ADDENDUM 1

Page 1, **1 Scope**, *replace the final bullet, and insert an additional bullet as follows*:

— design of pile foundations, and

— soil-structure interaction for risers, flowlines, and auxiliary subsea structures.

Page 1, **2 Normative References**, *replace the reference as follows:*

API RP 2A-WSD, 21st Edition, *Recommended Practice for Planning, Designing and Constructing Fixed Offshore Platforms — Working Stress Design*

Page 37, **Table 4**, **Key**, *replace*:

y_c equals $2.5 \times c \times D$;

with

y_c equals $2.5 \times \varepsilon c \times D$;

Page 38, **8.5.4 Lateral capacity for stiff clay**, *replace the last sentence:*

…the lateral resistance shall be reduced for cyclic design considerations.

with

…the lateral resistance shall be reduced for cyclic design considerations in accordance with acceptable best practice or available data.

Page 38, **8.5.5 Lateral soil resistance–Displacement (p-y) curves for stiff clay**, *replace the last sentence:*

…good judgment should reflect the rapid deterioration of load capacity at large deflections for stiff clays.

with

…good judgment should reflect the rapid deterioration of load capacity at large deflections for stiff clays in accordance with acceptable best practice or documented data.

Page 40, **8.5.7**, *last paragraph, replace*:

…the values of the rate of increase with depth of the initial modulus of subgrade reaction, k, given in Table 5 are recommended.

with

…the values of the initial modulus of subgrade reaction, k, given in Table 5 are recommended.

Page 40, **Table 5**, *title, replace*:

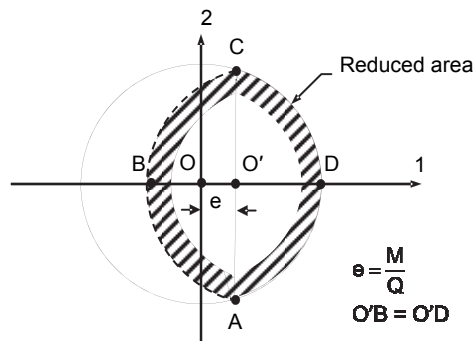**Table 5—Rate of increase with depth of initial modulus of subgrade reaction**

with

Table 5—Initial modulus of subgrade reaction values

Page 41, New **Section 9 Soil-structure interaction for risers, flowlines and auxiliary subsea structures**, *add new* **Section 9** after **Section 8.6.3**.

— Add the attached **Section 9**.

Page 50, **Figure A.2**, *replace with*:

Page 50, Equation (A.8), *at the end of the equation, insert*:

$$B' \leq L'$$

Page 50, *2nd paragraph following* Equation (A.8), *replace*

The centroid of the effective area is displaced a distance e_2 from the center of the base.

with

The centroid of the effective area is displaced a distance e from the center of the base.

Page 51, *replace* Equation (A.9) *with the following:*

$$\left.\begin{array}{l} A' = 2s = B'L' \\[2ex] L' = \left(2s\sqrt{\dfrac{R+e}{R-e}}\right)^{1/2} \\[3ex] B' = L'\sqrt{\dfrac{R-e}{R+e}} \end{array}\right\}$$

where

$$s = \frac{\pi R^2}{2} - \left[e\times\left(\sqrt{R^2 - e^2}\right) + R^2 \arcsin\left(\frac{e^2}{R}\right) \right]$$

Page 70, **Annex C, Pile Foundations**, *replace the annex title:*

    Pile Foundations.

with

    Pile foundation design commentary

Page 103, **Bibliography**, *add new listings at end of* **Bibliography**.

    — Add attached additional **Bibliography** pages.

An API-sponsored project [76] found, of the four group analysis methods examined in this study, the following methods to be the most appropriate for use in designing group pile foundations for the given loading conditions:

a) advanced methods, such as PILGP2R, for defining initial group stiffness (see Reference [76]);

b) the Focht-Koch (1973) method [77] as modified by Reese et al. (1984) [78] for defining group deflections and average maximum pile moments for design event loads. Deflections are probably underpredicted at loads giving deflections of 20 percent or more of the diameter of the individual piles in the group;

c) largest value obtained from the Focht-Koch and (b) methods for evaluating maximum pile load at a given group deflection.

9 Soil-structure interaction for risers, flowlines and auxiliary subsea structures

9.1 Site characterization

9.1.1 General considerations

Characterization of site conditions is necessary to develop a safe and economical layout and design of all facilities for a deepwater development including flowlines and risers. The site characterization required specifically for flowline design and riser touchdown point analyses should be included within the overall site characterization scope of work for the development. The scope of work may be optimized for a specific preliminary field layout and facilities design, but some final changes should be expected to accommodate final changes in field layout. The scope of work should anticipate the requirements for optimal design and code compliance.

9.1.2 Desktop assessment of site conditions

The initial step in site characterization is a desktop assessment of site conditions based on a review of available 2D and 3D exploration seismic data, regional and site specific geology and geotechnical data from engineering files, literature, or government files. The purpose of this review is to identify potential development constraints and to aid in planning and developing the scope of work for the subsequent shallow high resolution geophysical survey and geotechnical investigation.

9.1.3 Shallow high resolution geophysical survey

Upon completion of the desktop assessment, a high-quality, shallow high-resolution 2D geophysical survey should be performed over the entire areal extent of the development site prior to performing the geotechnical survey. The primary purpose of a shallow high resolution geophysical survey is to provide high resolution seismic data for a refinement of the shallow geology, geohazards, and seafloor features as expected from the desktop assessment, together with defining the uniformity and continuity of soil stratigraphy over the development site.

Shallow high resolution geophysical surveys in deepwater are typically performed using an autonomous underwater vehicle (AUV) equipped with the following:

a) multi-beam echo sounder for definition of water depth;

b) side scan sonar to define seafloor features;

c) sub-bottom profiler for definition of geologic structure and features.

Surveys can also be conducted using deep-towed equipment.

The scope of the geophysical survey field work typically also includes shallow soil sampling to help ground truth the side-scan sonar and sub-bottom profiler data with respect to soil type and stratigraphy at and near the seafloor and obtain soil data for preliminary design of flowlines and risers. The soil sampling equipment typically includes a triggered gravity drop core sampler with a core barrel length ranging from 3 to 6 m and a box core sampler with a typical maximum sampling depth of 0.5 m. Laboratory miniature vane and torvane plus possibly fall

cone tests are performed on the ends of gravity drop core liner segments to measure soil shear strength. In-situ laboratory miniature vane and hand operated miniature T-bar penetrometer tests are recommended to be performed on the soil sample inside the box core sampler, followed by the recovery and preservation of sub-samples from the box core sampler.

9.1.4   Geotechnical investigation

9.1.4.1   General

The purpose of the geotechnical investigation is to explore and define the soil conditions across the development site, including soil stratigraphy, soil type, and the pertinent soil properties for design of flowlines and risers. The results of the geophysical survey should be integrated with the results of the desktop assessment and any existing geotechnical data to guide the final location of flowlines and riser touchdown points and to develop a scope of work for the final geotechnical investigation.

9.1.4.2   Soil sampling and in-situ testing

The total scope of geotechnical work for a development should include some combination of the following soil sampling and in-situ testing techniques:

a)   drilled soil borings with downhole soil sampling and in-situ testing;

b)   continuous seabed cone penetration, T-bar or ball tests to penetrations as shallow as 3 m for flowlines and steel catenary risers and as deep as 40 m, for riser towers and top tension risers;

c)   large-diameter piston drop cores (core barrel length up to 20 to 30 m);

d)   large-diameter push samples (core barrel length up to 20 m);

e)   gravity drop cores and box cores.

