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Entwurf

Einsprüche bis 2015-04-27
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Erdöl- und Erdgasindustrie – Besondere Anforderungen an Offshore-Bauwerke – Teil 4: Geotechnische und Fundament-Auslegungsmerkmale (ISO/DIS 19901-4:2014); Englische Fassung prEN ISO 19901-4:2014, nur auf CD-ROM

Petroleum and natural gas industries –
Specific requirements for offshore structures –
Part 4: Geotechnical and foundation design considerations (ISO/DIS 19901-4:2014);
English version prEN ISO 19901-4:2014, only on CD-ROM

Industries du pétrole et du gaz naturel –
Exigences spécifiques relatives aux structures en mer –
Partie 4: Bases conceptuelles des fondations (ISO/DIS 19901-4:2014);
Version anglaise prEN ISO 19901-4:2014, seulement en CD-ROM

Anwendungswarnvermerk

Dieser Norm-Entwurf mit Erscheinungsdatum 2015-02-27 wird der Öffentlichkeit zur Prüfung und Stellungnahme vorgelegt.

Weil die beabsichtigte Norm von der vorliegenden Fassung abweichen kann, ist die Anwendung dieses Entwurfes besonders zu vereinbaren.

Stellungnahmen werden erbeten

- vorzugsweise online im Norm-Entwurfs-Portal des DIN unter www.entwuerfe.din.de bzw. für Norm-Entwürfe der DKE auch im Norm-Entwurfs-Portal der DKE unter www.entwuerfe.normenbibliothek.de, sofern dort wiedergegeben;
- oder als Datei per E-Mail an noeg@din.de möglichst in Form einer Tabelle. Die Vorlage dieser Tabelle kann im Internet unter www.din.de/stellungnahme oder für Stellungnahmen zu Norm-Entwürfen der DKE unter www.dke.de/stellungnahme abgerufen werden;
- oder in Papierform an den DIN-Normenausschuss Erdöl- und Erdgasgewinnung (NÖG), 10772 Berlin (Hausanschrift: Burggrafenstr. 6, 10787 Berlin).

Die Empfänger dieses Norm-Entwurfs werden gebeten, mit ihren Kommentaren jegliche relevanten Patentrechte, die sie kennen, mitzuteilen und unterstützende Dokumentationen zur Verfügung zu stellen.

Gesamtumfang 217 Seiten

DIN-Normenausschuss Erdöl- und Erdgasgewinnung (NÖG)

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Nationales Vorwort

Dieses Dokument (prEN ISO 19901-4:2014) wurde vom Technischen Komitee ISO/TC 67 „Materials, equipment and offshore structures for petroleum, petrochemical and natural gas industries“ in Zusammenarbeit mit dem Technischen Komitee CEN/TC 12 „Materialien, Ausrüstungen und Offshore-Bauwerke für die Erdöl-, petrochemische und Erdgasindustrie“ (Sekretariat: AFNOR, Frankreich) erarbeitet.

Für Deutschland hat hieran der Arbeitskreis NA 109-00-01-07 AK „Offshore Bauwerke“ im DIN-Normenausschuss Erdöl- und Erdgasgewinnung (NÖG) mitgearbeitet.

Dieser Europäische Norm-Entwurf enthält unter Berücksichtigung des DIN-Präsidialbeschlusses 1/2004 nur die englische Originalfassung von prEN ISO 19901-4:2014 und ISO/DIS 19901-4:2014.

Das Ausgabedatum des Europäischen Norm-Entwurfs stand zum Zeitpunkt der Veröffentlichung dieses DIN-EN-ISO-Entwurfs noch nicht fest; der prEN ISO wird jedoch vom CMC unter der angegebenen prEN-Nummer demnächst zur CEN-Umfrage verteilt. Um der deutschen Öffentlichkeit eine möglichst lange Einspruchsfrist einzuräumen, wurde dieser Norm-Entwurf bereits vorab veröffentlicht.

Dieser Norm-Entwurf enthält neben den gesetzlichen Einheiten auch die Einheiten „ft“, „in (inch)“, „kip“, „lb (pound)“ und „lb/ft^2“ die in Deutschland nicht zugelassen sind. Es wird ausdrücklich darauf angewiesen, dass die Anwendung dieser Einheit im nationalen amtlichen und geschäftlichen Verkehr aufgrund des Gesetzes über Einheiten im Messwesen nicht zulässig ist. Die Angabe dieser Einheiten dient lediglich als Hilfe im amtlichen und geschäftlichen Verkehr (z. B. bei Einfuhr und Ausfuhr) mit solchen Staaten, die diese Einheiten anwenden.

Nicht-SI-Einheit	**SI-Einheit**	**Umrechnung**	
ft	m	1 ft	= 0,304 8 m
in (inch)	mm	1 inch	= 25,4 mm
kip	N	1 kip	= 4448,22 N
lb (pound)	kg	1 lb	= 0,453 592 37 kg
lb/ft^2	kg/m^2	1 lb/ft^2	= 4,882 kg/m^2

Für die in diesem Dokument zitierten internationalen Normen wird im Folgenden auf die entsprechenden Deutschen Normen hingewiesen:

ISO 19900	siehe	DIN EN ISO 19900
ISO 19901-2	siehe	DIN EN ISO 19901-2
ISO 19901-7	siehe	DIN EN ISO 19901-7
ISO 19902	siehe	DIN EN ISO 19902
ISO 19903	siehe	DIN EN ISO 19903
ISO 19905-1	siehe	DIN EN ISO 19905-1
ISO/TR 19905-2	siehe	DIN CEN ISO/TR 19905-2 DIN SPEC 67998
ISO 19906	siehe	DIN EN ISO 19906

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Änderungen

Gegenüber DIN EN ISO 19901-4:2005-07 wurden folgende Änderungen vorgenommen:

a) Erweiterung des Anwendungsbereichs auf „Bemessung und Bau von Pfahlgründungen“, „Boden-Bauwerk-Wechselwirkungen für Hilfsbauwerke, z. B. Unterwasser-Produktionssysteme, Riser und Förderleitungen“ und „Auslegung von Ankern für die Positions-Erhaltungssysteme von schwimmenden Anlagen“;

b) Neuer Abschnitt 8 „Pile foundation desgin“ und entsprechende Erweiterung des Anhang A;

c) Neuer Abschnitt 9 „Pile instalation assessment“ und entsprechende Erweiterung des Anhang A;

d) Neuer Abschnitt 10 „Soil-structure interaction for auxiliary subsea structures, risers and flowlines” und entsprechende Erweiterung des Anhang A;

e) Neuer Abschnitt 11 „Design of anchors for floating structures“ und entsprechende Erweiterung des Anhang A.

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Nationaler Anhang NA
(informativ)

Begriffe, Symbole und Abkürzungen

Die Benummerung der folgenden Begriffe, Symbole und Abkürzungen ist identisch mit der Benummerung in der englischen Fassung.

3 Begriffe

Für die Anwendung dieses Dokuments gelten die Begriffe nach ISO 19900, der Normenreihe ISO 19901 und die folgenden Begriffe.

3.1
Einwirkung
externe Last, die (als direkte Einwirkung) auf das Bauwerk aufgebracht wird oder dem Bauwerk (als indirekte Einwirkung) auferlegte Verformung oder Beschleunigung

Anmerkung 1 zum Begriff: Eine Verformung kann durch die Herstellungstoleranzen, durch unterschiedliche Setzung, durch Veränderung der Temperatur oder durch Feuchtigkeitsschwankungen verursacht werden. Ein Erdbeben führt im Allgemeinen zu Beschleunigungen.

Anmerkung 2 zum Begriff: Siehe ISO 19900.

3.2
Grundvariable
Variable aus einem festgelegten Satz von Variablen, die physikalischen Größen entsprechen und zur Kennzeichnung von Material- und Bodeneigenschaften verwendet werden

Anmerkung 1 zum Begriff: Siehe ISO 19900.

3.3
charakteristischer Wert
einer Grundvariablen zugeordneter Wert in Verbindung mit einer vorgegebenen Wahrscheinlichkeit, dass er während einer bestimmten Bezugsdauer nicht durch ungünstige Werte überschritten wird

Anmerkung 1 zum Begriff: Der charakteristische Wert ist der wichtigste repräsentative Wert.

Anmerkung 2 zum Begriff: Siehe ISO 19900.

3.4
Bemessungswert der Einwirkung
Kombination aus repräsentativen Einwirkungen und Teilsicherheitsbeiwerten entsprechend einer Bemessungssituation, der zur Überprüfung der Zulässigkeit eines Entwurfs verwendet wird

3.5
Bemessungswert
aus dem repräsentativen Wert abgeleiteter Wert, der im Verfahren zur Verifizierung des Entwurfs verwendet wird

Anmerkung 1 zum Begriff: In einigen Bemessungssituationen können zwei Bemessungswerte, ein oberer und ein unterer Wert, festgelegt sein.

Anmerkung 2 zum Begriff: Siehe ISO 19900.

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3.6
dränierte Bedingungen
Bedingungen, bei denen aufgebrachte Spannungen und Spannungsänderungen vom Bodenskelett so aufgenommen werden, dass keine Veränderung des Porendrucks verursacht wird

3.7
wirksame Fundamentfläche
verringerte Fundamentfläche, deren geometrischer Mittelpunkt auf dem Schnittpunkt des resultierenden Einwirkungsvektors mit der Gründungssohle liegt

3.8
Grenzzustand
Zustand, bei dessen Überschreitung das Bauwerk die relevanten Entwurfskriterien nicht mehr erfüllt

Anmerkung 1 zum Begriff: Siehe ISO 19900.

