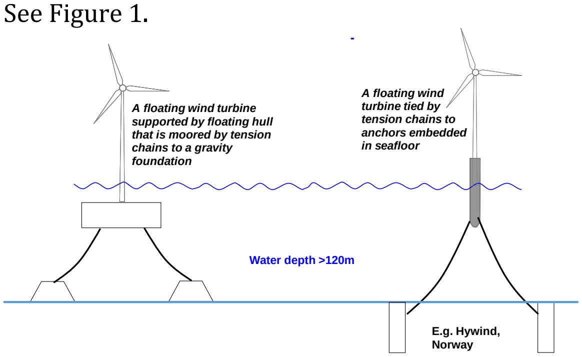

FIGURE 1 TYPES OF FLOATING ANCHOR FOUNDATIONS

At the time of writing this paper, floating wind turbines are not commercialized yet. However, test turbines have been evaluated by Norway's Statoil at Hywind, US-based Principal Power Inc, and the Netherland's Blue H Technologies, among others.

REVIEW OF CURRENT DEEP WATER WINDTURBINES

There are several deepwater floating turbines being tested with prototypes. These include:

- Hywind is the first full-scale floating wind turbine. Developed by Statoil, this turbine consists of a spar buoy anchored by three catenary cables installed in a water depth of 200 m in 2009, which is based on floating concrete constructions similar to those used in the oil industry. This turbine has a capacity of 6 MW. It was tested at SINTEF's wave tank in Trondheim and with a full prototype offshore Norway in 200m water depth. More tests are being carried out offshore Scotland in 100-m deep waters. See Figure 2.

FIGURE 2 HYWIND DEEPWATER WIND TURBINE. FROM REF [2]

- WindFLoat was developed by US based Principal Power Inc and tested offshore Portugal. The floating hull, tower and turbine are assembled onshore and the structure is then towed to place and tied to a moored anchor system. It relies on 4 mooring lines, with two attached to the column that support the turbine tower. The test WindFloat is rated at 2MW capacity. But 5 WindFloats with 6 MW Siemens turbines are currently being manufactured to be commissioned by end of 2017 in Port of Coos Bay offshore Washington state. See Figure 3.

FIGURE 3 WINDFLOAT DEEPWATER TURBINES. FROM WWW.PRINCIPALPOWERINC.COM

- Blue H developed a submerged deepwater platform (SDP), which was tested offshore Italy in a 113m water depth. This floating platform is similar to the tension leg platforms (TLP) used in the oil industry. The turbine tower and gravity counterweight can be manufactured onshore and towed to the site. The floating hull is held in the water by chains connected to the gravity base on the seabed by tension legs. See Figure 4.

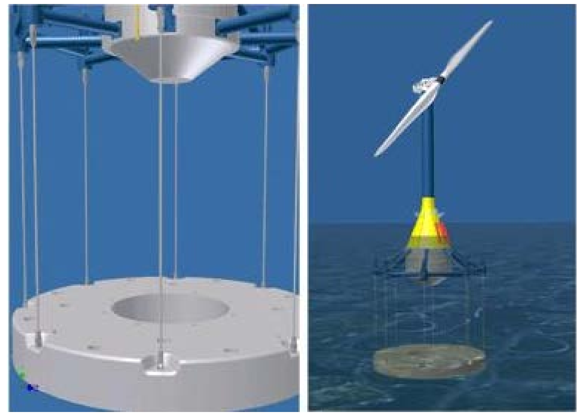

FIGURE 4 BLUE H SDP CONCEPT. FROM WWW.BLUEHGROUP.COM

More deepwater offshore designs are detailed in the European Wind Energy Report [2] issued in July 2013. The EWER report emphasizes that "more research must be done on mooring and anchoring systems. The industry could benefit from the experience of the oil and gas sector".