9.1.4.3   Laboratory soil testing

Accurate determination of an appropriate undisturbed undrained shear strength profile is fundamental to the geotechnical design of flowlines and risers. The only laboratory strength tests that can be performed on near-seafloor, very soft clay soil samples are motorized laboratory miniature vane, hand-operated vane shear device such as the torvane, and fall cone. In addition, the remolded shear strength should also be measured using the miniature vane and fall cone to evaluate soil sensitivity.

In addition to the above described strength testing, total (bulk) density, Atterberg limits, and moisture content should also be performed. Optional tests may include grain size distribution, specific gravity, carbonate content, and pH. Depending on the intended application, thermal conductivity and electrical resistivity tests may also be performed to assess the insulating and corrosive properties of the soil.

Laboratory soil testing should be performed in accordance with recognized industry standards such as ASTM Book of Standards, Volume 04.08: Soil and Rock (1) or BS 1377:1990, 'British Standard Methods of Test for Soils for Civil Engineering Purposes'.

9.1.4.4   Interpretation of soil design parameters

The undisturbed shear strengths measured in the field and laboratory should be plotted together with the interpreted shear strength based on in-situ penetrometer tests to develop a design shear strength profile. A design remolded shear strength profile can be based on data from a cyclic penetrometer test carried out in situ or in box cores, an interpretation of actual remolded strength test results on recovered soil, or by dividing the undisturbed design shear strength by an interpreted average soil sensitivity considered appropriate for the site. Upper and lower bound design and remolded shear strength profiles may be developed for use in parametric studies. Interpretation of design shear strength should be performed by an experienced geotechnical engineer/consultant qualified to consider all the factors influencing the measurement of shear strength.

If information about the regain of strength of remolded soil due to reconsolidation or thixotropy is required, thixotropy tests can be performed on site-specific samples after various storing times. Thixotropy strength testing can be done using the miniature vane and fall cone.

A design submerged unit weight profile should be interpreted from a plot of measured unit soil weights versus penetration. If required, the total unit weight of the soil can be computed using the measured moisture content and specific gravity for the soil, and assuming the soil is 100 percent saturated.

The over-consolidation ratio (OCR) of the soil cannot typically be measured in near-seafloor sediments. It can be estimated through extrapolation from deeper depths or from the measured water content. An understanding of in-situ OCR and expected dilatant or contractant behavior of the soil when sheared may prove useful for the design of flowlines. An understanding of the remolded strength conditions of the soil near the mudline can also be useful for better understanding the soil response.

9.1.5 Integrated study

Finalization of the site characterization may require the integration of the geotechnical data, geological study and the shallow high resolution data, depending on the uniformity of geologic and soil conditions. Such an integrated study can develop maps showing the areal extent of different soil or geologic units and isopach maps showing the depth below the seafloor for different soil or seismic horizons and the thickness of different soil or geologic units. The results of the integrated study can be used to assess restraints imposed on flowline and riser design by seafloor features, geohazards, and soil conditions. This integrated study is an extension and update to the desktop assessment discussed in Clause 9.1.2.

9.2 Steel catenary risers

9.2.1 Introduction

The geotechnical properties of the seabed can influence the design conditions for steel catenary risers (SCRs) in two of the following aspects:

— an ultimate limit state associated with excessive bending and tensile stresses in the riser wall;

— a fatigue limit state associated with cumulative damage to the riser from motion-induced changes in bending stress in the region of the touchdown point.

In an SCR, the maximum curvature occurs within the suspended part of the catenary and the seabed stiffness has a negligible effect on the maximum curvature. Thus the seabed properties have essentially no influence on the maximum in-plane bending stresses within the riser. However, the seabed properties have a significant influence on the shear force in the riser, and hence changes in bending moment due to environmentally-induced motions of the riser. The properties thus affect fatigue calculations. In addition, the seabed properties will affect local out-of-plane curvature of the riser during extreme environmental events or large transverse or out-of-plane motions, particularly where the riser has become partially embedded within the touchdown zone. They may also affect transient bending moments induced during any position changes of the floating facility from which they are suspended.

9.2.2 Design for ultimate limit state

An ultimate limit state can arise under extreme environmental events that cause out-of-plane motion, particularly where the riser has become embedded, or lies within a trench, thus giving rise to high lateral soil resistance and locally high curvature of the riser.

Specialist geotechnical advice should be sought in order to quantify the lateral soil resistance, which will usually exceed normal frictional resistance for pipelines lying on the seabed surface (see Clause 9.5). During out-of-plane motion the riser will encounter resistance from the sides of any trench that has formed, or soil berms lying to either side of the pipe.

9.2.3   Design for fatigue

The stress ranges used in the fatigue analysis of SCRs are calculated from the changes in riser stress caused by first and second order motions. Within the touchdown zone (TDZ) these motions can be simplified to moving the touchdown point (TDP) in-line with the riser and assessing the resulting changes in bending moment. A sketch of the change in maximum pipeline stresses arising from bending moments (Figure 5 shows stresses, not moments) in the TDZ due to example riser motions with both high and low values of soil stiffness is shown in Figure 5.
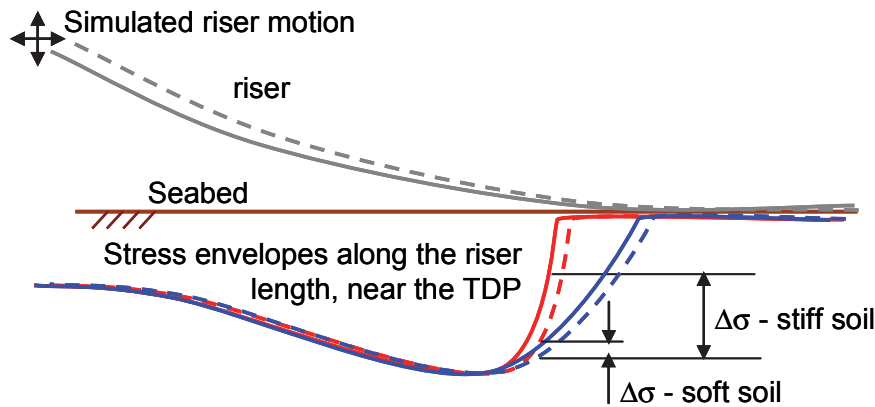
Simulated riser motion

riser

Seabed

Stress envelopes along the riser length, near the TDP

$\Delta\sigma$ - stiff soil

$\Delta\sigma$ - soft soil

Figure 5 – Example stress changes for fatigue calculations [212]

The cyclic stress range in the TDZ depends on the rate of change of the bending moment and thus the shear force. Analysis shows that the maximum shear force varies approximately linearly with the logarithm of the soil stiffness. Fatigue laws follow a power law relationship, with damage proportional to a high power (typically about 5) of the cyclic stress amplitude [229]. Even relatively minor differences in the shear force can therefore have a significant effect on the estimated fatigue life, and hence the non-linear response of the soil needs to be considered.

Either small or large waves can dominate the fatigue damage in the touchdown zone. The majority of fatigue damage can occur from either large waves (not necessarily the most extreme) with low probability of occurrence or continuous motions from small day-to-day waves.