3.9
Materialfaktor
Teilsicherheitsbeiwert, der auf die Festigkeit des Bodens angewendet wird

3.10
repräsentativer Wert
Wert, der einer Grundvariablen für die Verifizierung eines Grenzzustands zugeordnet wird

Anmerkung 1 zum Begriff: Siehe ISO 19900.

3.11
Widerstand
Widerstand eines Bauteils, den Einflüssen einer Einwirkung ohne Ausfall standzuhalten

Anmerkung 1 zum Begriff: Siehe ISO 19900.

3.12
Widerstandsfaktor
Teilsicherheitsbeiwert, der auf den Widerstand einer Gründung angewendet wird

3.13
Auskolkung
durch Strömungen, Wellen und Eis verursachte Ablösung vom Meeresboden

3.14
Meeresgrund
Materialien unter Wasser, in denen eine Anlage gründet; dies können Böden wie Sand, Schluff oder Ton sowie auch zementierte Materialien oder Gestein sein

Anmerkung 1 zum Begriff: Der Meeresgrund kann als der Halbraum unter dem Meeresboden betrachtet werden.

Anmerkung 2 zum Begriff: Offshore-Gründungen werden meist in Böden eingebaut, was die Terminologie in diesem Teil der ISO 19901 widerspiegelt. Die Anforderungen gelten jedoch ebenso für zementierte Materialien und Gestein. Somit schließt der Begriff „Boden“ die anderen Materialen auf oder unter dem Meeresboden nicht aus.

Anmerkung 3 zum Begriff: Da noch keine allgemein anerkannten Definitionen der verschiedenen Arten von Boden und Gestein bestehen, siehe ISO 19901-8.

3.15
Meeresboden
Grenzbereich zwischen Meer und Meeresgrund

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3.16
Gebrauchstauglichkeit
Fähigkeit eines Bauwerkes oder Bauelementes, seine bestimmungsgemäße Funktion angemessen zu erfüllen

Anmerkung 1 zum Begriff: Siehe ISO 19902.

3.17
Absacken
dauerhafte Abwärtsbewegung einer Plattform aufgrund ihres eigenen Gewichts und aufgrund anderer Einwirkungen

3.18
Festigkeit
mechanische Werkstoffeigenschaft, die die Fähigkeit eines Werkstoffs, Einwirkungen standzuhalten, angibt und üblicherweise in Spannungseinheiten angegeben wird

Anmerkung 1 zum Begriff: Siehe ISO 19902.

3.19
undränierte Bedingungen
Bedingungen, bei denen aufgebrachte Spannungen und Spannungsänderungen sowohl vom Bodenskelett als auch von der Porenflüssigkeit so aufgenommen werden, dass keine Volumenänderung hervorgerufen wird

3.20
undränierte Scherfestigkeit
maximale Scherbeanspruchung beim Fließen oder bei einer festgelegten maximalen Dehnbeanspruchung in einem undränierten Zustand

Anmerkung 1 zum Begriff: Fließen ist der Zustand eines Werkstoffs, bei dem bei einer nur geringfügigen oder bei überhaupt keiner Spannungszunahme eine große plastische Dehnung auftritt.

4 Symbole und Abkürzungen

4.1 Allgemeines

Gebräuchliche Symbole sind nachstehend aufgeführt; andere Symbole sind im Text nach der jeweiligen Gleichung definiert. Es sollte beachtet werden, dass Symbole in verschiedenen Gleichungen unterschiedliche Bedeutung haben können.

4.2 Symbole für die Bemessung von Flachgründungen

A	tatsächliche Fundamentfläche (Querschnittsebene)
A'	wirksame Fundamentfläche der Gründung in Abhängigkeit von der Exzentrizität der Einwirkungen
A_h	eingebundene vertikale Querschnittsfläche der Gründung
A_p	Endfläche des Schürzenfußes
A_s	Fläche der Seitenwand einer in bestimmter Tiefe eingebundenen Fundamentschürze (einschließlich beider Seiten)
$A_{idealized}$	idealisierte rechteckige Fundamentfläche für unregelmäßige Gründungsformen

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b_c, b_q, b_γ	einzelne Korrekturfaktoren für die Sohlflächenneigung
B	kleinstes Seitenmaß des Fundaments (auch Fundamentbreite)
B'	kleinstes wirksames Seitenmaß des Fundaments (auch wirksame Fundamentbreite)
C	Kompressionsbeiwert des Bodens über den betrachteten Lastbereich
d_c, d_q, d_γ	einzelne Korrekturfaktoren für die Einbindetiefe des Fundaments
D	Fundamentdurchmesser (für kreisförmige Gründungen)
D_b	Tiefe unterhalb des Meeresbodens bis zur Gründungssohle
e	Exzentrizität einer Einwirkung
e_o	Anfangs-Porenziffer des Bodens
e_1	Exzentrizität einer Einwirkung entlang der Fundamentlänge
e_2	Exzentrizität einer Einwirkung entlang der Fundamentbreite
f	Mantelreibungswiderstand entlang der Fundamentschürzen während des Einbaus
F	Korrekturfaktor
g_c, g_q, g_γ	einzelne Korrekturfaktoren für die Neigung der Gründung gegenüber dem Meeresboden
G	elastischer Schermodul des Bodens
h	Bodenschichtdicke
H	bei der jeweiligen Lastbedingung auftretende horizontale Einwirkung
H_b	berücksichtigte horizontale Gesamteinwirkung auf die Sohlfläche
H_d	höchste horizontale Gesamteinwirkung auf die Gründungssohle bei Versagen unter undränierten Bedingungen
H_d'	höchste horizontale Gesamteinwirkung auf die Gründungssohle bei Versagen unter dränierten Bedingungen
H_{ult}	Bemessungswert der horizontalen Tragfähigkeit im Fließflächen-Bemessungsverfahren
ΔH	horizontaler Bodenwiderstand aufgrund von aktiv und passiv auf Fundamentschürzen einwirkenden Bodendrücken
i_c, i_q, i_γ	einzelne Korrekturfaktoren für die geneigte Belastung der Gründung
K_c, K_q, K_γ	Korrekturfaktoren zur Berücksichtigung der Lastneigung, der Gründungsform, der Einbindetiefe, der Sohlflächenneigung sowie der Neigung des Meeresbodens
K_p	Koeffizient des passiven Bodendrucks

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K_{rd}	Koeffizient der horizontalen Bodenreaktion unter dränierten Bedingungen
K_{ru}	Koeffizient der horizontalen Bodenreaktion unter undränierten Bedingungen
L	größtes Seitenmaß des Fundaments (auch Fundamentlänge)
L'	größtes wirksames Seitenmaß des Fundaments (auch wirksame Fundamentlänge)
M	Kippmoment
M_{ult}	Bemessungswert der Momenttragfähigkeit im Fließflächen-Bemessungsverfahren
N_c	dimensionslose Konstante, 5,14
N_q, N_γ	dimensionslose Funktionen von ϕ'
p'_0	vertikal an der Gründungssohle (am Schürzenfuß bei Einsatz von Schürzen) einwirkende Vertikalspannung
p'_{in}	vertikal am Schürzenfuß einwirkende Vertikalspannung innerhalb der Fundamentschürze, sofern vorhanden
p'_{out}	vertikal am Schürzenfuß einwirkende Vertikalspannung außerhalb der Fundamentschürze, sofern vorhanden
q_0	Anfangswert der wirksamen Vertikalspannung in einer bestimmten Bodenschicht
Δq	zusätzlich wirksame Vertikalspannung in einer bestimmten Bodenschicht
q	Spitzentragfähigkeit am Fuß der Fundamentschürzen bei deren Eindringen
q_c	Widerstand der CPT-Sondierspitze, in Spannungseinheiten, zur Berechnung des Schürzen-Eindringwiderstands
Q	bei der jeweiligen Lastbedingung auftretende vertikale Einwirkung
Q_d	höchste vertikale Gesamt-Einwirkung auf die Gründungssohle bei Versagen (*abzüglich* des Gewichts von Bodenpfropfen innerhalb von Schürzen) unter undränierten Bedingungen
Q_d'	höchste vertikale Gesamt-Einwirkung auf die Gründungssohle bei Versagen (*abzüglich* des Gewichts von Bodenpfropfen innerhalb von Schürzen) unter dränierten Bedingungen
$Q_{d,h}$	Bemessungswert des Gleitwiderstands
$Q_{d,v}$	Bemessungswert der Tragfähigkeit in Abwesenheit horizontaler Einwirkungen
$q_{d,v}$	Bemessungswert der Einheitstragfähigkeit in Abwesenheit horizontaler Einwirkungen
Q_f	Mantelreibungswiderstand entlang der Fundamentschürzen bei deren Eindringen
Q_p	Spitzentragfähigkeit am Fuß der Fundamentschürzen bei deren Eindringen
Q_r	Gesamt-Bodenwiderstand bei Eindringen von Schürzen

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Q_{ult} Bemessungswert der vertikalen Tragfähigkeit im Fließflächen-Bemessungsverfahren

Q_0 höchste vertikale Einwirkung je Längeneinheit einer Gründung mit unbegrenzter Länge

R Sohlflächenradius einer kreisförmigen Gründung

RP Bezugspunkt für die Umlagerung von Einwirkungen

s_u, c_u undränierte Scherfestigkeit des Bodens

s_{u0}, c_{u0} undränierte Scherfestigkeit des Bodens an der Gründungssohle (am Schürzenfuß bei Fundamenten mit Schürzen)