CHOICE OF FOUNDATION

The choice of foundation type depends on soil conditions, as well as the wind turbine size. If the soil is sandy, then a gravity based foundation is usually recommended. A gravity based structure (GBS) is built from steel reinforced concrete onshore, then towed and lowered in place. For larger GBS units, a buoyancy tank may be constructed that can keep the GBS afloat while in transport to the required location. An example of GBS foundation is that used for the Blue H turbine in Figure 4 above. Alternatively, conventional driven piles may be used as anchors in sandy soil conditions.

For clay soil stratigraphy, suction anchors or suction buckets can be used. These are quicker to install and easy to remove. The inverted bucket relies on negative pressure to help penetrate the anchor into the clay soil. Conventional driven piles can also be used in clay. However, due to noise pollution while driving, driven piles may not be desirable.

Another alternative for anchors in clay is the use of Suction Embedded Plate Anchor (SEPLA). A plate is embedded into the ground in a vertical direction by use of a conventional suction anchor as illustrated in Figure 5. The anchor is then removed from the ground (through use of over-pressure) and the plate is tensioned by pulling the pre-connected mooring line.

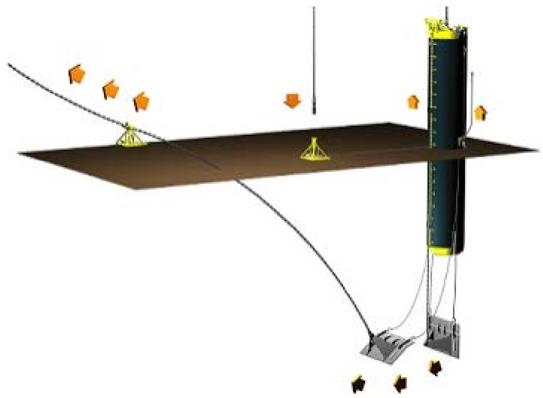

FIGURE 5 SEPLA ANCHOR. FROM [WWW.INTERMOOR.COM](http://www.intermoor.com)

GENERAL DESIGN CONSIDERATIONS

Important design aspects include the design soil parameters, determination of the failure mechanism and foundation capacity to resist applied environmental loads, evaluation of the foundation response to long term cyclic loading and foundation installation and removal aspects. Refer to Rahim & Stevens [3] for more details on the applicable industry standards.

SOIL DATA REQUIRED FOR DESIGN

The geotechnical investigation is generally performed by drilling a borehole to pre-selected depths and using downhole equipment which is lowered to the bottom of the borehole to perform sampling and in-situ testing. Cone penetration tests (CPTs) are often used in combination with drilling and sampling techniques. Typical borehole depths range from about 30 m to about 100 m. The soil investigation may consist of a single borehole with alternating in situ testing and sampling, separate boreholes for testing and sampling, or multiple boreholes with various combinations of testing and sampling.

The cyclic shear strength is load history dependent and is assessed on the basis of the shear strain and pore pressure contour diagrams in Figure 6 as part of the bearing capacity or horizontal sliding capacity checks.

CONFIGURATION OF EMBEDDED CHAIN

Since anchor piles are typically connected to mooring chains through the use of a padeye (see Figure 7), it is necessary to evaluate the chain load at the mudline and use chain analysis to calculate the load at the pile's padeye. Pile dimensions are determined using pile capacity analyses based on the padeye loads and the soil parameters.