9.2.4   Seabed-riser response in vertical plane

9.2.4.1   Background

Riser interaction with the seabed involves complex non-linear processes including plastic penetration during initial touchdown, softening during cycles of upward and downward motion and potential suction-induced tensile resistance prior to breakaway. In most cases, design is undertaken using simplified models where the riser-soil interaction is idealized by a series of linear springs, with zero tension capacity, distributed along the riser throughout the touchdown zone. Ideally, the choice of spring stiffness should consider the amplitude of vertical displacement and other effects such as the cyclic motion of the riser. While the soil response will also be affected by out-of-plane motion of the riser, the discussion here is restricted to vertical stiffness of the seabed.

Bridge et al. (2004) gave the conceptual description of the seabed resistance shown in Figure 6 for a robust load cycle involving soil-riser separation. Following initial riser penetration into the seabed, unloading occurs as the pipe is uplifted. The soil response in the early stages of uplift is much stiffer than that under conditions of virgin penetration as shown in the 'unloading' curve in Figure 6. With continued uplift the net resistance force goes into tension ('pipe-soil suction' in Figure 6) until maximum uplift resistance of the soil is reached and the pipe begins to detach from the soil. Uplift resistance decreases until the pipe completely detaches from the soil. Upon re-penetration the pipe comes back into contact with the soil, with the re-loading stiffness typically being less than the unloading stiffness. Upon completion of a full load cycle, the load path does not return to the initial point of departure from the backbone curve; rather the pipe penetrates a small additional depth into the soil.
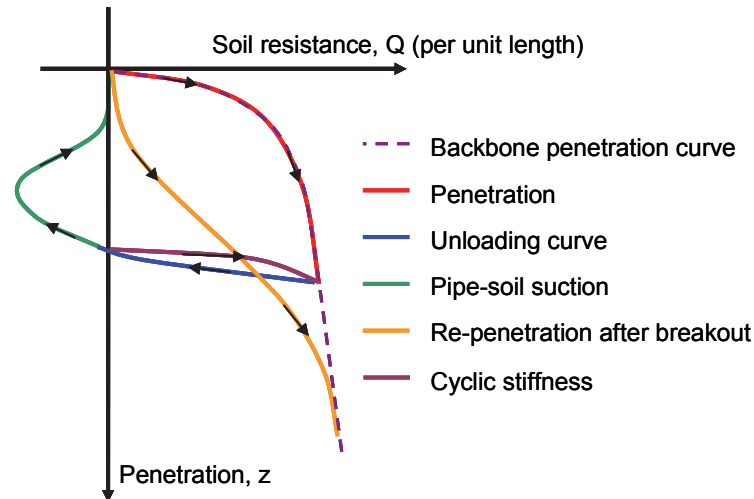
Figure 6 – Conceptual diagram of seabed stiffness [213]

NOTE   The uplift resistance is referred to here as 'suction', although, strictly speaking, under submerged conditions pore pressures will normally remain positive. For consistency with much of the published literature, the term 'suction' is retained, understanding that it refers to a net upward force acting on the seabed.

9.2.4.2   Plastic penetration resistance

The backbone penetration curve in Figure 6 may be estimated by considering the seabed strength profile and an appropriate bearing capacity factor for a given penetration. For conditions where the soil strength profile increases approximately linearly with depth, the limiting penetration resistance per unit length may be expressed as shown by Equation 29 [210]:

$$Q_u = N_c \, s_u \, D \approx a \left(\frac{z}{D} \right)^b s_u \, D$$

(29)

where

s_u is the shear strength at the pipe invert;

D is the pipe diameter;

z is the depth to the pipe invert;

N_c is a bearing capacity factor;

a is a parameter fitted to results of finite element analyses, with an average value of ~6;

b is a parameter fitted to results of finite element analyses, with an average value of ~0,25.

NOTE 1   See Reference [239] regarding parameters a and b.

Allowance for buoyancy effects should also be included.

NOTE 2 s_u refers to an average shear strength (between that measured in triaxial compression, extension and simple shear), or that deduced from a field penetrometer test such as the T-bar.

In certain regions of the world, a crust of higher strength soil exists in the upper 0.5 to 1 m, before the strength profile reverts to a linear trend. The potential for the SCR to punch through the crust, and the consequences for fatigue studies, deserves careful consideration.

9.2.4.3   Secant stiffness

The soil resistance behavior depicted in Figure 6 may be characterized in terms of equivalent springs having secant stiffness k_v supporting the riser pipe; the secant stiffness k_v in the vertical plane is defined by Equation 30.

$$k_v = \frac{\Delta Q}{\Delta z}$$

(30)

where

ΔQ is the change in vertical force per unit length of pipe;

Δz is the change in vertical displacement.

The non-linearity of the riser-soil interaction will lead to a variation in seabed stiffness along the length of the touchdown zone, which may be estimated based on the soil strength profile $s_u(z)$ and the predicted trench geometry, i.e. trench depth as a function of distance within the touchdown zone. The spatial variation in seabed stiffness is also affected by the temporal variability of the actual point of touchdown.

A hyperbolic model has been proposed for estimating the soil stiffness up to the point at which maximum penetration or maximum suction (uplift) is mobilized [211] [213]. The model may be expressed as shown in Equation 31.

$$K = \frac{k_v}{N_c \, s_u} = \frac{f \, K_{max}}{f + K_{max} \, \Delta z / D}$$

(31)

where

N_c is a bearing factor defining the backbone curve during virgin penetration;

K_{max} is the maximum value of the normalized secant stiffness on initial unloading or reloading;

D is the pipe diameter;

f is the asymptotic value of $\Delta Q/N_c s_u D$ at large displacements (i.e. $f \sim I \, (Q_{initial} - Q_{limit}) \, I/N_c s_u D$, where Q_{limit} is Q_{max} for penetration or suction),

NOTE   For soft clays, Reference [213] suggested a value of K_{max} of about 250, which is consistent with the first load cycle of small amplitude laboratory model tests in kaolin [224].

9.2.4.4   Uplift and breakaway

When the riser pipe is continuously uplifted, a maximum soil uplift resistance is reached, after which uplift resistance declines and breakaway of the pipe from the seabed occurs. The resistance of the soil to uplift can lead to bending stresses in uplift exceeding those in lay-down. In contrast, separation of the pipe from the soil tends to relieve bending stresses in the pipe. Accordingly, realistic estimates of the magnitude of maximum suction resistance and the displacement levels associated with suction mobilization and when breakaway occurs are important to characterize soil-riser interaction accurately.

Soil resistance to uplift of the riser pipe can arise from two mechanisms. The first is the ability of the soil to resist suction. The second is the resistance mobilized by backfill of soil above the pipe created by processes such as deposition or collapse of the side walls of the trench.

The maximum suction force per unit length of pipe, Q_{smax}, depends on a number of factors such as the effects of cyclic movement, the pipe velocity and the time over which the uplift resistance is sustained. These may be expressed through factors applied to the (compression) bearing capacity as shown in Equation 32 [213].

$$Q_{smax} = f_c\, f_V\, f_t\, N_c\, s_u\, D$$

(32)

where

f_c is a dimensionless cyclic factor;

f_V is a dimensionless velocity factor;

f_t is a dimensionless time factor.

NOTE   For recommendations for f_c, f_V, and f_t refer to Reference [213].

For conditions of cyclic and fatigue loading, Reference [213] recommended the use of a remolded strength s_{ur} rather than intact strength s_u, in addition to a cyclic reduction factor f_c, although it may be more consistent to use the original intact shear strength as the benchmark, relying on the various factors to quantify adjustments in estimating Q_{smax}.

Video surveys have shown that risers often cut a trench of significant depth in the seabed, as is considered further in Clause 9.5. Additional uplift resistance will occur where partial backfilling of the trench occurs, leaving the pipe embedded. The trench backfill is likely to be a product of mixing with water as well as remolding; therefore, its strength is likely to be less than the remolded strength of the seabed soil.