$s_{u\,ave}$ mittlere Bodenfestigkeit zwischen Meeresboden und Gründungssohle

s_{u2} äquivalente Scherfestigkeit unterhalb der Gründungssohle

s_c, s_q, s_γ einzelne Korrekturfaktoren für die Gründungsform

T Torsionsmoment

u_q, u_h vertikale und horizontale elastische Kurzzeit-Verschiebungen an der Gründungssohle

u_q vertikale Langzeit-Setzung einer Bodenschicht

V_b berücksichtigte vertikale Gesamteinwirkung auf die Sohlfläche

β Neigungswinkel des Bodens, in Radian, für die Berechnung von Neigungsfaktoren

δ Reibungswinkel der Kontaktfläche zwischen Boden und Gründung (bestimmend für das Gleiten bei glatter Kontaktfläche)

ϕ' wirksamer Reibungswinkel der Mohr'schen Hüllkurve für ebene Dehnungsbedingungen

γ' wirksames Gewicht des Bodens

γ_m Materialbeiwert

κ Anstiegsrate der undränierten Scherfestigkeit in Abhängigkeit von der Tiefe bei linearen Scherfestigkeitsprofilen

υ Poissonzahl des Bodens

ν Neigungswinkel der Gründungssohle, in Radian, für die Berechnung von Neigungsfaktoren

4.3 Symbole für die Bemessung von Pfahlgründungen

A_p Gesamt-Pfahlspitzenfläche, $A_p = \frac{\pi \cdot D^2}{4}$

A_r Pfahlverschiebungsverhältnis, $A_r = \frac{A_w}{A_p} = 1 - \left(\frac{D_i}{D}\right)^2$

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A_w	Fläche eines Stahlpfahlrandes, $A_w = \frac{\pi}{4} \cdot \left(D^2 - D_i^2\right)$
A_s	Fläche der Seitenwand eines Gründungspfahls im Boden
C_1, C_2, C_3	dimensionslose Koeffizienten, bestimmt als Funktion von ϕ' für p-y-Kurven für Sand
D	Außendurchmesser eines Gründungspfahls
D_i	Innendurchmesser eines Gründungspfahls, $D_i = D - 2\ WT$
D_{50}	mittlerer wirksamer Durchmesser der Bodenteilchen
D_{CPT}	Durchmesser des CPT-Werkzeugs, $D_{CPT} = 36$ mm bei einer Standard-Drucksonde mit einer Sondenfläche von 1 000 mm^2
D_r	relative Dichte von Sand für die CPT-Verfahren 1 und 4
E_S	Anfangsmodul der Baugrundreaktion
f	Mantelreibung
$f(z)$	Mantelreibung bei Tiefe z
$f_c(z)$	Mantelreibung unter Druckbelastung bei Tiefe z
$f_p(z)$	Mantelreibung zwischen Sand-Boden-Pfropfen und Pfahl-Innenwand für das CPT-Verfahren 4
$f_t(z)$	Mantelreibung unter Zugbelastung bei Tiefe z
f_{lim}	Grenzwert der Mantelreibung
h	Abstand über der Pfahlspitze $= L - z$
k	Anstiegsrate des Anfangsmoduls der Baugrundreaktion in Abhängigkeit von der Tiefe für p-y-Kurven für Sand
K_0	Koeffizient des seitlichen Bodendrucks im Beharrungszustand
L	Einbindelänge des Gründungspfahls unterhalb des ursprünglichen Meeresbodens
L_s	Länge des Bodenpfropfens in Sandschichten
N_q	dimensionsloser Tragfähigkeitsbeiwert
p	mobilisierte seitliche Tragfähigkeit für p-y-Kurven
p_a	Atmosphärendruck ($p_a = 100$ kPa)
$P_{d,e}$	Bemessungswert der axialen Einwirkung auf einen Gründungspfahl, bestimmt anhand eines gekoppelten linearen Bauwerks- und nichtlinearen Gründungsmodells unter Verwendung der Bemessungswerte für die Einwirkungen unter Extrembedingungen

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$P_{d,p}$	Bemessungswert der axialen Einwirkung auf einen Gründungspfahl, bestimmt anhand eines gekoppelten linearen Bauwerks- und nichtlinearen Gründungsmodells unter Verwendung der Bemessungswerte für ständige und veränderliche Einwirkungen oder für axiale Einwirkung unter Betriebsbedingungen
p_r	repräsentative seitliche Tragfähigkeit, für p-y-Kurven, in Krafteinheit je Längeneinheit des Gründungspfahls
p_{rd}	repräsentative tiefe seitliche Tragfähigkeit, für p-y-Kurven, in Krafteinheit je Längeneinheit des Gründungspfahls
p_{rs}	repräsentative flache seitliche Tragfähigkeit, für p-y-Kurven, in Krafteinheit je Längeneinheit des Gründungspfahls
$p'_0(z)$	wirksame Vertikalspannung bei Tiefe z
$p'_{0,tip}$	wirksame Vertikalspannung an der Pfahlspitze
$p'_m(z)$	wirksame mittlere Spannung bei Tiefe z
$p'_{0h}(z)$	wirksame Horizontalspannung bei Tiefe z
P_o	äußerer Umfang eines Gründungspfahls $= \pi D$
q	Spitzendruck an der Pfahlspitze
$q_c(z)$	CPT-Sondierspitzenwiderstand bei Tiefe z, in Spannungseinheiten
$q_{c,f}(z)$	verringerter CPT-Sondierspitzenwiderstand bei Tiefe z zur Berücksichtigung allgemeiner Auskolkung
$q_{c,av,1,5D}$	Mittelwert von $q_c(z)$ zwischen 1,5 D oberhalb und 1,5 D unterhalb der Pfahlspitze
$q_{c,tip}$	CPT-Sondierspitzenwiderstand an der Pfahlspitze
Q	mobilisierte Spitzentragfähigkeit in Q-z-Kurven
$Q_{f,c}$	Mantelreibungswiderstand unter Druckbelastung
$Q_{f,t}$	Mantelreibungswiderstand unter Zugbelastung
$Q_{f,i,clay}$	kumulativer Mantelreibungswiderstand von Tonschichten innerhalb des Bodenpfropfens für das CPT-Verfahren 3
Q_{lim}	Grenzwert des Spitzendrucks
Q_p	Spitzentragfähigkeit
Q_r	repräsentativer Wert der axialen Tragfähigkeit eines Gründungspfahls
$Q_{r,c}$	repräsentativer Wert der axialen Tragfähigkeit eines Gründungspfahls unter Druckbelastung
$Q_{r,t}$	repräsentativer Wert der axialen Tragfähigkeit eines Gründungspfahls unter Zugbelastung

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s_u	charakteristischer Wert der undränierten Scherfestigkeit des Bodens
$s_u(z)$	charakteristischer Wert der undränierten Scherfestigkeit des Bodens bei Tiefe z
WT	Wanddicke eines Gründungspfahls
t	mobilisierte Boden-Pfahl-Adhäsion für t-z-Kurven für axiale Scherübertragung
$t_{max}f(z)$	höchste Boden-Pfahl-Adhäsion oder Mantelreibung bei Tiefe z für t-z-Kurven für axiale Scherübertragung
t_{res}	verbleibende Boden-Pfahl-Adhäsion oder Mantelreibung für t-z-Kurven
y	seitliche Pfahlverschiebung für p-y-Kurven
z	Tiefe unterhalb des ursprünglichen Meeresbodens
z	lokale axiale Pfahlverschiebung für t-z-Kurven für axiale Scherübertragung
z	axiale Verschiebung der Pfahlspitze für Q-z-Kurven
z'	Endtiefe unterhalb des Meeresbodens nach allgemeiner Auskolkung
z_{peak}	axiale Pfahlverschiebung, bei der die höchste Boden-Pfahl-Adhäsion, t_{max}, erreicht wird, für t-z-Kurven
z_{res}	axiale Pfahlverschiebung, bei der die verbleibende Boden-Pfahl-Adhäsion, t_{res}, erreicht wird, für t-z-Kurven
α	dimensionsloser Mantelreibungsbeiwert für bindige Böden
β	für die Erstellung von p-y-Kurven für Sand verwendeter Parameter; $\beta = 45 + \frac{\phi'}{2}$
β	dimensionsloser Mantelreibungsbeiwert für nichtbindige Böden
δ_{cv}	konstanter Volumen-Reibungs-Winkel an der Kontaktfläche zwischen Sand und Pfahlwand
ε_c	Dehnung bei halbem Höchstwert der Deviatorspannung für p-y-Kurven für weichen Ton
ϕ'	innerer Reibungswinkel von Sand unter dränierten *triaxialen* Bedingungen
γ'	Wirk- oder Unterwasser-Gewicht des Bodens
γ_{pile}	Gewicht eines Gründungspfahls (Stahl, Beton usw.)
γ_{water}	Gewicht des Wassers
$\gamma_{R,Pe}$	Teilwiderstandsbeiwert für Extrembedingungen
$\gamma_{R,Pp}$	Teilwiderstandsbeiwert für ständige und veränderliche Einwirkungen unter Betriebsbedingungen