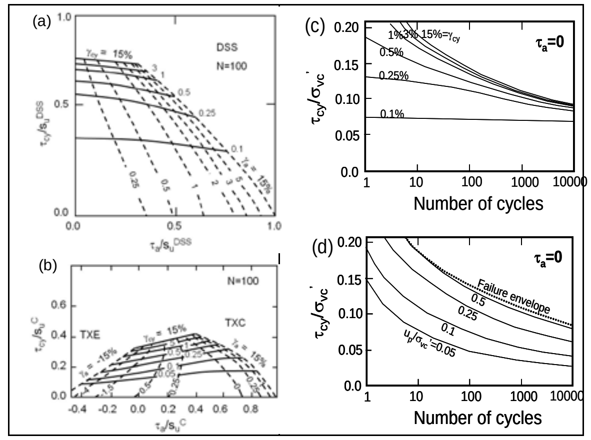

FIGURE 6 TYPICAL CONTOURS OF ACCUMULATED AVERAGE SHEAR STRAIN, CYCLIC SHEAR STRAIN AMPLITUDES AND PORE PRESSURE

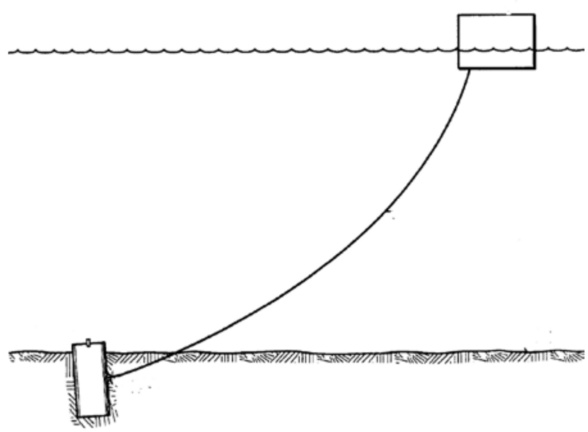

FIGURE 7 ILLUSTRATION OF MOORING CHAIN AND ANCHOR

The loads at the padeye of the anchor are different in both magnitude and angle from the loads of the corresponding mooring line at the mudline. The foundation load at the padeye becomes smaller than the corresponding line load at the mudline, and the loading angle at the padeye will be greater than the loading angle at the mudline. The change in shape and load is due to soil-chain friction acting tangentially to the chain and bearing resistance acting normally to the chain. The soil resistance results in an inverse-catenary mooring line shape of the embedded chain. Figure 8 shows (a) how the line angle may vary below the mudline and (b) the load situation against one chain element.

The soil around the chain is assumed to fail similarly to a strip footing with a loading direction perpendicular to the chain (Degenkamp and Dutta, 1989). Gault and Cox (1974) observed that the soil resistance normal to the chain has little influence on the total load transferred to the padeye but has a greater influence on the direction of a line.

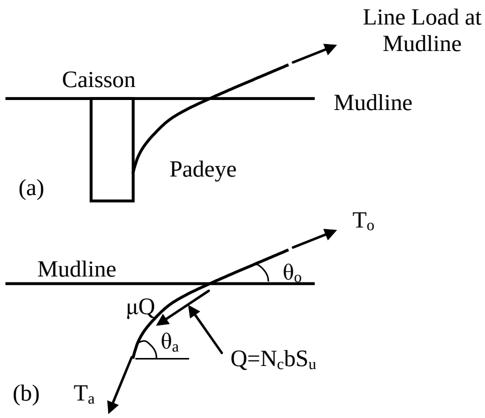

FIGURE 8 (A) INVERSE CATENARY MOORING LINE BELOW MUDLINE; (B) CHAIN-SOIL INTERACTION BELOW MUDLINE

The normal force per unit length, Q acting on the chain can be expressed in terms of undrained shear strength as

$Q = b \cdot N_C \cdot S_U$

where *b* is the effective width (*b*=2.5 x d_b) of bar diameter, d_b, of a chain, N_c is a bearing capacity factor, (usually equal to 9) and S_u is the local undrained shear strength of the soil.

PILE CAPACITY FOR DRIVEN PILES

For single piles, capacity may be evaluated by using any of the methods detailed in Section C.8.1 of API RP 2GEO. Some of these methods use the CPT resistance near the pile tip to find the tip resistance. Each of the CPT-based methods use a different set of formulae and make different assumptions in order to determine if an open-ended pile is plugged or coring. These methods are generally applicable for piles in clay soils, either in compression or in tension. If soil layers consist mostly of sand, it may be difficult to use driven piles as anchors, unless the sand is very dense. It may, therefore, be necessary to use an alternative foundation such as a gravity foundation.

The consideration of whether an open ended pile is plugged during driving is based on the general observation that open ended piles seldom plug when driving through cohesive soil profiles. Stevens [4] presents an equation to determine at what penetration a pile will plug that includes the inertial resistance of the soil plug.