For conditions of no trench backfill with uplift resistance being mobilized purely from suction, Reference [213] proposed the following relationship shown by Equation 33 for breakout displacement Δz_b, i.e. the uplift displacement at which the pipe completely detaches from the soil, measured from the point at which the net force, Q, becomes negative.

$$\Delta z_b = \zeta_V\, \zeta_t\, D$$

(33)

where

ζ_V is a dimensionless velocity factor;

ζ_t is a dimensionless time factor.

NOTE   Reference [213] provides recommendations for the factors ζ_V and ζ_t based on STRIDE and CARISIMA data.

For the case of a backfilled trench, general relationships analogous to Equation 33 have not appeared in the published literature. However, relationships for uplift-displacement behavior have been developed on an ad hoc basis for specific sites (e.g. Reference [242]).

9.2.4.5   Stiffness adjustment for cyclic loading

Cyclic loading is recognized to degrade the soil stiffness. Based on cyclic model pipe-soil tests in kaolin, Reference [224] reported values of normalized cyclic stiffness, K, of less than 5, in contrast to monotonic K values ranging from approximately 250 (see 9.4.3) down to 40 at $\Delta z/D$ about 0.025. Model tests also suggested that the stiffness reduces by a factor of 10 to 20 where soil-riser separation and re-contact occurs [225].

The magnitude of the distance along the pipe over which soil-riser separation occurs will vary with water depth, riser properties, type of floating facility and environmental conditions. Table 6 summarizes typical motions of the riser in the touchdown region for different storm conditions for spar platforms in the Gulf of Mexico. For typical riser diameters of 0.3 m to 0.4 m, these distances correspond to about ±15 m for day-to-day wave loading, and ±25 m for extreme storm conditions. Transverse or out-of-plane vessel motions can lead to trebling of the range of separation.

Table 6 – Summary of distance and occurrence of SCR TDP motions [242]

Motion	Probability of occurrence	Limit of in-plane TDP motions	Limit of transverse TDP motions
Day-to-day	95 %	±43D	±0.5D
Extreme storm	2.5 %	±70D	±1D
Second order vessel motions	2.5 %	±200D to ±260D	±7D

On the catenary side of the mean TDP, the relevant soil stiffness may be taken as that associated with large displacements, and hence an order of magnitude lower than the maximum unload-reload stiffness. On the flowline side of the TDP, the gradual decay in magnitude and frequency of the motions suggests that, in the zone between 'day-to-day' and 'extreme storm' conditions in Table 6, the 'average' operative value of stiffness should be increased in steps towards the maximum value. Histograms of the vertical SCR motion can assist in assessing the magnitude and spatial variation of stiffness decay.

In practice, software used for assessing fatigue damage may be limited in the extent to which a spatially varying seabed stiffness can be implemented. An equivalent effect can be achieved with a suitable non-linear model for riser-seabed interaction, incorporating a relatively soft backbone curve and higher unload-reload stiffness, as indicated in Figure 6 Such non-linear models are now starting to become available in commercial software packages for riser design.

9.2.5   Trenching

9.2.5.1   Process

Riser movements at the touchdown zone produce seafloor trenching that affects stress analysis and fatigue performance of the riser. Trenching is a process of soil scour and plowing due to riser responses to global motions of the floating system. The process generally takes durations of months to years of cumulative soil displacements to form trenches, but may be accelerated with greater incidence of environmental events, heave-prone vessels or vortex-induced vibration.

9.2.5.2   Features

Observations of riser trenches reported by Reference [214] reveal a typical, ladle-shaped profile that tends to be deepest near the nominal touchdown point and gradually slopes up to the natural seafloor towards the flowline. The riser is often buried inside the trench in the region of the mean touchdown point. In plan, trenches are generally shaped as a bell mouth, with the widest portion on the vessel side of the touchdown and progressively narrowing towards the flowline. A generalized trench shape is provided in Figure 7.

Trench dimensions for a specific riser depends on several factors, including the following:

a)   soil conditions;

b)   riser dimensions and configuration (e.g. pipe-in-pipe);

c)   environmental conditions;

d)   time after installation;

e)   vessel type.
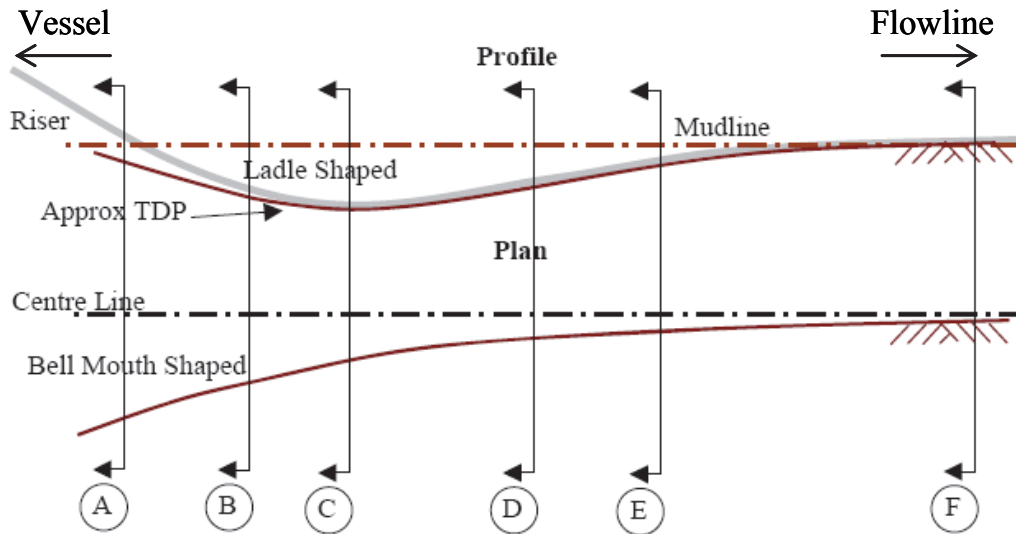
Figure 7 – Overview of Generalized Trench [214]

9.2.5.3   Influence on analysis

The impact of a trenched seafloor on SCR touchdown region fatigue performance is not well defined due to competing factors related to trench geometry and soil conditions. In contrast, trenching may cause large bending stresses in the riser for transverse motions at or near the touchdown. The riser may be overstressed in cases of abrupt interaction with a trench during extreme environmental events or large transverse or out-of-plane motions. Also, persistent transverse motions inside a trench by prevailing environmental conditions, second-order motions and vortex-induced vibration can adversely impact the fatigue life at or near the riser touchdown.

9.2.6   Three-dimensional motion

Under extreme conditions, such as for broken mooring lines, transverse or out-of-plane motions of the vessel or platform can lead to 3-dimensional motion of the riser sufficient to cause it to 'break out' from the current trench. Consideration should therefore be given to wide sweeps of the nominal SCR touchdown point as the surface vessel translates through large horizontal distances, which may be in the range 5 % to 10 % of the water depth. This distance is likely to be larger than the largest trench width considered in previous sections. The surface vessel may swing back and drag the riser large distances past the original SCR trench, before reaching equilibrium at a position that is not quantifiable in a failure scenario like this.

Under normal (i.e. with intact moorings) operations in major storms and large current events, it is also possible for the floating support structure to lift an SCR out of its trench and set it down in virgin soil many trench widths away as the surface and subsea environmental conditions build up, possibly change direction, and then ease off.