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ψ	Parameter für die Bestimmung des dimensionslosen Mantelreibungsbeiwertes für Tone $= s_u(z)/p'_0(z)$ bei Tiefe z
$\sigma'_{v0}(z)$	wirksame Vertikalspannung bei Tiefe z
Δz_{GS}	allgemeine Auskolkungstiefe
Δz_{LS}	lokale Auskolkungstiefe
e	Basis natürlicher Logarithmen ≈ 2,718
ln	natürlicher Logarithmus (Basis e)

4.4 Symbole für die Wechselwirkungen zwischen Boden und Bauwerk bei Unterwasser-Hilfsbauwerken, Risern und Förderleitungen

D	Förderleitung oder Pipeline, Durchmesser
f_c	dimensionsloser zyklischer Faktor
f_t	dimensionsloser Zeitfaktor
f_V	dimensionsloser Geschwindigkeitsfaktor
G_{max}	Anfangswert des elastischen Schermoduls des Bodens (geringe Dehnung)
H_{brk}	Ausbruchwiderstand für seitlichen Bodenwiderstand
K_{max}	Höchstwert der normierten Sekantensteifigkeit nach erster Entlastung oder erneuter Belastung
k_v	Sekantensteifigkeit einer äquivalenten Feder $= \Delta Q/\Delta z$
N	integrierte Normalkontaktkraft
N_c	dimensionsloser Tragfähigkeitsbeiwert
PI	Plastizitätszahl des Bodens
Q_{smax}	höchste Saugzug-(Auftriebs-)Kraft je Längeneinheit einer Pipeline
Q_u	Grenz-Eindringwiderstand je Längeneinheit einer Pipeline
s_u	undränierte Scherfestigkeit des Bodens
s_{uDSS}	im direkten Einfachscherversuch bestimmte undränierte Scherfestigkeit des Bodens
s_{ur}	undränierte Scherfestigkeit von gestörtem Boden
T	axialer Widerstand unter dränierten Bedingungen je Längeneinheit einer Pipeline
V	vertikale Einwirkung auf eine Pipeline
z	Tiefe bis zur Sohle von Förderleitung oder Pipeline

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ΔQ	Änderung der Vertikalkraft je Längeneinheit einer Pipeline
Δz	Änderung der Vertikalverschiebung
Δz_b	Verschiebung durch Auftrieb (Ausbruch)
δ	Reibungswinkel an der Boden-Pipeline-Kontaktfläche
μ	Boden-Pipeline-Reibungsbeiwert
ξ	dimensionsloser Verstärkungsfaktor
ξ_t	dimensionsloser Zeitfaktor
ξ_V	dimensionsloser Geschwindigkeitsfaktor
$\theta_{D'}$	Halbwinkel der Boden-Pipeline-Kontaktstrecke

4.5 Symbole für die Bemessung von Ankern für Verankerungssysteme

a	Beschleunigung eines Schwerkraftankers
A	Flunkenfläche eines Schleppankers
A_{eff}	wirksame Fläche eines Plattenankers zur Berücksichtigung von Form und Projektionsfläche
A_{in}	Draufsicht der Innenfläche eines Saugankerpfahls, der während des Einbringens mit Unterdruck beaufschlagt wird
A_{inside}	Innenfläche der Seitenwand eines Saugankerpfahls
A_p	Projektionsfläche eines/-r Schwerkraftankers/-verankerungsleine
A_{tip}	Querschnittsfläche einer Ankerpfahlspitze
A_{wall}	Summe der inneren und äußeren Wandflächen eines Ankerpfahls
B	Ankerflunkenbreite
C_D	Widerstandsbeiwert eines/-r Schwerkraftankers/-verankerungsleine
f	Reibungsbeiwert zwischen Meeresboden und Kette bzw. Drahtseil
F_b	Tragfähigkeit eines/-r eindringenden Schwerkraftankers/-verankerungsleine
F_{drag}	auf eine(n) Schwerkraftanker/-verankerungsleine einwirkende hydrodynamische Widerstandskraft
F_f	Reibungswiderstand eines/-r eindringenden Schwerkraftankers/-verankerungsleine
F_{max}	Bemessungswert der Haltefähigkeit (UHC) eines Plattenankers
FOS_{axial}	Sicherheitsbeiwert der axialen Belastung eines Ankers

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$FOS_{combined}$	Sicherheitsbeiwert der kombinierten Belastung eines Ankers
$FOS_{lateral}$	Sicherheitsbeiwert der seitlichen Belastung eines Ankers
H	horizontale Einwirkungskomponente
H	Haltefähigkeit eines Schleppankers unter horizontaler Einwirkung
L	Ankerflunkenlänge
L_{cw}	Länge der Kette bzw. des Drahtseils in Kontakt mit dem Meeresboden
m	Masse eines Schwerkraftankers
n	dimensionsloser Haltefähigkeitsbeiwert eines Schleppankers
N_c	dimensionsloser Tragfähigkeitsbeiwert
P_{cw}	Haltefähigkeit der Kette oder des Drahtseils einer Verankerungsleine
Q_{tot}	Gesamt-Eindringwiderstand eines Ankerpfahls
Q_{side}	Widerstand entlang der Seiten eines Ankerpfahls
Q_{tip}	Widerstand an der Ankerspitze
S_e	Verformungsgeschwindigkeitsbeiwert der Bodenfestigkeit
S_t	Empfindlichkeit des Bodens
s_u	undränierte Scherfestigkeit des Bodens am betrachteten Punkt
$s_{u,AVE}$	mittlere undränierte Scherfestigkeit innerhalb der Versagenszone in der Bemessungs-Eindringtiefe, korrigiert hinsichtlich der Einwirkungen zyklischer Beanspruchung
$s_{u,tip\ AVE}$	Mittelwert der triaxialen Zusammendrückung, der triaxialen Dehnung und der in direkten Einfachscherversuchen bestimmten undränierten Scherfestigkeit in der Eindringtiefe der Ankerspitze
$s_{u,DSS}$	in direkten Einfachscherversuchen bestimmte undränierte Scherfestigkeit des Bodens
t	Zeit
t_r	Zeit für das Aufholen eines Ankers
v	Geschwindigkeit des freien Falls eines Schwerkraftankers
V	vertikale Einwirkungskomponente
W_s	Unterwasser-Gewicht eines Schwerkraftankers
W'	Unterwasser-Gewicht eines Ankers
W'_{cw}	Unterwasser-Gewicht der Kette bzw. des Drahtseils

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z	Einbinde- oder Eindringtiefe
ΔU_{req}	für die Einbindung eines Saugankerpfahls erforderlicher Unterdruck
ΔU_{crit}	zum Bruch des Bodenpfropfens innerhalb eines Saugankerpfahls führender kritischer Unterdruck
α_{ins}	Adhäsionsfaktor während des Einbringens eines Ankers
δ	Wandadhäsionsfaktor entlang des Schaftes und der Flunken eines eindringenden Schwerkraftankers
γ'	wirksames Gewicht des Bodens
η	empirischer Reduzierfaktor zur Berücksichtigung des fortschreitenden Versagens eines Plattenankers
ρ	Fluiddichte
θ	Winkel der Verankerungsleine am Laschpunkt des Ankerauges (zur Horizontalen)
θ_{axial}	Winkel der Verankerungsleine am Laschpunkt des Ankerauges (zur Horizontalen), über dem der Bemessungswert der Tragfähigkeit des Ankers durch die axiale Tragfähigkeit bestimmt wird
$\theta_{lateral}$	Winkel der Verankerungsleine am Laschpunkt des Ankerauges (zur Horizontalen), unter dem der Bemessungswert der Tragfähigkeit des Ankers durch die seitliche Tragfähigkeit bestimmt wird

4.6 Abkürzungen

BOP	Blowout-Preventer (en: blow-out preventer)
CPT	Drucksondierung (en: cone penetration test)
FEM	Finite-Elemente-Verfahren (en: finite element method)
FOS	Sicherheitsbeiwert (en: factor of safety)
PFD	Nachweis mit Teilsicherheitsbeiwert (en: partial factor design)
REB	umgekehrte Spitzentragfähigkeit (en: reverse end bearing)
SCR	Stahlsteigrohrleitung (en: steel catenary riser)
SLS	Grenzzustand der Gebrauchstauglichkeit (en: serviceability limit state)
SRD	Bodenwiderstand bei der Rammung (en: soil resistance to driving)
TDP	Aufsetzpunkt (en: touch-down point)
UHC	Bemessungswert der Tragfähigkeit (en: ultimate holding capacity)
ULS	Grenzzustand der Tragfähigkeit (en: ultimate limit state)
VLA	Anker unter Vertikallast (en: vertically loaded anchor)

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Literaturhinweise

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DIN EN ISO 19901-2, *Erdöl- und Erdgasindustrie — Spezielle Anforderungen für Offshore-Anlagen — Teil 2: Seismische Auslegungsverfahren und -kriterien*

DIN EN ISO 19901-7, *Erdöl- und Erdgasindustrie — Spezielle Anforderungen an Offshore-Anlagen — Teil 7: Betriebssysteme für das Positionshalten schwimmender Anlagen und mobiler Offshore-Einheiten*

DIN EN ISO 19902, *Erdöl- und Erdgasindustrie — Gegründete Stahlplattformen*

DIN EN ISO 19903, *Erdöl- und Erdgasindustrie — Feststehende Offshore-Betonkonstruktionen*

DIN EN ISO 19905-1, *Erdöl- und Erdgasindustrie — Beurteilung von mobilen Offshore Einheiten bezüglich ihres Einsatzgebietes — Teil 1: Hubinseln*