Soil set-up effect may be evaluated based on published methods such as Stevens [5] and the NGI-05 method [6] for clay.

PILE CAPACITY FOR SUCTION PILES

The capacity of a suction pile may be checked by either limit equilibrium methods or the finite element method. Suction piles are usually wider than drive piles and may be assumed as ‘rigid’ in comparison to the surrounding soil.

Clays can generally be considered as undrained under the environmental loads, while sand often can be considered as undrained during one single cycle component of the environmental loads.

The effect of cyclic degradation before application of the peak cyclic loads may be found by calculating the equivalent number of cycles of the maximum cyclic load components that gives the same degradation as the actual loading history.

Pile tilt and miss orientation during installation should be accounted for during capacity calculations (see Figure 9). The padeye that connects the mooring line is usually placed about 2/3 of the way down the pile from the seabed. This ensures that the pile rotates anti-clockwise during extreme operation loads, thus no gap develops on the upper back side of the pile. An open gap may reduce the holding capacity and in extreme cases may undermine the tension suction resistance at the pile base. Industry standards such as DNV RP E303 may be used for the design and installation of suction piles.

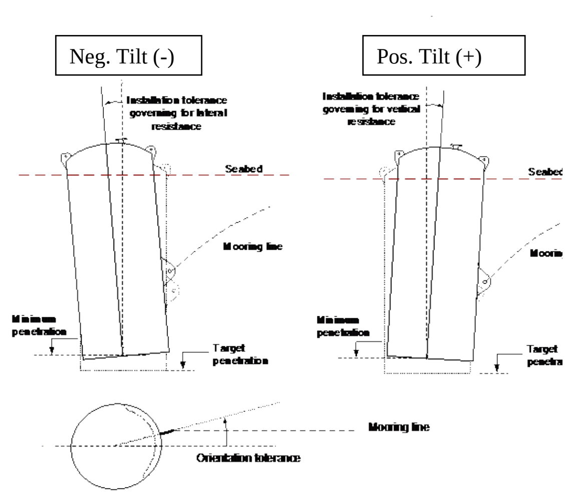

FIGURE 9 PILE TILT AND MIS-ORIENTATION DURING INSTALLATION

CAPACITY FOR SEPLA

A Suction Embedded Plate Anchor (SEPLA) is illustrated in Figure 5 above. The SEPLA holding capacity can be evaluated based on limit equilibrium methods. Soil resistance in the form of compression in front of the plate and tension (suction) behind the plate can be calculated simply as the plate area multiplied by both the soil strength and the bearing capacity factor, *Nc*. See Figure 10.

The SEPLA installation requires a suction pile follower that helps the anchor plate be embedded to target depth.

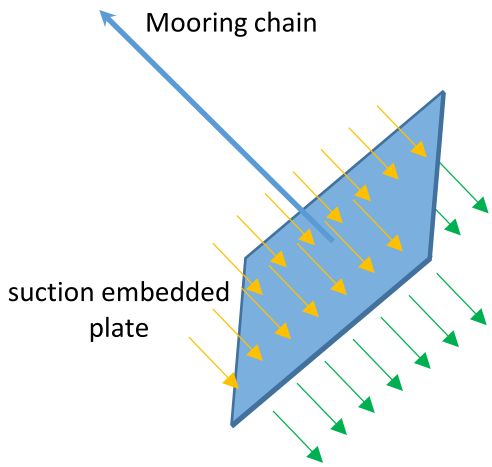

FIGURE 10 ILLUSTRATION OF SOIL REACTIONS ON A SEPLA ANCHOR

BEARING CAPACITY FOR GBS (ULS AND ALS)

Gravity-based structure (GBS) foundations for a fixed platform were used at Nysted, offshore Denmark, due to ice loading. Phase I was built in 2003. GBS for fixed platforms were also used at Thornton Bank offshore Belgium in an average water depth of 27 m.

