Floating structures that are moored impose a variety of load conditions on the anchor system. These loads range from vertical uplift loads for a TLP to almost horizontal loads on a catenary mooring line such as for a floating production system. Part of the load is sustained such as pretension in TLP tendons while environmental components tend to cycle with a range of periods. Wave loads might last for seconds, drift loads for hours, and loop current loads for days. These loading conditions provide a challenge to the geotechnical engineer in selecting and configuring the anchors.

To meet this challenge a number of anchor concepts are available including conventional driven pipe piles, the drag embedment anchor, the popular suction anchor, as well as hybrids such as the suction embedded plate anchor (SEPLA).

For permanent production systems the foundation design requires the full range of geotechnical design procedures including site investigation, soil characterization, foundation installation analysis, and foundation capacity assessment.
Contents

• Site investigation
• Soil characterization
• Anchor line mechanics
• Pile anchors
• Drag embedded anchors
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One of the first steps in a site investigation is a site survey employing geophysical methods such as reflection profiling and side scan sonar imaging. The survey provides several key pieces of information:

- It helps to identify potential geohazards such as slope instabilities, shallow gas, and hydrates.
- It provides a bathymetric picture of the seabed.
- It reveals the continuity of subsurface soil strata thereby reducing the number of soil borings that may be required.

These features are particularly important for catenary and taut mooring systems due to the number and lateral extent of the anchor sites.

Shown here are:

- A schematic of the reflection profiling process.
- A side scan sonar image showing a shallow ‘mudflow’.
- A seismic image showing a diapir and a potential shallow gas pocket.
The continental slope in the Gulf of Mexico has been greatly affected by salt migration. The bathymetry looks more like an artillery impact zone than the featureless area it was once thought to be. This rough topography with steep slopes, canyons, and furrows poses significant challenges to siting anchor locations not to mention locating flowlines and pipelines.
Soil Boring Operations

For permanent facilities it is necessary to obtain and test soil samples and/or carry out in situ tests at some or all of the proposed anchor locations. In shallow water a simple workboat arrangement may be adequate for drilling and sampling. In deeper water a purpose built rig capable of lifting large drilling loads is often used.
Conventional soil borings are conducted using uncased, open hole techniques. A drag bit is used to drill an oversized hole with the cuttings carried to the seafloor through the outer annulus. Sampling devices such as the push sampler shown are dropped through the drill string and latched in at the bottom. The sample is taken using the drill string as a reaction. The sampler is then retrieved with an overshot attached to a wireline. Different types of samplers are used depending on the type and condition of the soil.
Insitu testing for soil strength can be carried out in a borehole or using a seafloor test frame as shown above. Field vane devices test the soils undrained strength by measuring the torque required to rotate the embedded vane causing a cylindrical soil failure. The cone penetrometer is pushed into the soil and the resistance is measured at the cone tip and at a sleeve on the upper part of the tool body. Using correlations these measurements provide information on soil type as well as soil strength parameters.
Typical offshore soil deposits consist of soft normally consolidated clay with occasional sand layers. Over the past many thousands of years sea level has varied by some 300 feet or more. In near shore areas particularly near river mouths, deposition can be relatively rapid and the soil accumulates faster than the pore water can escape. This condition leads to very weak or so-called “underconsolidated” conditions. In deep water or away from sediment sources the accumulation rate can be extremely slow (millimeters per thousand years) such that the pore pressures remain at hydrostatic values during the deposition process. This gives rise to normally consolidated clays. The clay shear strength in these soils will increase more or less linearly with depth. Typical shear strength gradients range from 6-10 psf per foot of depth (1.0 to 1.5 KPa/m). If soil is eroded from an area subsequent to deposition it will be stronger than a normally consolidated soil, i.e. overconsolidated.
One of the key soil parameters in the design of a foundation or anchor is the undrained shear strength, termed $S_u$. The strength is termed undrained if the soil is sufficiently impermeable that the pore pressures cannot equalize in the time required for the shearing process. Offshore clays generally meet this condition for anchors loaded by environmental forces. There are a number of tests that can be used for determining the shear strength. For example, unconsolidated-undrained (UU) triaxial tests can be carried out on cylindrical soil samples or field vane tests can be carried out in situ.
Design Shear Strength Profile