DIN CEN ISO/TR 19905-2 DIN SPEC 67998, *Erdöl- und Erdgasindustrie — Offshore Anlagen; Beurteilung von mobilen Offshore Einheiten bezüglich ihres Einsatzgebietes — Teil 2: Hubinseln, Erläuterungen*

DIN EN ISO 19906, *Erdöl- und Erdgasindustrie — Offshore-Bauwerke für den Arktis-Bereich*

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Petroleum and natural gas industries — Specific requirements for offshore structures — Part 4: Geotechnical and foundation design considerations

Industries du pétrole et du gaz naturel — Exigences spécifiques relatives aux structures en mer — Partie 4: Bases conceptuelles des fondations

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ISO 19901 consists of the following parts, under the general title *Petroleum and natural gas industries — Specific requirements for offshore structures*:

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— *Part 2: Seismic design procedures and criteria*

— *Part 3: Topsides structure*

— *Part 4: Geotechnical and foundation design considerations*

— *Part 5: Weight control during engineering and construction*

— *Part 6: Marine operations*

— *Part 7: Stationkeeping systems for floating offshore structures and mobile offshore units*

— *Part 8: Marine soil investigations.*

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— ISO 19900, *Petroleum and natural gas industries — General requirements for offshore structures*

— ISO 19901 (all parts), *Petroleum and natural gas industries — Specific requirements for offshore structures*

— ISO 19902, *Petroleum and natural gas industries — Fixed steel offshore structures*

— ISO 19903, *Petroleum and natural gas industries — Fixed concrete offshore structures*

— ISO 19904, *Petroleum and natural gas industries — Floating offshore structures*

— ISO 19905-1, *Petroleum and natural gas industries — Site-specific assessment of mobile offshore units — Part 1: Jack-ups*

— ISO/TR 19905-2, *Petroleum and natural gas industries — Site-specific assessment of mobile offshore units — Part 2: Jack-ups commentary and detailed sample calculation*

— ISO 19906, *Petroleum and natural gas industries — Arctic offshore structures*

Other ISO standards can have implications for the geotechnical design of foundations for offshore structures, in particular:

— ISO 13623 (all parts), *Petroleum and natural gas industries — Pipeline transportation systems*

— ISO 13628 (all parts), *Petroleum and natural gas industries — Design and operation of subsea production systems*

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Introduction

The International Standards for offshore structures, ISO 19900 through to ISO 19906, constitute a common basis covering those aspects that address design requirements and assessments of all offshore structures used by the petroleum and natural gas industries worldwide. Through their application the intention is to achieve reliability levels appropriate for manned and unmanned offshore structures, whatever the type of structure and the nature of the materials used.

It is important to recognize that structural integrity is an overall concept comprising models for describing actions, structural analyses, design rules, safety elements, workmanship, quality control procedures and national requirements, all of which are mutually dependent. The modification of one aspect of design in isolation can disturb the balance of reliability inherent in the overall concept or structural system. The implications involved in modifications, therefore, need to be considered in relation to the overall reliability of all offshore structural systems.

For foundations, some additional considerations apply. These include the time, frequency and rate at which actions are applied, the method of foundation installation, the properties of the surrounding soil, the overall behaviour of the seabed, effects from adjacent structures and the results of drilling into the seabed. All of these, and any other relevant information, need to be considered in relation to the overall reliability of the foundation.

These International Standards are intended to provide wide latitude in the choice of structural configurations, materials and techniques without hindering innovation. The design practice for the foundations of offshore structures has proved to be an innovative and evolving process over the years. This evolution is expected to continue and is encouraged. Therefore, circumstances can arise when the procedures described herein or in ISO 19900 to ISO 19906 (or elsewhere) are insufficient on their own to ensure that a safe and economical foundation design is achieved.

Seabed soils vary. Experience gained at one location is not necessarily applicable at another, and extra caution is necessary when dealing with unfamiliar soils or foundation concepts. Sound engineering judgment is therefore necessary in the use of this part of ISO 19901.

For an offshore structure and its foundations, the action effects at the interface between the structure's subsystem and the foundation's subsystem(s) are internal forces, moments and deformations. When addressing the foundation's subsystem(s) in isolation, these internal forces, moments and deformations can be considered as actions on the foundation's subsystem(s) and this approach is followed in this part of ISO 19901.

Some background to and guidance on the use of this part of ISO 19901 is provided for information in informative Annex A. Guidance on foundations in carbonate soils is provided for information in A.6.5, but there is, as yet, insufficient knowledge and understanding of such soils to produce normative requirements.

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Petroleum and natural gas industries — Specific requirements for offshore structures — Part 4: Geotechnical and foundation design considerations

1 Scope

This part of ISO 19901 contains requirements and recommendations for those aspects of geoscience and foundation engineering that are applicable to a broad range of offshore structures, rather than to a particular structure type. Such aspects are:

— site and soil characterization,

— identification of hazards,

— design and installation of shallow foundations supported by the seabed,

— design and installation of pile foundations

— soil–structure interaction for auxiliary structures, e.g. subsea production systems, risers and flowlines (guidance given in A.10);

— design of anchors for the stationkeeping systems of floating structures (guidance given in A.11).

Particular requirements for marine soil investigations are detailed in ISO 19901-8.

Aspects of soil mechanics and foundation engineering that apply equally to offshore and onshore structures are not addressed. The user of this part of ISO 19901 is expected to be familiar with such aspects.

ISO 19901-4 outlines methods developed primarily for the design of shallow foundations with an embedded length (L) to diameter (D) ratio L/D<1 (Clause 7) or relatively long and flexible pile foundations with L/D>10 (Clause 8). This standard does not apply to intermediate foundations with 1<L/D<10. Such intermediate foundations, often known as 'caisson foundations', comprise either shallow foundations with skirts penetrating deeper into the seabed than the width of the foundation, or shorter, more rigid and larger diameter piles than those traditionally used for founding offshore structures. The design of such foundations can require specific analysis methods, and any extrapolation from the design methods described in this standard to intermediate foundations shall be treated with care and assessed by a geotechnical specialist on a case by case situation.

2 Normative references

The following documents, in whole or in part, are normatively referenced in this document and are indispensable for its application. For dated references, only the edition cited applies. For undated references, the latest edition of the referenced document (including any amendments) applies.

ISO 19900, *Petroleum and natural gas industries — General requirements for offshore structures*

ISO 19901-2, *Petroleum and natural gas industries — Specific requirements for offshore structures — Part 2: Seismic design procedures and criteria*

ISO 19901-7, *Petroleum and natural gas industries — Specific requirements for offshore structures — Part 7: Stationkeeping systems for floating offshore structures and mobile offshore units*

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ISO 19901-8, *Petroleum and natural gas industries — Specific requirements for offshore structures — Part 8: Marine soil investigations*

ISO 19902, *Petroleum and natural gas industries — Fixed steel offshore structures*

ISO 19903, *Petroleum and natural gas industries — Fixed concrete offshore structures*

ISO 19905-1, *Petroleum and natural gas industries — Site-specific assessment of mobile offshore units — Part 1: Jack-ups*

ISO/TR 19905-2, *Petroleum and natural gas industries — Site-specific assessment of mobile offshore units — Part 2: Jack-ups commentary and detailed sample calculation*

ISO 19906, *Petroleum and natural gas industries — Arctic offshore structures*

3 Terms and definitions

For the purposes of this document, the terms and definitions given in ISO 19900, the ISO 19901 series and the following apply.

3.1
action
external loading applied to the structure (direct action) or an imposed deformation or acceleration (indirect action)

Note 1 to entry: An imposed deformation can be caused by fabrication tolerances, differential settlement, temperature change or moisture variation. An earthquake typically generates imposed accelerations.

Note 2 to entry: See ISO 19900.

3.2
basic variable
one of a specified set of variables representing physical quantities which characterize material properties including soil properties

Note 1 to entry: See ISO 19900.

3.3
characteristic value
value assigned to a basic variable associated with a prescribed probability of not being violated by unfavourable values during some reference period

Note 1 to entry: The characteristic value is the main representative value.

Note 2 to entry: See ISO 19900.

3.4
design actions
combination of representative actions and partial safety factors representing a design situation for use in checking the acceptability of a design

3.5
design value
value derived from the representative value for use in the design verification procedure

Note 1 to entry: In some design situations, two design values can be defined, an upper and a lower value.

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Note 2 to entry: See ISO 19900.3.6

drained condition
condition whereby the applied stresses and stress changes are supported by the soil skeleton and do not cause a change in pore pressure

3.7
effective foundation area
reduced foundation area having its geometric centre at the point where the resultant action vector intersects the foundation base level

3.8
limit state
state beyond which the structure no longer satisfies the relevant design criteria

Note 1 to entry: See ISO 19900.

3.9
material factor
partial safety factor applied to the strength of the soil

3.10
representative value
value assigned to a basic variable for verification of a limit state

Note 1 to entry: See ISO 19900.

3.11
resistance
resistance of a component to withstand action effects without failure

Note 1 to entry: See ISO 19900.

3.12
resistance factor
partial safety factor applied to the resistance of a foundation

3.13
scour
removal of seabed soils caused by currents, waves or ice

3.14
seabed
materials below the sea in which a structure is founded, whether of soils such as sand, silt or clay, cemented materials or of rock

Note 1 to entry: The seabed can be considered as the half-space below the seafloor.

Note 2 to entry: Offshore foundations are most commonly installed in soils, and the terminology in this part of ISO 19901 reflects this. However, the requirements equally apply to cemented seabed materials and rocks. Thus, the term 'soil' does not exclude any other material at or below the seafloor.