GBS can also be used for floating wind turbines by providing sufficient counterweight on the seabed that would resist the tension in the mooring anchors. GBS foundations may be used when the soil is sandy since tension piles may not be suitable in difficult sand layers. The foundation may also have a short skirt around the periphery to resist sliding.

Checks of the foundation bearing capacity and sliding stability are done for the environmental loads in the ultimate limit state (ULS) and for the accidental limit state (ALS), if that load case governs. This provides a basis for the assessment of the required minimum submerged weight (on-bottom weight), W', of the structure, the need for ballast and skirts and, sometimes, the foundation area.

Loads to be considered are the permanent loads and the environmental or functional loads with components of average and cyclic.

For most GBS foundations the soil will be essentially undrained for at least one single cycle and the bearing capacity and sliding check will then be based on the undrained cyclic shear strength, $\tau_{f,cy}$, using a total stress approach. The strength definition and the laboratory data typically required for establishing the strength is given in Andersen [7]. The cyclic strength $\tau_{f,cy}$ is defined as $\tau_{f,cy} = (\tau_a + \tau_{cy})_f$ at N_{eq} number of cycles. The equivalent number of cycles N_{eq} accounts for both the degrading effect of increasing number of cycles of load and the strengthening effect of drainage through dissipation of pore pressure with time for partial drained loading. N_{eq} expresses the number of cycles of the peak load that would have the same degrading effect as the total number of cycles N of the design storm at different load levels.

The drained static and undrained cyclic bearing capacities may be evaluated using Brinch Hansen [8] and slip surface methods, such as Lauritzen and Schjetne [9].

Drained bearing capacity of GBS on sand

Brinch Hansen [8] provided the following comprehensive equation for the determination of ultimate bearing capacity, q_f, called the Generalized Bearing Capacity equation:

$$q_f = cN_c s_c d_c i_c g_c + qN_q s_q d_q i_q g_q + 0.5\gamma BN_\gamma s_\gamma d_\gamma i_\gamma g_\gamma$$

The bearing capacity factors are established based on the angle of internal friction, ϕ, as follows:

$$N_c = (N_q - 1)\cot\phi$$
$$N_q = (e^{\pi\tan\phi})\tan^2(45 + \frac{\phi}{2})$$
$$N_\gamma = 1.5(N_q - 1)\tan\phi$$

Equations are available for shape factors (s_c, s_q, s_γ), depth factors (d_c, d_q, d_γ), load inclination factors (i_c, i_q, i_γ) and ground inclination factors (g_c, g_q, g_γ). The effects of these factors are to modify the general bearing capacity due to various complications.

Cyclic bearing capacity of GBS on sand

For shallow foundations, bearing capacity can be checked analytically using Lauritzen and Schjetne [9], which is used to estimate the minimum bearing capacity safety factor of the gravity anchor. The methodology applied is a limiting equilibrium analysis related to the slip surface method. The method is based on the force equilibrium of the platform and the sliding body, as shown in Figure 11. The cross-section over the entire length L is constant, and the sliding body is cut off by vertical planes at both ends of the foundation. In order to compute the forces involved, the sliding body is divided into four sections: an active section, a flat section, an inclined section and a passive section, as indicated in Figure 11.

The resistance to sliding for all surfaces of the sliding body is evaluated and the resulting horizontal force in each section is calculated under force equilibrium conditions. The safety factor is found by the overall horizontal force equilibrium. The depth z to the lowest point on the slip surface is varied in steps and the critical depth giving the minimum safety factor is established.

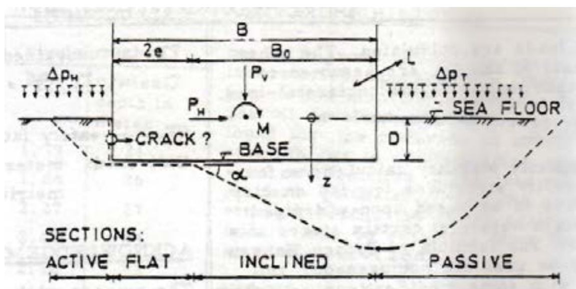

FIGURE 11 GEOMETRY OF FAILURE SURFACE.