In either of the above situations a new trench may form at the new location, which will affect the fatigue life of the riser. A more critical issue, however, is that the break-out resistance from the original trench may result in high localized bending moments and a potential ultimate limit state. The effect of the high break-out resistance during three-dimensional motion of a trenched riser should be quantified.

9.3   Top tension riser

9.3.1   Introduction

The design of top tension risers near or below the blowout preventer (BOP) stack is dependent on the riser (conductor)-soil interaction. The following two design issues should be considered:

a)   the reaction of the riser at the ultimate limit state when the host facility has moved a considerable distance from the mean position;

b)   the fatigue that occurs within the riser as a result of repeated cyclic motions with a range of amplitudes and frequencies.

The first problem occurs during a move-off condition or during environmental loading. The move-off case can be an intentional static type of condition that occurs when the host facility is moved to facilitate drilling operations or when the host temporarily loses station keeping capabilities.

The fatigue problem is governed by cyclic loads that occur throughout the life cycle of the riser-conductor system. These loads can occur from:

—   environmental wave, wind and current loads on the host facility;

—   vortex induced vibrations (VIV) on the host facility or riser;

—   environmental loads (wave and current) mostly in the top portion of the riser.

Analyses have shown that peak loads are not the major contributor to the fatigue damage. Rather, smaller more frequent loads are responsible for most fatigue damage. Therefore, characterization of the soil response at small amplitude displacements is particularly important for the fatigue problem. Previous work indicates that the criteria in API 2A-WSD, 21st Edition, may underestimate the lateral stiffness of the soil, especially at low amplitude displacements. It is often difficult to assess whether soft or stiffer soil stiffness estimates are the more conservative. Stiffer estimates suggest that the critical cyclic bending moments will occur above the mudline while softer estimates would suggest the opposite.

Finally, temperature effects will also have an impact on the conductor. Increased temperatures can result in:

—   upheaval forces on the conductor,

—   changes in the shear strength of the soil around the conductor, or

—   dissociation of seafloor hydrates.

The impact of temperature changes on the soil properties along the conductor is likely to be more severe for axial loads. However, the axial capacity is quite robust due to the length of the conductor and associated casing strings. The impact on lateral loads is mitigated due to the constant temperature condition imposed by seawater at the mudline.

9.3.2   Soil response

The response of the near seafloor portion of top tension riser-conductors to fatigue loading is highly dependent on soils to about 15 m to 18 m. below the mudline. The overall challenge of assessing this lateral response is analogous to that of a laterally loaded pile. Accordingly, lateral soil springs provided for offshore piles have often been used to assess the lateral response of a conductor. Despite the apparent similarities, the following differences exist.

a)   The soil springs for piles were originally developed for steel jackets subjected to large storm loads. As such, the primary intent was to characterize the soil near yield while less attention was paid to the soil response at smaller displacements.

b)   The maximum moments developed in a pile are relatively insensitive to the lateral soil response. Differences in the soil springs will tend to change the location rather than the magnitude of the maximum moment. Since the wall thickness for offshore piles is usually constant, inaccuracies in the *p-y* springs will have a lesser impact. Top tension risers-conductors, however, are assembled in sections with connectors. The connectors are critical locations for fatigue.

c)   Recommendations from API 2A-WSD, 21st Edition, and the supplement/errata published in 2007 were based on tests with loads applied over a few days while the loads that cause fatigue are applied over a much shorter time period.

d)   The relationship between cyclic bending moment and fatigue life is highly non-linear so that with many cycles fatigue life is highly sensitive to soil stiffness.

9.3.3   Development of *p-y* springs via finite element (FE) analyses

Instead of relying exclusively on API 2A-WSD, 21st Edition, recommendations for lateral load analyses of riser-conductor systems, the *p-y* springs can alternatively be developed using the finite element method. The following provides guidelines for this approach.

An important aspect of developing *p-y* springs with the finite element method is developing a representative soil model for riser-conductor problems. The small strain or initial shear modulus is an important part of this soil model. The initial shear modulus, G_{max}, can be determined from resonant column testing on samples taken during the site investigation. Samples should be taken from depths where the maximum fatigue damage might occur, for example the upper 15 m. of soil. The resonant column test results should be adjusted for actual field conditions, for example using the correlations proposed in Reference [231].

Values of G_{max} obtained from resonant column tests should also be modified to account for:

—   the increase in shear modulus that occurs after primary consolidation;

—   the reduction in shear modulus due to the lower rate of loading expected for riser-conductor types of loading, compared to the higher rate in the resonant column tests; and

—   cyclic degradation, with the influence on steel stress and fatigue behavior being bracketed between pre- and post-cyclic behavior of the soil mass.

The increase per log cycle in time due to secondary consolidation has been estimated as proportional to the square root of the soil plasticity index [231]. This increase will be offset in part by the reduction due to the lower rate of loading which, in the absence of site specific data, may be approximated as a 10 % reduction in shear modulus per log difference in frequency relative to the resonant column tests.

Alternately, Reference [207] presents the following for a normalized shear modulus (G_{max}/s_{uDSS}) for normally consolidated clays:

$$\frac{G_{max}}{s_{uDSS}} \approx \frac{300}{PI/100} \tag{32}$$

where

s_{uDSS} is the undrained shear strength from direct simple shear tests;

PI is the plasticity Index of the soil.

Generally, values of normalized shear modulus for over-consolidated clays are lower than those for the normally consolidated data [207].

Reference [230] shows example *p-y* curves developed with the finite element approach compared with API 2A-WSD, 21st Edition, recommendations (Figure 8 [230]). Also shown on the figure are centrifuge results for tests where the load was applied in less than one minute. The green and black curves show the direct comparison between the centrifuge results and finite element model. The blue curve represents API 2A-WSD, 21st Edition, recommendations.
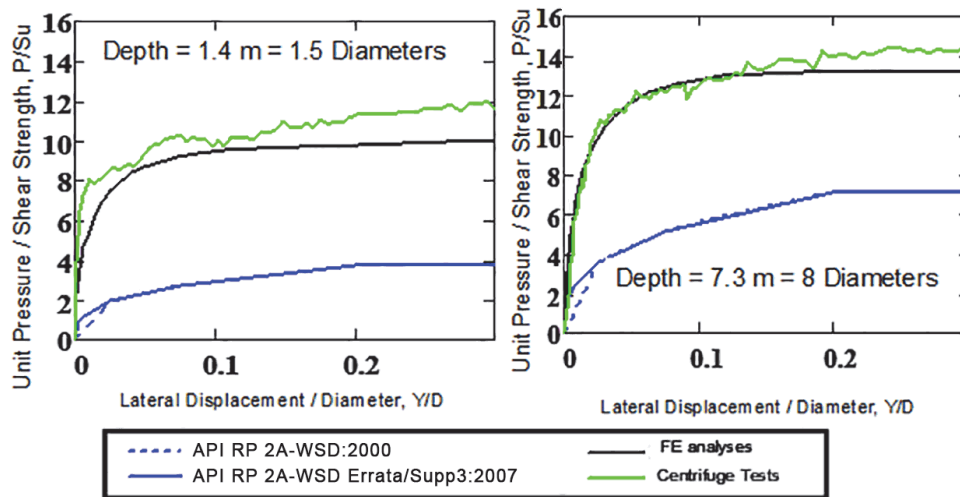
Figure 8 – Comparison between *p-y* springs measured in centrifuge tests on kaolinite and computed using the FEA technique with the API 2A-WSD, 21st Edition, recommendations [230]

Additional details for developing *p-y* curves through numerical modelling procedures are described in Reference [241].