Note 3 to entry: As yet there are no universally accepted definitions of the various types of soil and rock, see ISO 19901-8.

3.15
seafloor
interface between the sea and the seabed

3.16
serviceability

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ability of a structure or structural component to perform adequately for normal functional use

Note 1 to entry: See ISO 19902.

3.17
settlement
permanent downward movement of a structure as a result of its own weight and other actions

3.18
strength
mechanical property of a material indicating its ability to resist actions, usually given in units of stress

Note 1 to entry: See ISO 19902.

3.19
undrained condition
condition whereby the applied stresses and stress changes are supported by both the soil skeleton and the pore fluid and do not cause a change in volume

3.20
undrained shear strength
maximum shear stress at yielding or at a specified maximum strain in an undrained condition

Note 1 to entry: Yielding is the condition of a material in which a large plastic strain occurs at little or no stress increase.

4 Symbols and abbreviated terms

4.1 General

Commonly used symbols are listed below; other symbols are defined in the text following the applicable formula. It should be noted that symbols can have different meanings between formulae.

4.2 Symbols for shallow foundations design

A	actual (cross-sectional plan) foundation area
A'	effective foundation area of foundation depending on eccentricity of actions
A_h	embedded vertical cross-sectional area of foundation
A_p	end area of skirt tip
A_s	side surface area of foundation skirt embedded at a particular depth (including both sides)
$A_{idealized}$	idealized rectangular foundation area, for irregular foundation shapes
b_c, b_q, b_γ	individual correction factors related to foundation base inclination
B	minimum lateral foundation dimension (also foundation width)
B'	minimum effective lateral foundation dimension (also foundation effective width)
C	compression index of soil over loading range considered
d_c, d_q, d_γ	individual correction factors related to foundation embedment depth
D	foundation diameter (for circular foundations)

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D_b	depth below seafloor to foundation base level
e	eccentricity of action
e_o	initial void ratio of the soil
e_1	eccentricity of action along foundation length
e_2	eccentricity of action along foundation width
f	unit skin friction resistance along foundation skirts, during installation
F	correction factor
g_c, g_q, g_γ	individual correction factors related to foundation seafloor inclination
G	elastic shear modulus of soil
h	soil layer thickness
H	acting horizontal action for relevant loading condition
H_b	factored horizontal total action on base area
H_d	maximum total horizontal action applied to foundation base at failure under undrained conditions
H_d'	maximum total horizontal action applied to foundation base at failure under drained conditions
H_{ult}	ultimate horizontal capacity in yield surface design method
ΔH	horizontal soil resistance due to active and passive earth pressures on foundation skirts
i_c, i_q, i_γ	individual correction factors related to foundation loading inclination
K_c , K_q , K_γ	correction factors which account for loading inclination, foundation shape, depth of embedment, inclination of base, and inclination of the seafloor
K_p	coefficient of passive earth pressure
K_{rd}	drained horizontal soil reaction coefficient
K_{ru}	undrained horizontal soil reaction coefficient
L	maximum lateral foundation dimension (also foundation length)
L'	maximum effective lateral foundation dimension (also foundation effective length)
M	overturning moment
M_{ult}	ultimate moment capacity in yield surface design method
N_c	a dimensionless constant, 5,14
N_q , N_γ	dimensionless functions of ϕ'
p'_0	vertical effective vertical stress at foundation base level (skirt tip level when skirts are used)
p'_{in}	vertical effective vertical stress at skirt tip level, inside the skirts of a skirted foundation
p'_{out}	vertical effective vertical stress at skirt tip level, outside the skirts of a skirted foundation

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q_0	initial effective vertical stress at level of a given soil layer
Δq	added effective vertical stress at level of a given soil layer
q	unit end bearing resistance on foundation skirt tip, during penetration
q_c	CPT cone-tip resistance, in stress units, for calculation of skirt penetration resistance
Q	acting vertical action, for relevant loading condition
Q_d	maximum total vertical action applied to the base of the foundation at failure (*excluding* weight of soil plug inside skirts) under undrained conditions
Q_d'	maximum total vertical action applied to the base of the foundation at failure (*excluding* weight of soil plug inside skirts) under drained conditions
$Q_{d,h}$	design sliding resistance
$Q_{d,v}$	design bearing capacity in the absence of horizontal actions
$q_{d,v}$	design unit bearing capacity in the absence of horizontal actions
Q_f	skin friction resistance along foundation skirts during skirt penetration
Q_p	end bearing resistance at foundation skirt tip during skirt penetration
Q_r	total soil resistance during skirt penetration
Q_{ult}	ultimate vertical capacity in yield surface design method
Q_0	maximum vertical action, per unit length, of an infinitely long foundation
R	radius of the base of a circular foundation
RP	reference point for action transfer
s_u, c_u	undrained shear strength of soil
s_{u0}, c_{u0}	undrained shear strength of soil at foundation base level (skirt tip level for skirted foundations)
$s_{u\ ave}$	average soil strength from seafloor to foundation base level
s_{u2}	equivalent shear strength below foundation base
s_c, s_q, s_γ	individual correction factors related to foundation shape
T	torsional moment
u_q, u_h	vertical and horizontal short term elastic displacements at foundation base
u_q	vertical long term settlement of soil layer
V_b	factored vertical total action on base area
β	ground inclination angle in radians, in calculation of inclination factors
δ	interface friction angle between soil and foundation (governing sliding for smooth interface)

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ϕ' effective friction angle of Mohr envelope, for plane strain conditions

γ' effective unit weight of soil

γ_m material factor

κ rate of increase of undrained shear strength with depth, for linear shear strength profiles

υ Poisson's ratio of the soil

ν foundation base inclination angle in radians, in calculation of inclination factors

4.3 Symbols for pile foundations design

A_p gross end area of pile, $A_p = \frac{\pi \cdot D^2}{4}$

A_r pile displacement ratio, $A_r = \frac{A_w}{A_p} = 1 - \left(\frac{D_i}{D}\right)^2$

A_w area of steel pile rim, $A_w = \frac{\pi}{4} \cdot \left(D^2 - D_i^2\right)$

A_s side surface area of pile in soil

C_1, C_2, C_3 dimensionless coefficients determined as function of ϕ', for p–y curves for sand

D pile outside diameter

D_i pile inside diameter, $D_i = D - 2\ WT$

D_{50} mean effective soil particle diameter

D_{CPT} diameter of CPT tool, D_{CPT} = 36 mm for a standard cone penetrometer with a cone area of 1 000 mm^2

D_r relative density of sand, for CPT-based methods 1 and 4

E_S initial modulus of subgrade reaction

f unit skin friction

$f(z)$ unit skin friction at depth z

$f_c(z)$ unit skin friction in compression at depth z

$f_p(z)$ unit skin friction between sand soil plug and inner pile wall, for CPT-based method 4

$f_t(z)$ unit skin friction in tension at depth z

f_{lim} limiting unit skin friction value

h distance above pile tip = $L - z$

k rate of increase with depth of initial modulus of subgrade reaction, for p–y curves for sand

K_0 coefficient of lateral earth pressure at rest

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L	embedded length of pile below original seafloor
L_s	length of soil plug in sand layers
N_q	dimensionless bearing capacity factor
p	mobilized lateral capacity, for p–y curves
p_a	atmospheric pressure (p_a = 100 kPa)
$P_{d,e}$	design axial action on the pile, determined from a coupled linear structure and non-linear foundation model using the design actions for extreme conditions
$P_{d,p}$	design axial action on the pile, determined from a coupled linear structure and non-linear foundation model using the design actions for permanent and variable actions or the design axial action for operating situations
p_r	representative lateral capacity, for p–y curves, in unit of force per unit length of pile
p_{rd}	representative deep lateral capacity, for p–y curves, in unit of force per unit length of pile
p_{rs}	representative shallow lateral capacity, for p–y curves, in unit of force per unit length of pile
$p'_0(z)$	effective vertical stress at depth z
$p'_{0,tip}$	effective vertical stress at pile tip
$p'_m(z)$	effective mean stress at depth z
$p'_{0h}(z)$	effective horizontal stress at depth z
P_o	pile outer perimeter = πD
q	unit end bearing at pile tip
$q_c(z)$	CPT cone-tip resistance at depth, z, in stress units
$q_{c,f}(z)$	reduced CPT cone-tip resistance at depth, z, to account for general scour
$q_{c,av,1,5D}$	average value of q_c (z) between 1,5 D above pile tip and 1,5 D below pile tip
$q_{c,tip}$	CPT cone-tip resistance at pile tip
Q	mobilized end bearing capacity in Q–z curves
$Q_{f,c}$	skin friction capacity in compression
$Q_{f,t}$	skin friction capacity in tension
$Q_{f,i,clay}$	cumulative skin friction capacity of clay layers within soil plug, for CPT-based method 3
Q_{lim}	limiting unit end bearing value
Q_p	end bearing capacity
Q_r	representative value of axial pile capacity
$Q_{r,c}$	representative value of axial pile capacity in compression