This is a typical plot of shear strength measurements from unconfined compression test ($\sigma_3 = 0$) results vs. depth below mudline along with an interpreted shear strength profile. As suggested here, most things that happen to a sample after removal from the ground make it weaker. Unconfined compression tests are particularly subject to such sample disturbance effects. The interpreted strength profile compensates for disturbance effects to some degree.
A very important aspect of any mooring system is the interaction the mooring line has with the soil. As the line is tensioned up it cuts through the soil in a quasi-inverse catenary shape. The soil resists the line by bearing (normal load) and shear (tangential load). Invoking equilibrium on a differential element of the line provides the governing equations. These equations are nonlinear but can be solved by numerical methods fairly easily.
Simplified Anchor Line Analysis

Recipe for Catenary Anchor Line Analysis
(After Neubecker and Randolph, 1996)

\[ T_a = \frac{2D\bar{Q}}{\theta_a^2} \]
\[ T_o = T_a \exp(\mu \theta_o) \]

Where
- \( T_a \) = anchor holding capacity
- \( T_o \) = mudline line load
- \( Q \) = average bearing resistance of line per unit length = \( bN_c\bar{s}_{uavg} \)
- \( b \) = effective width of line
- \( N_c \) = bearing capacity factor
- \( s_u \) = undrained shear strength
- \( \mu \) = “friction” coefficient (approx 0.4)

Neubecker and Randolph (1996) made some simplifying assumptions and obtained the closed form solution to the anchor line equations shown here. They first neglected the effects of anchor line weight and they further assumed small anchor line angles. Even with these assumptions the solutions check closely with numerical solutions of the more rigorous equations over a very wide range of parameters. Errors are only significant in very soft soils (usually very near the mudline and for anchor line angles over 45 degrees.)
This plot shows an example of an anchor line calculation. The anchor here is at 15.25 m (50 ft.) depth; the line is 121 mm and horizontal at the mudline (catenary). The plot shows the line shape as a function of mudline load based on the Neubecker-Randolph solution. An important part of the solution is the line inclination at the anchor. The load inclination can have a significant effect on the anchor holding capacity.
Deepwater Mooring Systems

Load Conditions

• Vertical mooring (TLP)

• Catenary mooring (FPS&FPSO)

• Taut mooring (SPAR)

Loading conditions for anchor systems vary from almost pure horizontal loading (at the mudline) for catenary anchor lines to almost pure vertical loading such as for TLPs. However, even catenary lines can have a significant uplift component where the anchor line attaches to the anchor. Taut mooring systems significantly reduce the lateral extent of the mooring system over catenary systems but have a larger uplift component.
Piles can be driven 100 meters or more into the seabed where they can mobilize the high soil strengths at these depths and individually develop very large uplift resistances e.g. 50 MN or more. Thus, in principle, they are well suited for TLP support (direct uplift) compared to alternatives. They are less effective in resisting lateral loads however. A major drawback for using piles in deep water is the technically difficult and expensive installation process however significant advances are being made on that front.
The development of underwater pile driving hammers has made it possible to drive piles without a follower section to the surface. This has made driving piles in deep water a practical reality. There are a variety of hammers available. The hammers are hydraulic; the power pack may be coupled to the hammer or operated from the deck with an hydraulic umbilical. An electrical umbilical is required in any case. Some hammers are encased in tubular members and are basically extensions of the pile as shown on the left here. These are able to fit through pile guides on a platform. A different hammer configuration is shown on the right.
For most design applications the pile-soil system is represented as shown in the schematic. The pile is modeled as a linearly elastic beam column. The soil is represented as continuous, uncoupled, non-homogeneous, non-linear soil springs. These springs are idealized such that they resist the movement of the pile in the axial and lateral directions. The characteristics of these springs are based on empirical procedures that are calibrated with test data and on site specific soil properties. Once the full pile-soil model is developed it can be used for structural modeling in order to represent a realistic, compliant foundation as well as being used for the design of the pile itself. Pile designs are usually based on the most severely loaded pile.
The most critical element of pile design for uplift loading is the pile’s axial capacity. The procedure used for estimating axial pile capacity is shown qualitatively in the slide. Two cases can be considered. In one case (on the left) the soil plug is assumed to remain in place while the pile moves up in ‘cookie cutter’ style around the plug and through the soil. In this model, soil shear resistance develops on the outside (external skin friction) and inside (internal skin friction) as well as reverse bearing resistance on the pile annulus. For calculation purposes the unit internal and external skin friction values are taken to be equal. In the other case (on the right) the plug is assumed to move up with the pile such that a reverse bearing failure develops across the whole pile tip. In this case, which generally controls for long piles, only external skin friction is mobilized. The capacity of the pile is computed for both assumptions and the minimum value taken for design. In normally consolidated clays the uplift resistance of the tip is a small fraction of the external shaft resistance such that it is often ignored. Of course the detailed calculations depend on the soil type and design parameters.
This slide shows calculated pile capacity vs. pile length for a 60 inch diameter pipe pile in compression as well as uplift in a normally consolidated clay deposit with several significant sand layers. Note that in this plot the uplift tip resistance is neglected whereas, in compression, the tip resistance can contribute significantly to the overall capacity.
Pile-Soil Interaction Model for Lateral Loads