Variable undrained shear strength with depth should be accounted for.

A foundation capacity envelope should be produced that shows the actual load in relation to the failure envelope of maximum horizontal and vertical loads as shown in Figure 12.

INSTALLATION OF DRIVEN PILES

Submersible hydraulic hammers can handle pile diameters up to 6 m using an anvil-connector sleeve. These hammers have rated energies of up to 2000 kJ.

Pile hammers can be chosen based on the pile size, penetration depth and type of soil. Methods of calculating soil resistance to driving include Toolan and Fox [10], Stevens et al. [11] and Alm and Hamre [12].

Pile self-weight penetration is estimated using the lower bound coring case for clay and the lower bound plugged case for sand computed using Stevens et al. [11].

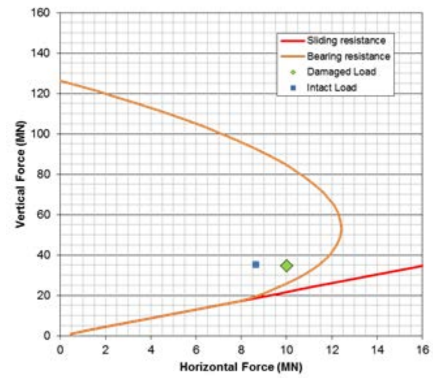

FIGURE 12 EXAMPLE OF FAILURE ENVELOPE FOR GRAVITY FOUNDATION

For clay layers skin friction will be taken equal to the measured remolded strength. In case reliable test results are not available, the α-value (where the α-value = τ_{skin} / su intact) to be used in clays will be calculated based on the SHANSEP model as detailed in API.

Often back-calibration of skin friction may be used of measured data are available from driving piles in similar soils.

More details on driven pile installation can be found in Rahim & Stevens [3].

INSTALLATION OF SUCTION PILES

Penetration calculations should be made in order to check the feasibility and the required suction to install the skirt to the required penetration depth. Standard codes such API RP 2SK can be used (section E3.2.1.1)

The penetration resistance is calculated as the sum of the side shear and the bearing capacity as given below:

$$Q_{tot} = Q_{side} + Q_{tip}$$

where:

Q_{tot}=Total penetration resistance
Q_{side} =Resistance along the skirt walls and possible stiffeners
Q_{tip} =Resistance at the skirt tip and possible stiffeners

The required underpressure to be applied within the skirt compartment is calculated using the expression:

$$u_{req}=(Q_{tot} - W') / A_{in}$$

where:

u_{req} =Required underpressure
W' =Submerged weight of anchor and installation equipment

A_{in} =Inside area where underpressure is applied

The allowable underpressure within the cylinder can be calculated by reversed bearing capacity considerations:

$$u_{max} = q_{tip} + Q_{inside} / A_{in}$$

where:

u_{max} =Maximum allowable underpressure.
q_{tip}=Reversed bearing capacity of soil plug at skirt tip level.
Q_{inside}=Resistance along the inside skirt wall and possible stiffeners.

The skirt tip reverse bearing is found as follows.

$$q_{tip} = N_c^{3D} \cdot s_{u,tip}^{av}$$

where:

N_c^{3D}=Bearing capacity factor for circular footings
$s_{u,tip}^{av}$=Average undrained shear strength at skirt tip

INSTALLATION OF GBS FOUNDATIONS

Gravity base foundations usually have sufficient weight to self-penetrate even if they have a short skirt. However, checks should be made to find out whether the foundation requires suction in order to penetrate the skirts. If so, then the same procedure for skirted suction anchors can be used to check the required suction.

Generally a check of the foundation bearing capacity and sliding stability is performed for loading in the installation phase, e.g. prior to assembly of tower and turbine and before the turbine is set into operation. It takes some time to get all the ballast in place and, for clay soils, it takes time to obtain the strengthening effect of full consolidation for the weight. Full strength may not be obtained; however, usually a less severe load case can be considered for this phase. The design may also include steps for the potential removal of the GBS.