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$Q_{r,t}$	representative value of axial pile capacity in tension
s_u	characteristic value of undrained shear strength of soil
$s_u(z)$	characteristic value of undrained shear strength of soil at depth z
WT	pile wall thickness
t	mobilized soil–pile adhesion, for axial shear transfer t–z curves
t_{max} $f(z)$	maximum soil–pile adhesion or unit skin friction at depth z, for axial shear transfer t–z curves
t_{res}	residual soil–pile adhesion or unit skin friction, for t–z curves
y	lateral pile displacement, for p–y curves
z	depth below original seafloor
z	local pile axial displacement, for axial shear transfer t–z curves
z	axial pile tip displacement, for Q–z curves
z'	final depth below seafloor, after general scour
z_{peak}	axial pile displacement at which maximum soil–pile adhesion, t_{max}, is reached, for t–z curves
z_{res}	axial pile displacement at which residual soil–pile adhesion, t_{res}, is reached, for t–z curves
α	dimensionless skin friction factor, for cohesive soils
β	parameter used for the development of p–y curves for sand; $\beta = 45 + \frac{\phi'}{2}$
β	dimensionless skin friction factor, for cohesionless soils
δ_{cv}	constant volume friction angle at sand–pile wall interface
ε_c	strain at one-half maximum deviator stress, for p–y curves for soft clay
ϕ'	angle of internal friction of sand, for drained *triaxial* conditions
γ'	submerged, or effective, soil unit weight
γ_{pile}	unit weight of pile (steel, concrete, etc.)
γ_{water}	unit weight of water
$\gamma_{R,Pe}$	partial resistance factor for extreme conditions
$\gamma_{R,Pp}$	partial resistance factor for permanent and variable actions for operating situations
ψ	parameter to determine the dimensionless skin friction factor, for clays = $s_u(z)$ / $p'_0(z)$ at depth z
$\sigma'_{v0}(z)$	effective vertical stress at depth z
Δz_{GS}	general scour depth
Δz_{LS}	local scour depth

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e	base natural logarithms ≈ 2,718
ln	natural logarithm (base e)

4.4 Symbols for soil–structure interaction for auxiliary subsea structures, risers and flowlines

D	flowline, or pipeline, diameter
f_c	dimensionless cyclic factor
f_t	dimensionless time factor
f_V	dimensionless velocity factor
G_{max}	initial elastic (small strain) shear modulus of soil
H_{brk}	breakout resistance for lateral soil resistance
K_{max}	maximum value of normalized secant stiffness on initial unloading or reloading
k_v	secant stiffness of equivalent spring = $\Delta Q / \Delta z$
N	integrated normal contact force
N_c	dimensionless bearing factor
PI	plasticity index of soil
Q_{smax}	maximum suction (uplift) force, per unit length of pipeline
Q_u	limiting penetration resistance, per unit length of pipeline
s_u	undrained shear strength of soil
s_{uDSS}	undrained shear strength of soil obtained in direct simple shear mode
s_{ur}	remoulded undrained shear strength of soil
T	drained axial resistance per unit length of pipeline
V	vertical action on pipeline
z	depth to flowline, or pipeline, invert
ΔQ	change in vertical force, per unit length of pipeline
Δz	change in vertical displacement
Δz_b	uplift (break-out) displacement
δ	interface friction angle at soil–pipeline interface
μ	pipeline–soil friction coefficient
ξ	dimensionless enhancement factor
ξ_t	dimensionless time factor

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ξ_V dimensionless velocity factor

$\theta_{D'}$ half-angle of pipeline–soil contact perimeter

4.5 Symbols for design of anchors for mooring systems

a acceleration of a gravity embedded anchor

A fluke area of a drag anchor

A_{eff} effective area of a plate anchor accounting for shape and projected area

A_{in} plan view inside area of suction anchor pile where underpressure is applied during installation

A_{inside} inside lateral area of suction anchor pile wall

A_p projected area of a gravity embedded anchor/line

A_{tip} tip cross-sectional area of an anchor pile

A_{wall} sum of inside and outside wall areas of an anchor pile

B fluke width

C_D drag coefficient of a gravity-embedded anchor/line

f coefficient of friction between chain or wire rope and the seafloor

F_b bearing resistance of a penetrating gravity-embedded anchor/line

F_{drag} hydrodynamic drag force action on a gravity-embedded anchor/line

F_f frictional resistance of a penetrating gravity-embedded anchor/line

F_{max} ultimate holding capacity (UHC) of a plate anchor

FOS_{axial} factor of safety with respect to axial loading of anchor

$FOS_{combined}$ factor of safety with respect to combined loading of anchor

$FOS_{lateral}$ factor of safety with respect to lateral loading of anchor

H horizontal action component

H holding capacity of drag anchor under horizontal action

L anchor fluke length

L_{cw} length of chain or wire rope in contact with the seafloor

m mass of a gravity-embedded anchor

n dimensionless holding capacity factor for a drag anchor

N_c dimensionless bearing capacity factor

P_{cw} holding capacity of mooring line chain or wire rope

Q_{tot} total penetration resistance of an anchor pile

Q_{side} resistance along the sides of an anchor pile

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Q_{tip}	resistance at the tip of an anchor tip
S_e	soil strength strain rate factor
S_t	soil sensitivity
s_u	undrained shear strength of the soil at the point in question
$s_{u,AVE}$	average undrained shear strength within the failure zone at the design penetration depth, corrected for effects of cyclic loading
$s_{u,tip\ AVE}$	average of triaxial compression, triaxial extension, and DSS undrained shear strength at anchor tip penetration depth
$s_{u,DSS}$	undrained shear strength of the soil obtained from direct simple shear tests
t	time
t_r	time of anchor retrieval
v	free-fall velocity of a gravity-embedded anchor
V	vertical action component
W_s	submerged weight of a gravity-embedded anchor
W'	submerged weight of anchor
W'_{cw}	submerged unit weight of chain or wire rope
z	embedment or penetration depth
ΔU_{req}	required underpressure to embed a suction anchor pile
ΔU_{crit}	critical underpressure causing failure of soil plug inside a suction anchor pile
α_{ins}	adhesion factor during anchor installation
δ	wall adhesion factor along the shaft and fins of a penetrating gravity embedded anchor
γ'	effective unit weight of soil
η	empirical reduction factor accounting for progressive failure of a plate anchor
ρ	fluid density
θ	angle of mooring line at anchor padeye attachment point (measured from horizontal)
θ_{axial}	angle of mooring line at anchor padeye attachment point (measured from horizontal) above which the anchor ultimate capacity is controlled by axial capacity
$\theta_{lateral}$	angle of mooring line at anchor padeye attachment point (measured from horizontal) below which the anchor ultimate capacity is controlled by lateral capacity

4.6 Abbreviated terms

BOP	blow-out preventer

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CPT	cone penetration test
FEM	finite element method
FOS	factor of safety
PFD	partial factor design
REB	reverse end bearing
SCR	steel catenary riser
SLS	serviceability limit state
SRD	soil resistance to driving
TDP	touch-down point
UHC	ultimate holding capacity
ULS	ultimate limit state
VLA	vertically loaded anchor

5 General requirements

5.1 General

The design methodology inherent to the ISO 19900 set of International Standards is based on the partial factor design (PFD) approach with specified action and material factors (see Clause 9 of ISO 19900). Requirements regarding partial factors for actions and the combination of actions into design situations are given in the relevant International Standards for offshore structures, i.e. ISO 19900 to ISO 19906. However, there are some aspects of design for which the PFD design formulations have not been developed and for these, other approaches are used (e.g. use of global safety factors in ISO 19901-7 for the design of stationkeeping systems).

The material factor for soil can be expressed as the ratio of the undrained shear strength to the shear stress mobilized for equilibrium, or as the ratio of the tangent of the representative friction angle to the tangent of the friction angle mobilized for equilibrium. The material factor should not be lower than 1,25. It can be modified with regard to the combination of consequences of failure, the failure mechanism, the accuracy of the applied calculation method and the way in which the characteristic strength of the soil material was determined and expressed.

Consideration should also be given to what is recognized practice for the calculation procedures applied and the stability mechanisms analysed. It is advised that the material factor should be reviewed and possibly increased particularly when new types of structures are considered and/or soil conditions with no or little experience are encountered (to be agreed between the designer and the end user for every type of foundation).

The PFD design methodology in this part of ISO 19901 involves the use of action and material factors which aim to result in comparable overall foundation reliability with that achieved from use of the working stress design (WSD) methodology in the American Petroleum Institute's Recommended Practice API RP2GEO [1]. For shallow foundations, this is outlined in [2], which highlights that broadly comparable foundation size should be obtained to resist a given set of unfactored action from either method. However, as the design recommendations are largely aligned in both this document and [1], one should be aware that the use of either PFD or WSD methods can lead to differences in the design outcome. In critical situations, it is recommended to seek guidance from a geotechnical specialist.

The foundation shall be designed to carry static and dynamic (repetitive as well as transient) actions without causing excessive deformation of or vibrations in the structure. Special attention shall be given to the effects of repetitive and transient actions on the structural response, as well as on the strength of the supporting soils.

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The possibility of movement of the seabed shall be considered. Any actions resulting from such movements on foundation members shall be considered in the design. The potential for disturbance to foundation soils by conductor installation or shallow well drilling shall be assessed (see 9.12). Furthermore, the guidance herein does not necessarily apply to unconventional soils such as carbonate material (see A.6.5), volcanic sands or highly sensitive clays.

5.2 Design situations and safety factors

5.2.1 Design cases for fixed steel structures

The design situations that shall be considered for fixed steel structures, with the corresponding partial action factors are given in Table 8.2-1 of ISO 19902. For easier reference for the users of this standard, the partial action factors from ISO 19902 are repeated in A.5.2.1, but designers shall refer to the normative standard ISO 19902.