9.3.4　Additional considerations

In addition to soil stiffness at small strains the soil should also include effects of work hardening. This work hardening results from either elasto-plastic isotropic hardening of the soil at larger strains or kinematic hardening from cyclic loading.

Figure 9 [230] presents results that demonstrate this potential impact. This figure shows that cyclic loading can degrade the soil stiffness. If the static *p-y* backbone curve is used in the analysis at the mid-point for the cyclic load, the actual unload-reload secant stiffness (as is used in several riser fatigue software products) is greater. Therefore, using the tangent stiffness from the backbone curve can produce conservative results (more fatigue damage) when the critical fatigue point is below the mudline. However, if the critical fatigue point is above the mudline, softer soil springs can result in a non-conservative solution.
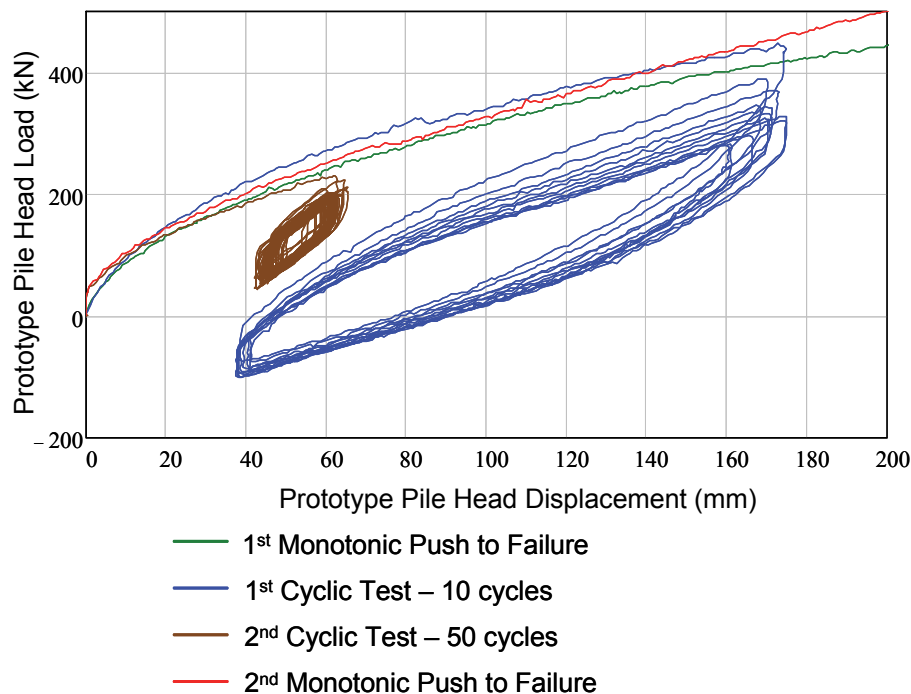

Figure 9 – Expanded view of pile head displacement versus pile head load [230]

9.3.5 Summary and recommendations for top tension risers

The following lists a summary and recommendations for top tension risers.

a) Top tension riser-conductor design should consider both ultimate and fatigue limit state. Because of variable wall thicknesses and irregular geometries, the critical bending moment can occur either above or below the mudline.

b) The p-y curves specified by API 2A-WSD, 21st Edition, criteria for piles provide a significantly softer response than p-y curves developed via finite element analyses. Advanced soil models should be used based on site-specific soils data that captures both small displacement soil behavior and work hardening from cyclic loading.

c) Although finite element modelling produces a stiffer soil response, physical model tests demonstrate cyclic softening with repeated load cycles.

d) Whenever the critical fatigue point is below the seafloor, the tangent stiffness from adjusted static p-y curves based on finite element analyses can be conservative, because the tangent stiffness at the mid-point of cycling can be less than the load-unload stiffness. If the critical fatigue point is above the mudline the p-y curves given by API 2A-WSD, 21st Edition, can be non-conservative. For this case the amount of cyclic degradation should be conservatively estimated, i.e. less degradation.

e) Developing p-y curves with finite element analysis requires expert skills and is time consuming. If the critical bending moments are below the mudline, the curves given by API 2A-WSD, 21st Edition, can be initially used. If this initial attempt leads to unacceptable failure levels, then the site-specific data can be used to determine the benefit from increasing soil stiffness.

f) For a drilling riser with heavier lower stacks (LMRP, BOP), the soft soil (low soil stiffness) may cause the system natural frequency to move into wave energy zone. So using stiffer soil (or fixed at mudline) does not guarantee the results to be in the conservative side even if the critical fatigue point is above the mudline.

9.4 Riser tower foundations

9.4.1 Introduction

The riser tower concept consists of a free-standing riser assembly, incorporating several risers in a bundle configuration, tensioned from the top by a buoyancy tank, and anchored to the sea-bed. The connection to the surface vessel or platform is generally ensured by flexible jumpers.

A riser tower supports axial tension generated by buoyancy and by cyclic wave action, and is secured into the sea-bed. A significant part of the tension acts permanently during the life of the development.

9.4.2 Foundation options

There are a number of possible foundation options: gravity base, suction caissons and driven piles.

The concept selection for the foundation should be based on technical and economic criteria, taking into account the soil data, the installation aspects, as well as the in-place performance. A preliminary conceptual study should be performed to select the most appropriate foundation type.

9.4.3 Loads and safety factor

9.4.3.1 Loads

Design loads should be evaluated for the following conditions:

— foundation installation and retrieval;

—   normal operating conditions;

—   extreme conditions.

Load combinations should be selected so as to anticipate the most unfavorable result, for each of the stability mechanisms and deformation analyses performed.

9.4.3.2   Recommended safety factors

Safety factors should be as recommended by API 2T for driven piles and gravity base anchors and by API 2SK for suction caissons.

NOTE 1   The API 2SK safety factors were developed with NO consideration of permanent uplift loads on suction caissons. Reference [223] provides information on the potential uplift response under sustained loading.

NOTE 2   It should be noted that API 2T does not address the case of gravity loads explicitly. Additionally, Section 10.3.3 of API 2T states that:

> For axial pile design where the weight of the foundation system is less than approximately 10 % of the ultimate axial capacity, the underwater weight of the foundation system may be subtracted from the applied loads in determining the safety factor of the foundations. For other weight-dominated systems, the foundation system weight should be added to the resistance side of the equation.

9.4.4   Soil design parameters

High quality in-situ measurements and/or soil borings should be obtained to select the soil design parameters.

The depth of the geotechnical borings should exceed the foundation depth by at least 3 anchor diameters unless a more regional site characterization has shown that there are no major changes in stratigraphy within that depth. If there are no major changes in stratigraphy, the depth beneath the anchor tip may be reduced to 1 anchor diameter. It is critical to establish whether any high permeability layers are present within or above the zone of influence for reverse end bearing under sustained loading.

The number of borings should be defined as a function of the soil variability. One boring should be performed at each anchor location when unusually large lateral variability of the soil properties is expected.

The main soil properties that are needed for the foundation design are defined in API 2SK.

9.4.5   Design issues

9.4.5.1   General principles

The following general principles are to be considered in assessing the stability of tower riser foundations.

a)   Limit equilibrium methods can generally be used to evaluate the capacity of riser tower foundations. The shear strength used in the analysis should account for effects of creep and potential drainage under sustained load and cyclic degradation. The reduction in effective stresses and shear strength due to potential drainage can be studied by finite element analyses.

b)   Due consideration should be incorporated into the design regarding displacement and deformation during the life of the foundation. Where displacement and deformation are critical, complex analysis methods may be warranted. The displacement analysis should include contributions from undrained shear strains due to application of the sustained load, undrained creep during the sustained load, and permanent and cyclic components from the wave loading. Displacements due to shear strain, volumetric strains and flow of water through the soil due to potential drainage during the sustained load period should also be considered.

c)   The anchors should be installed within specified tolerances of tilt and mis-orientation. The design analyses should account for the effect of the tolerance limits.

d) Installation should be planned so as to ensure the foundation can be properly seated at the intended site, without excessive disturbance to the supporting soil. Where excessive disturbance does occur, this should be considered in the assessment of foundation capacity.

e) Measures should be taken to avoid erosion and scour of the soil beneath or near the foundation base.