SOIL REACTION STRESSES

Soil reactions against the structural members (e.g. base plate and skirts) should be evaluated. These reactions are used for structural design of the foundation and in general all design conditions and limit states should be considered. In practice, the governing design conditions are identified in collaboration with the structural designer.

The soil reaction's magnitude and distribution are estimated from conservative variations in strength, stiffness, seabed unevenness, and soil variability and by accounting for installation effects. Soil reactions may also be obtained from finite element analysis. Examples of such idealized stress distributions are shown in Figure 13. These idealized stress reactions are used for the installation and potential removal phases (effect of suction on buckling of the structure), as well as on the operation case where lateral stress from the padeye load is dominant.

High and low estimates of the different stress components must be given in order to account for uncertainties in the actual distribution.

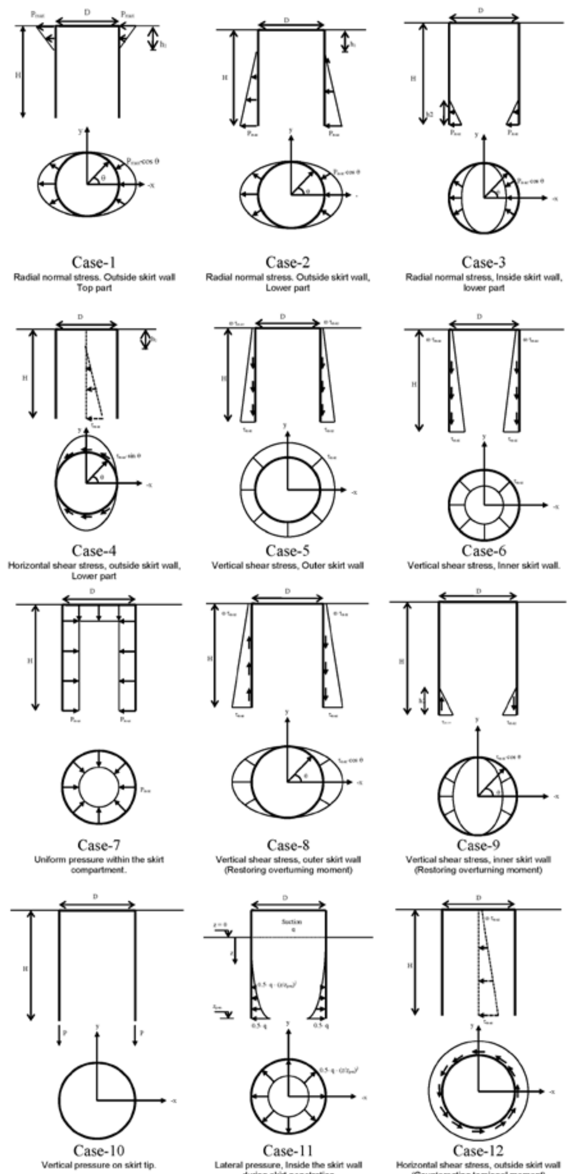

FIGURE 13 UNIT LOAD CASES FOR SOIL REACTIONS FOR THE STRUCTURAL DESIGN OF SUCTION ANCHOR

INSTRUMENTATION AND MONITORING

It is recommended that instrumentation and monitoring systems be installed to check the foundation performance for the installation and operational phase. This is especially useful with suction anchors. Such systems monitor the vertical and lateral displacement of the foundation, the tilt, and the pore pressures inside the suction pile foundation in addition to verifying that the valve system remains sealed in such foundations.

CONCLUSIONS

Floating turbines are an ideal choice for deepwater wind farm sites. Such turbines are anchored with mooring chains to the sea floor using suction anchors, embedded plates, driven piles or gravity foundations. This paper presents design methods for these types of foundations.

ACKNOWLEDGEMENTS

The writers wish to express sincere thanks to the ASCE COPRI Marine Renewable Energy Committee.

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