Piles can also be used as anchors to resist lateral loads or inclined loads although they are generally not as effective in this role.

One of the earliest attempts to model laterally loaded piles was to idealize the soil as a bed of linear springs. In this model there is no coupling of soil resistance from point to point along the pile, i.e. the soil resistance at any point on the pile is simply proportional to the displacement of that point. This is referred to as a Winkler foundation after its creator. Although this behavior is clearly oversimplified, the model does seem to capture the basic physics of the system, is surprisingly robust, and is widely used.

A number of investigators have carried out experiments to characterize the soil springs and correlate their behavior with measurable soil properties such as soil type, soil stress history and strength parameters. Recommendations for characterizing soil springs for clay and sand deposits are detailed in API Recommended Practice RP2A. This model is used to design the pile steel for both working stress design and LRFD.
Attaching the anchor line well below the mudline is much more effective than attaching it to the pile top. This slide shows the calculated moment diagrams for a 48 inch pile with a lateral load of 400 kips at the top and at 50 ft below the mudline in a normally consolidated soft clay. The moment for the top loaded case is over three times larger than the below mudline case. The moment generally decreases as the depth of the attachment point increases however the anchor line inclination angle increases such that the load becomes more vertical.
A good check on the laterally loaded pile design is to determine the reserve strength of the pile anchor by estimating the ultimate failure load of the pile. A closed form solution can be obtained for some simple cases as shown above. For a top loaded pile the failure mechanism involves the formation of a plastic hinge at some depth below the mudline. The solution for a uniform soil resistance is shown here on the left where \( R_o \) is the soil resistance per unit length and \( M_p \) is the plastic moment capacity of the pile. When the load is applied at significant depth the mechanism involves the formation of three plastic hinges. In this case the capacity is approximately three times that for the top loaded case. These results can differ significantly for different soil strength profiles.
Conventional steel pipe piles can be effectively employed as anchors for a variety of mooring systems. They are particularly effective for vertical loads as they can be driven to significant depths where soil strengths can be quite large. They can also be used for horizontal or inclined loads although they are generally not as effective in this role. The biggest obstacle for piles is the installation process which requires the use of an underwater hammer. In deep water this is a very sensitive operation that can be technically difficult and expensive, however improvements continue to be made in this area.
Drag embedment anchors have traditionally been used for temporary moorings of mobile offshore drilling units (MODUs) employing catenary mooring lines. While guidelines are available to estimate the ultimate holding capacity of anchors as a function of size and soil type there is significant uncertainty in this estimate. The installation process is basically a trial and error one which relies on experience and judgement. This ‘going in’ uncertainty has dissuaded some operators from using them for permanent facilities.

It is important to note, however, that drag anchors are generally load tested although not necessarily to design load conditions. Such testing, even at sub design loads can markedly increase reliability.

It has been recognized fairly recently that these anchors do have significant uplift capacity that has typically not been counted on. This has lead to the development of special purpose anchors specifically designed for uplift or vertical loading. These are termed vertically loaded anchors or VLA’s.
Among the simplest drag anchor capacity prediction methods are charts similar to this one which provide estimates of holding capacity vs anchor weight for a range of soil types. These charts are usually anchor specific. From a mechanics perspective the anchor weight itself plays only a minor role in capacity development. The more important factor is the fluke area which, of course, is correlated with anchor weight.
A number of models have been developed which attempt to invoke mechanics arguments in predicting the anchor loads and trajectories. These methods can be classified as semi-empirical since they rely on assumptions regarding the various components of soil resistance acting on the fluke and shank of the anchor. Once these assumptions are made for a given depth, the equilibrium equations are solved while invoking compatibility with the anchor line equations. The anchor is advanced and rotated a small increment and the process is repeated until the anchor reaches ultimate depth. These models provide estimates of the anchor depth, capacity, and required drag distance.
A recent study sponsored by API and the Deepstar JIP was carried out to test the consistency of such predictions. A set of hypothetical cases was developed with specific soils data and very simplified anchor geometries in an attempt to remove interpretation from the model description. Five volunteers from industry made estimates of trajectory, capacity, anchor orientation, etc. using their own models. Since these were hypothetical cases there was no known solution. The point of the exercise was to determine the variability among leading models with identical input data. As shown in this slide the variability in all predicted parameters was significant.