Where removal is anticipated, an analysis should be made of the forces generated during removal to ensure that removal can be accomplished with the means available.

9.4.5.2   Geotechnical design methodology for the foundation

The design of driven piles and gravity base is covered by the recommendations in API 2T. The design of suction caissons is covered by the recommendations in API 2SK, with more detailed aspects considered in the literature [205], [206], [208], [219], [220], [221], [222], [223], [227], [228], [238]. The following aspects should be considered:

⎯ penetration and retrieval;

⎯ holding capacity including long-term uplift capacity;

⎯ long-term displacement;

⎯ soil reactions to be used for the structural design.

The capacity should be checked both for the permanent load and for the sum of permanent and cyclic loads. If the cyclic loads are small compared to the permanent loads, the permanent load condition may be critical because the strength can be smaller for this condition than for the condition where the cyclic load is included. The following should be considered:

a) The undrained shear strength for the permanent load condition should be reduced to account for creep effects (e.g. Reference [234]). The effect of pore pressure redistribution and swelling should also be considered, since this may lead to reduction in effective stresses and undrained shear strengths, and hence reduce the capacity under transient wave loading (Reference [208]). In cases where the permanent load acts for months, it is also necessary to consider whether drained conditions may develop, and the extent to which full base suction can be maintained. The possibility of drainage channels between inside skirt stiffeners, above inside stiffeners and extended skirt wall thicknesses, and along open cracks outside the anchor at the active side of the anchor should be considered. References [223] and [226] show that the capacity of a typical suction caissons under sustained loading may be only 70 % of the capacity for rapid loading.

b) Drainage and pore pressure redistribution can also influence the undrained shear strength under cyclic loading, but the shear strength for the sum of permanent and cyclic loads will be higher than for the permanent load due to rate effects (e.g. Reference [209]).

The potential of gapping along the wall above stiffeners or sections with increased wall thickness should be evaluated. In cases with ring stiffeners it is necessary to evaluate the potential of trapped water between ring stiffeners (e.g. Reference [206]). If gapping or trapped water is possible, the effect that such gaps will have on the drainage path and on resistance due to lack of contact along the wall should be considered.

The capacity of suction caissons may depend on passive underpressure inside the caisson. If passive underpressure is relied upon, proper sealing may be critical, especially for the part of the underpressure generated by long period environmental loads such as loop currents. The anchor top may be sealed, but if the valve seals cannot be guaranteed, consideration should be given to either a back-up cap behind the valves or a monitoring program to assure the desired integrity over the lifetime of the suction caissons. If proper sealing is not ensured, the suction caissons should be designed to resist sustained uplift load without taking passive underpressure into account.

9.4.6   Inspection and monitoring

An inspection program should be considered as integral part of the foundation design. The inspection program should include the use of instrumentation to monitor critical aspects of the foundation performance, during both installation and operation.

If at any time during the service life of the structure, the inspection program reveals conditions, or behavior, which are detrimental to the integrity of the foundation or structure, then maintenance or remedial action should be carried out as necessary.

9.5   Flowlines and pipelines

9.5.1   Introduction

Pipeline design should consider ultimate and fatigue limit states related to the stresses in the pipeline and the movements of the associated end connections including sections which transition into a catenary riser. The response is influenced by the geotechnical interaction forces between the pipeline and the seabed as well as other external loads on the pipeline and internal loads within the pipeline.

Geotechnical advice should be sought to predict the as-laid pipe embedment and the resulting force-displacement response in the axial and lateral directions. The most basic forms of axial and lateral pipe-soil model are linear elastic-perfectly plastic. More sophisticated designs require more complex pipeline-soil models. Either upper or lower bound values of the pipe-soil interaction forces can be critical for a limit state. Each bound should be assessed as necessary.

9.5.2   Loads on seabed pipelines

The loads and motions imposed during laying govern the pipeline embedment and any residual tension at the start of operation and initial in-service performance. Hydrodynamic loading and subsequent scour and seabed liquefaction processes can lead to changes in embedment during the operational life of the pipeline.

After installation, the loads imposed on an individual element of pipeline are balanced by reaction forces from the soil. The loads on the pipeline element arise from adjacent or bounding elements of the pipeline, a connected SCR, internal and external pressure, hydrodynamic or thermal loading and internal and external pressure. A pipeline may also be susceptible to external loading from debris flows and turbidity currents that arise from submarine slides and snag or impact loading from foreign objects. The compressive axial force created by operating cycles of internal pressure and temperature can lead to lateral buckling of the pipeline, or the accumulation of axial movements (pipeline 'walking').

9.5.3   Soil reaction forces

9.5.3.1   Pipe-soil interaction models

The interaction between the pipe and the seabed is incorporated into the structural analysis of a pipeline by attaching pipe-soil model elements at intervals along the pipe. This approach is analogous to the 't-z' and 'p-y' load transfer methods of analyzing pile response.

For some simple pipeline design functions, pipe-soil response is represented by limiting values of axial or lateral pipe-soil resistance or bi-linear-elastic – perfectly-plastic behavior in the axial and lateral directions. The pipe-soil resistance is typically expressed as an equivalent friction factor, linking the limiting resistance to the effective pipeline weight. However, the axial and lateral resistances can depend on factors other than the pipe weight, in particular the embedment. Therefore, a friction factor is not an intrinsic soil property.

To capture the more complex effects of interaction – particularly the large displacement behavior – it is necessary to model other aspects of the response, including brittle breakout behavior and cyclic berm growth during lateral movement (Bruton et al. 2007).

9.5.3.2   Drained and undrained soil behavior

In fine-grained sediments, pipe laying is usually an undrained process. Dissipation of the lay-induced excess pore pressure typically takes days or weeks. Lateral pipeline movements generally involve undrained deformation although consolidation between events may cause disturbed soil to regain strength. Axial pipe movements may be drained or undrained, since the relevant drainage distances are shorter than for lateral movement.

In coarse-grained sediments, pipeline installation and operation will generally occur under fully drained conditions. In a design analysis, the anticipated rates of axial and lateral pipeline movement should be compared with the relevant rates of soil drainage and consolidation to establish whether drained or undrained conditions will prevail.

9.5.4   Analysis of pipeline-soil interaction

9.5.4.1   Vertical penetration

9.5.4.1.1   Lay effects

Observations show that the as-laid pipeline embedment is typically much greater than would be expected from the static weight alone, due to over-stressing and dynamic motion in the touchdown zone during laying [245]. The contact bearing stresses (or vertical force per unit length) between the pipe and the soil in the vicinity of the touchdown point exceeds the as-laid self-weight of the pipe due to the catenary shape. The degree of over-stress is dominated by the bending rigidity of the pipeline, the apparent stiffness of the seabed, and the tension in the pipe in the vicinity of the touchdown point [239]. The applicable apparent stiffness of the seabed is a secant stiffness, for the anticipated degree of plastic penetration, and can be much lower than that customarily used for fatigue assessment within the touchdown zone of an SCR. In deep water, the overstress may be negligible because of soft seabed conditions. If the pipeline is laid empty then the maximum static loading may occur during hydrotesting, when the pipeline is heavier [249]. The as-laid embedment may therefore be governed either by the as-laid weight (with a touchdown overstress and dynamic lay effects) or by the hydrotest condition.