The uncertainty clearly increases if we include the interpretation required in estimating soil strength profiles from test data, reducing a complex 'real' anchor geometry to an idealized model, etc.

One conclusion from the study is that the models seem to provide reasonable predictions if they can be calibrated for specific sites, anchor geometries etc. but uncertainty is large for uncalibrated predictions.
The potential holding capacity of an anchor can be greatly enhanced if the anchor line pull is perpendicular to the fluke and located near the fluke centroid. In this mode a plate anchor can develop large uplift capacity. Several manufacturers have developed special anchors which exploit this behavior. These are termed vertically loaded anchors or VLAs.
In general the anchor is installed by drag embedment similar to a conventional drag anchor. Once installed to design depth the VLA is rotated using various mechanisms so that it is perpendicular (or nearly so) to the direction of loading.

This slide shows the installation scheme (left) and the removal scheme (right) for a Vryhof Stevmanta anchor. The capacity in the normal loading mode can be 2-3 time the installation pull-in load.
The anchor holding capacity depends on the inclination of the load (shear component) as well as the distance from the centroid (moment component). A series of finite element analyses was carried out to assess the interaction effects in a clay soil (undrained conditions). The results are summarized here in normalized form on the plot at the right. As shown the ultimate capacity of the plate is significantly diminished by load inclination and load offset from the centroid.
Plate anchors have the potential to develop large holding capacities, however, to date their primary application has been in their use for mobile drilling units. The uncertainty in the holding capacity of a plate installed at a known depth with known soil properties is commensurate with other anchor types. The major uncertainties are in the estimation of the trajectory, ultimate anchor depth, and final anchor orientation. These uncertainties have dissuaded some operators from using them for permanent deepwater moorings although some have been used with apparent success.
The anchor that has enjoyed the most widespread use for deepwater moorings is the so-called suction caisson. This anchor is a large diameter, thin-walled cylinder which is open at the bottom. Most suction caissons are steel however a few concrete ones have been employed. It is installed by a combination of dead weight penetration and pressure drawdown. This anchor has been used for direct uplift loading (e.g. Snorre TLP) but is most commonly used for catenary and taut mooring systems.
Initially the suction caisson is vented at the top to allow water to escape during dead weight penetration. The penetration is resisted by soil shear stresses acting on the walls of the caisson (external and internal) and by bearing resistance acting at the tips. Once the caisson comes to rest under its self weight the top is sealed and the pressure inside the caisson is slowly drawn down. Clay soils are relatively impermeable so the pressure gradient through the soil causes negligible flow. This pressure difference forces the caisson into the soil until its top is flush with the mudline. Careful monitoring is carried out during installation to insure that the soil plug does not fail and heave upward and that the pressure drawdown does not cause buckling of the caisson. Either occurrence would require removal of the caisson.
The prediction models for installation are relatively simple and borrow from conventional pile design. The soil shear resistance is estimated using the remolded shear strength of the soil and the tip resistance is estimated from conventional bearing capacity theory. At any depth the total soil resistance is estimated to determine the resistance that must be overcome by a combination of the caisson weight and pressure drawdown.

The biggest uncertainty in this prediction has been found to be in estimating the resistance of internal stiffeners. The behavior of ring stiffeners is particularly difficult. For example – does the soil flow around the stiffener or is a void created behind it?

The prediction model provides the installation team with expected performance including likely ranges of uncertainty. This coupled with careful installation monitoring has been very successful— with a few notable exceptions.
In addition to the installation prediction it is necessary to estimate the behavior of the caisson under load. In particular the foundation engineer needs to be able to estimate the caisson capacity for the geometry and soil profile being considered. One of the key issues is the depth of attachment of the anchor line which can have a substantial effect on caisson capacity.

An approximate solution to the lateral capacity of a caisson has been developed by Murff and Hamilton(1993) using plastic limit analysis. The mechanism used in this analysis is shown schematically here. It consists of a wedge forming in front of the caisson (and behind if suction on the windward side is assumed); a flow-around zone where the soil basically flows around the caisson in a horizontal plane; and a spherical tip failure mechanism. Several geometric parameters defining the mechanism are systematically varied to find the specific mechanism which gives the lowest capacity. This mechanism is then used to find a profile of soil resistance for some specific soil strength variations (uniform and linearly increasing with depth).
The lateral resistance profile is then defined as a bearing factor $N_p$ times the local undrained shear strength. As shown here, the bearing factor, $N_p$, is a minimum at the mudline and increases to a maximum value at a depth depending on the strength profile characteristics. Using the rules for constructing this profile simplifies the analysis further allowing us to calculate the lateral resistance accounting for the following conditions:

- Linearly increasing strength with finite mudline strength
- Soil-caisson interface conditions (degree of adhesion or roughness)
- Formation of a gap on the windward side or no gap
- Varying anchor line attachment depth

Predictions using this model are in close agreement with alternative analysis methods including extensive finite element studies as well as scale model tests in a geotechnical centrifuge.
Using the prescribed soil resistance profile coupled with the spherical failure surface at the caisson tip we search for the center of rotation of the caisson (failure mechanism) that gives the lowest capacity. This is done using the upper bound method of plasticity which employs virtual work calculations including soil resistance dissipation estimates.

Additional developments of this approach have recently been published by Aubeny, et. al (2003) which account for the inclination of the load, i.e. the effect of vertical load on the capacity.
The plot in this slide shows some typical results of an analysis giving capacity estimates where the lateral load is applied at different depths along the caisson. Note that the maximum capacity occurs where the load is applied well below the mudline. Further, the capacity is fairly sensitive to the exact load application point. This point will vary depending on the aspect ratio of the caisson, the soil strength profile, and other soil parameters.

Anchor line attachment at the optimum point results in a mechanism of pure translation; above this point results in 'forward' rotation with the top moving in the direction of the load; below this point results in backward rotation.

In addition, the lateral capacity is significantly affected by the soil adhesion at the caisson interface.
In situations which do not fall in the range of the 'conventional' assumptions more advanced analysis techniques may be required. Finite element modeling is a flexible analysis tool that allows one to include a wide range of parameter variations. Such studies do require three dimensional models hence they are time consuming, expensive, and technically difficult to achieve correct solutions.
Even for a catenary mooring system the anchor line at an attachment point below the mudline results in an inclined load. Load inclinations can, in turn, result in a reduced failure load. This slide shows a predicted interaction diagram showing the effect of vertical load on lateral bearing factors (and vice-versa). This effect has been observed in both analytical and experimental studies. Again the degree of this interaction depends on the geometry of the caisson, soil strength profile, and other parameters.
This slide shows suction caissons for the Njord Field FPS in the North Sea. They are 5m in diameter and 7-10m in length. Note the pad eye at the anchor line attachment point near the bottom of the caisson.
Sometimes the devil is in the details. This slide shows a suction caisson that has been extracted because it penetrated too easily. It is clear that the soil adhered well to the unprepared surfaces but did not adhere to the painted areas. Loss of adhesion can significantly reduce capacity particularly axial capacity.
Suction caissons are a highly effective anchor system. They can be placed at a desired anchor location with high precision, can be installed without the use of overly complicated mechanical systems, and their capacity can be predicted with reasonable accuracy. Experience with these anchors has been generally excellent.
Several innovative deepwater anchor systems have been developed in the past few years to compete with the mainstays. For example, the suction-embedded plate anchor (SEPLA) is shown here. This system combines some of the most favorable attributes of plate anchors and suction caissons. A reusable suction caisson is employed to place the plate anchor at an exact location. The anchor is then rotated in place such that the anchor line applies a normal load. One of the issues with this concept is the reliability of the post-installation alignment process.
Summary

• Several anchor concepts are available for mooring permanent deepwater facilities.
• To date piles tend to be favored for vertical moorings and suction caissons for catenary and taut moorings. Drag embedment anchors are commonly used for temporary moorings.
• The large number of variables involved make it difficult to develop blanket recommendations; designs require a case by case analysis.
Selected References

American Petroleum Institute (1995), Recommended practice for design and analysis of stationkeeping systems for floating structures, API-RP-2SK.