Vessel motion, changes in pipeline tension, and hydrodynamic loading of the hanging pipe, will induce a combination of vertical and horizontal motion of the pipeline at the seabed during laying [233] [218] [245]. Even small lateral or vertical movements can cause disturbance, local softening and erosion of the seabed in the touchdown zone, increasing the pipe embedment.

9.5.4.1.2   Static vertical penetration response

For seabed sediments where drained conditions will prevail, conventional bearing capacity approaches may be used to estimate the static pipeline penetration. The pipeline may be treated as a surface strip foundation of width equal to the (nominal) chord length of pipe-soil contact at the assumed embedment. In most cases, however, the penetration resistance will be such that minimal embedment of the pipeline will be predicted based on static loading. Other processes such as cyclic motion of the pipeline, and scour and partial liquefaction of the seabed will determine the as-laid embedment depth.

For fine-grained sediments, where undrained conditions prevail during embedment, theoretical solutions for estimating the pipe penetration resistance have been provided in References [236], [210], and [240]. These solutions use the conventional bearing capacity equation, modified for the curved shape of a pipeline. In soft soils, the enhanced soil buoyancy created by heave may be significant [239].

9.5.4.2   Axial soil resistance

Axial pipeline movement involves shear failure at or close to the pipeline-soil interface. The vertical effective contact force can be used to calculate the effective stresses and forces at the pipeline-soil interface. The integrated normal contact stresses around the pipe periphery exceed the vertical contact force, V, due to the curved shape of the pipe surface (Reference [246]). Using the enhancement factor, ζ, to account for this effect, the drained axial resistance per unit length of pipeline, T, is given by Equation 34:

$$T = \mu N = \mu \zeta V \tag{34}$$

where μ is the pipeline-soil friction coefficient, which can be alternatively expressed in terms of an interface friction angle, δ, where $\mu = \tan \delta$, and N is the integrated normal contact force.

Based on an elastic solution, Reference [246] provides the following expression for ζ:

$$\zeta = \frac{N}{V} = \frac{2\sin\theta_{D'}}{\theta_{D'} + \left(\sin\theta_{D'}\cos\theta_{D'}\right)}$$

where $\theta_{D'}$ is the half-angle of the pipe-soil contact perimeter, which varies with normalized embedment, z/D, with D the pipe diameter, according to:

$$\cos\theta_{D'} = 1 - \frac{2z}{D} \tag{35}$$

Due to the stress-dependency of soil friction angle, this parameter should be assessed by tests conducted at the correct stress level [237] [246]. During undrained axial pipe movement, the apparent friction coefficient may increase or decrease depending on whether negative or positive excess pore pressure is generated by shearing at the interface.

Laboratory tests can be used to assess an appropriate friction coefficient, μ, for drained shearing including separate values for peak and residual resistance. Tests can be conducted in a low stress shear box or on a tilt table, using a sample of the pipe surface coating. Alternatively, a model pipe section can be tested in a larger test chamber. These tests should replicate the relevant lay-induced consolidation history, and the speeds and pause periods relevant to the design situation. It should be established whether undrained or partially drained conditions may apply during pipe motion. These conditions can lead to a significant reduction in the apparent friction coefficient, μ, compared to the drained case.

9.5.4.3   Lateral soil resistance

Lateral pipe-soil resistance during breakout and large-amplitude cyclic movement is influenced by the initial pipe embedment and weight, the development of soil berms ahead of the laterally sweeping SCR or pipeline segment, and the soil properties. Two characteristic types of large-amplitude lateral response are typically observed (Figure 10) depending on the ratio of the pipeline weight to the seabed strength, $V/s_u D$. For values of $V/s_u D$ below approximately 2 ('light' pipes), the pipeline tends to rise after breaking out from the as-laid position (Figure 10a). As the pipe rises, the lateral resistance reduces from the break-out value to a residual resistance. The pipeline sweeps horizontally with a berm of soil being pushed ahead of the pipe. This mechanism governs the residual resistance, H_{res}. Subsequent cycles of lateral movement lead to a steady increase in the restraint provided by the soil berms (Figure 10a). The 'light' pipe form of response is also typically observed in drained conditions.

For values of $V/s_u D$ greater than approximately 2 ('heavy' pipes), the pipeline typically moves downwards after the initial break-out resistance is mobilized. This downward movement, coupled with the growth of a soil berm ahead of the pipe, leads to a steady increase in the lateral resistance (Figure 10b).

Empirical expressions exist for predicting lateral pipe-soil resistance, which have evolved primarily through calibration against model tests [243] [244] [215] [246]. These expressions are subject to significant uncertainty, and their relevance should be established for a particular design situation. In undrained conditions, the break-out resistance, H_{brk}, is generally divided into two contributions:

— a component proportional to the current vertical load, V, (which is essentially the pipe weight);

— a passive resistance component, linked to the embedment depth of the pipe, z,

An alternative approach is to use yield envelopes (or interaction diagrams) in vertical and horizontal load space that bound the allowable load combinations for a given pipeline embedment [235] [240].
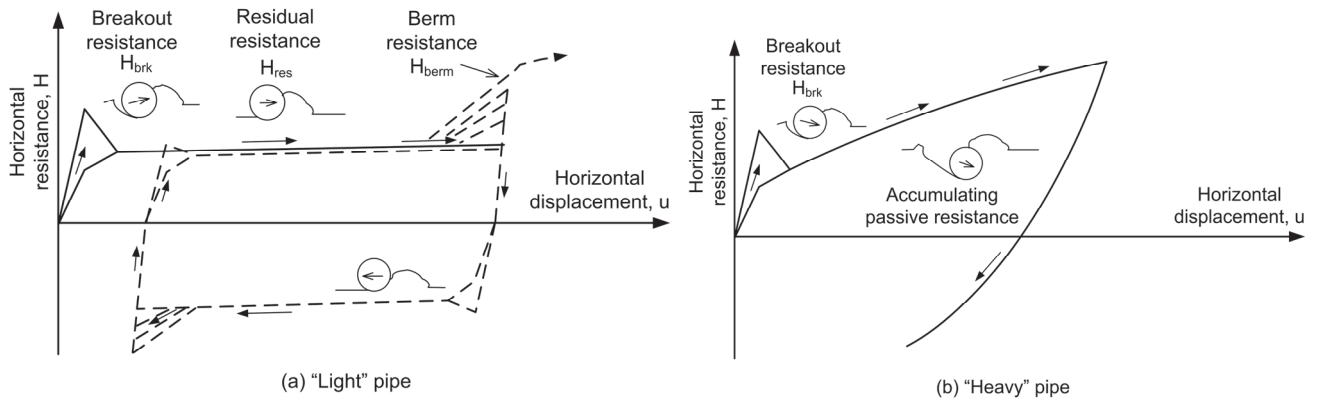
Figure 10 – Types of large-amplitude cyclic lateral response in undrained conditions

For assessment of fatigue performance of a pipeline during large amplitude cyclic movement, the size and strength of the growing berm formed ahead of the sweeping pipe segments and at the extremities of the pipe movement are significant [216].

Expressions for assessing lateral pipe-soil resistance at breakout and during cyclic motion can be calibrated and validated by laboratory models, centrifuge tests [232] [247], seabed test sections, or analytical methods.

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