The resistance factors applicable to the design of pile foundations are given in 8.1.1. The material factors applicable to the design of shallow foundations are given in 7.3.1 and 7.3.3. In assessing the stability of shallow foundations, the resistance is computed by applying a material factor to the soil strength. This differs from the practice for design of piles, where a resistance factor is applied to the foundation capacity.

The specific requirements and design procedures and criteria under dynamic actions from earthquakes are given in ISO 19901-2.

5.2.2 Design cases for anchors for stationkeeping systems for floating structures

The global safety factors applicable for the ultimate limit state (ULS) for the design of anchors for floating structures are given in ISO 19901-7, in which see:

— Table 6 of ISO 19901-7 for the design of drag anchors;

— Table 7 of ISO 19901-7 for the design of anchor piles, comprising driven piles, suction piles and gravity-embedded anchors (i.e. free-fall ‘torpedoes’);

— Table 8 of ISO 19901-7 for the design of gravity anchors and plate anchors.

For easier reference for the users of this standard, the safety factors from ISO 19901-7 are repeated in A.5.2.2, but designers shall refer to the normative standard ISO 19901-7.

5.3 Characteristic values of soil properties

This section provides generic principles and guidelines for selecting characteristic values of soil properties, in line with the partial factors format or partial factor design (PFD) approach. The term 'soil' is used as per ISO 2394 and ISO 19900. This term is equivalent to 'seabed' defined in 3.14. Soil includes pore fluid, pore gas and made ground.

Soil stratification and estimation of characteristic values for soil properties should consider:

— the assumptions made in the calculation model;

— the spatial variability of soil within each stratum;

— the amount and quality of site investigations and possible environmental influences, including insufficient data and imprecise knowledge;

— *a priori* knowledge such as geological information and physically credible values;

— measurable physical quantities that correspond to, and are representative of, the population of the properties considered in the calculation model;

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— appropriate factors or functions, to convert the properties obtained from test specimens, *in situ* tests and other methods to properties corresponding to the assumptions made in the calculation model;

— measurement error, conversion factor uncertainty and statistical uncertainty; and

— variance reduction by appropriate methods (if relevant).

Additional guidance is given in A.5.3.

5.4 Testing and instrumentation

Where there is uncertainty regarding the behaviour of foundations, testing or instrumentation should be undertaken. Possible methods include the following:

a) loading tests

Loading tests or large-scale field tests should be performed where there is particular uncertainty in the foundation capacity and where safety and/or economy are of particular importance.

b) model tests

Model tests should be performed where:

— the foundation configuration differs significantly from earlier configurations where operational experience exists;

— the soil conditions differ significantly from those where operational experience exists;

— new methods of installation or removal are envisaged; or

— a high degree of uncertainty exists as to how the structure or its foundation will behave.

c) temporary instrumentation

Structures should be fitted with temporary instrumentation where:

— the installation method presupposes the existence of measured data for control of the operation; or

— an installation method is to be applied with which little or no experience has been gained.

d) permanent instrumentation

Structures should be fitted with permanent instrumentation where:

— the safety or behaviour of the foundation is dependent on active operation;

— the foundation configuration, the soil conditions, or the actions differ substantially from those with which experience has been gained;

— there is a need for monitoring the whole foundation with regard to penetration, settlement, tilt, or other behaviour; or

— the method of removal presupposes the existence of measured data for control of the operation.

6 Geotechnical data acquisition and identification of hazards

6.1 General

The determination of geotechnical parameters, and the assessment of geological hazards and constraints result from an integrated study of the area using geophysics, geology and geotechnical engineering

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(additional guidance is provided in ISO 19901-8). Geophysical surveys should be performed before the geotechnical investigation.

Geophysical data are acquired to develop a geological model so as to better understand depositional and other processes and features of an area. The geophysical data are also used to help interpret the stratigraphy from geotechnical boreholes, to define lateral variability across a site, and to provide guidance on optimizing the location of the proposed facilities. Incorporation of geotechnical data into the geological model gives insight into the potential impact of geological conditions on man-made facilities, such as structures, pipelines, anchors and wellheads.

6.2 Shallow geophysical investigation

Shallow geophysical investigation can provide information about soil stratigraphy and evidence of geological features, such as slumps, scarps, irregular or rough topography, mud volcanoes, mud lumps, collapse features, sand waves, slides, faults, diapirs, erosional surfaces, gas bubbles in the sediments, gas seeps, buried channels, and lateral variations in stratum thicknesses. The areal extent of shallow soil layers can sometimes be mapped if good correspondence is established between the soil boring and *in situ* test information and the results from the seabed surveys.

The types of equipment for performing shallow geophysical investigation that should be considered are discussed below.

a) Echo sounders or swathe bathymetric systems (in which a series of sweeps of the bathymetric equipment are used) define water depths and seafloor morphology. On complex seafloors, swathe systems have the advantage of providing higher data density and better definition of variable topography. Seismic three-dimensional data acquired for exploration purposes also provide useful data for developing water-bottom (bathymetry) maps.

   These data should only be used for preliminary evaluations because the resolution could be of the order of a few metres depending on the variability of the topography.

b) Sub-bottom profilers (tuned transducers) define structural features within the near-surface sediments.

NOTE 1 These systems can also provide data to develop water-bottom maps.

c) Side-scan sonar defines seafloor features and seafloor reflectivity.

NOTE 2 Backscatter measurements from some swathe systems can also provide morphological information.

d) Seismic sources, such as boomers or minisparkers, can define the structure to deeper depths up to approximately 100 m below the seafloor and either single or tuned arrays of sparkers, air guns, water guns or sleeve-exploders can define structure to deeper depths and can tie in with deep seismic data from reservoir studies. Seismic source signals are received either with single channel analogue or multi-channel hydrophones. Digital processing of the recorded signals enhances the quality of the images recorded and removes extraneous noise and multiples from the recorded signals.

e) Seabed refraction equipment provides information on the stratification of the top few metres of the seabed.

Shallow sampling of near surface sediments using drop, piston or grab sampling, or vibrocoring together with cone penetrometer tests along geophysical tracklines can be useful for calibration of results and improved definition of the shallow geology.

Direct observation of the seafloor using a remotely operated vehicle (ROV), autonomous underwater vehicle (AUV), or manned submersible can also provide important confirmation or characterization of geological conditions.

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6.3 Geological modelling and identification of hazards

6.3.1 General

The nature, magnitude, and return intervals of potential active geological processes should be evaluated by site investigation techniques. Judicious use of analytical modelling can provide input for determination of the effects of active geological processes on structures and foundations. Due to uncertainties associated with definition of these processes, a parametric approach to studies is also helpful in the development of design criteria.

A geological model is constructed by hypothesizing a depositional process. The geophysical data should be mapped within the context of the hypotheses made. Features within the same geological period should be mapped together. Features not associated with a particular process should be mapped separately. If necessary, the mapping strategy should be adjusted to fit the model until agreement exists between the data and the model. The results of the geological modelling phase should ideally allow the interpreter to discuss in a report how features have developed over time, in order to allow assessment of how the features can affect future man-made developments.

Some of the more familiar geological processes, events and conditions are discussed in 6.3.2 to 6.3.7.

6.3.2 Earthquakes

Seismic actions shall be considered in design of structures for areas that are determined to be seismically active. Areas are considered seismically active on the basis of the historical record of earthquake activity, both in frequency of occurrence and in magnitude, or on the basis of a tectonic review of the region (for more details, see ISO 19901-2).

Seismic considerations for such areas shall include investigation of the subsurface soils for instability due to liquefaction, submarine slides triggered by earthquake activity, proximity of the site to seismogenic faults, the characteristics of the ground motions expected during the life of the structure, and the acceptable seismic risk for the type of operation intended. Structures in shallow water that can be subjected to tsunamis shall be investigated for the effects of the resulting actions.

6.3.3 Fault planes

In some offshore areas, fault planes can extend to the seafloor with the potential for vertical and horizontal movement. Fault movement can occur as a result of tectonic activity, removal of fluids from deep reservoirs or long-term creep related to large-scale sedimentation or erosion. Siting of facilities in close proximity to fault planes intersecting the seafloor should be avoided, if possible.

If circumstances dictate siting structures near potentially active faults, the effect of future fault movement on the foundation shall be assessed. If these effects are shown to be detrimental, the magnitude and time scale of expected movement shall be estimated on the basis of a geological study for use in the design of structures. See A.6.3.3 for more guidance.

6.3.4 Seafloor instability

Movements of the seafloor can be caused by ocean wave pressures, earthquakes, soil self-weight, hydrates, shallow gas, faults, or other geological processes. Weak, underconsolidated sediments occurring in areas where wave pressures are significant at the seafloor are susceptible to wave-induced movement and can be unstable under very small slope angles. Earthquakes can induce failure of seafloor slopes that are otherwise stable under the existing soil self-weight and wave actions.

Rapid sedimentation (such as actively growing deltas), low soil strength, soil self-weight, and wave-induced pressures are generally controlling factors for the geological processes that continually move sediment downslope. Important design considerations under these conditions include the effects of large-scale movement of sediment (i.e. mud slides and slumps) in areas subjected to strong wave pressures, downslope creep movements in areas not directly affected by wave/seafloor interaction and the effects of sediment erosion and/or deposition on structure performance.

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The scope of site investigations in areas of potential instability shall focus on identification of metastable geological features surrounding the site and definition of the soil engineering properties required for modelling and estimating seafloor movements.

Estimates of soil movement as a function of depth below the seafloor based on geotechnical analyses can be used to predict actions on structural members. Geological studies employing historical bathymetric data or age dating of sediments can be useful for quantifying deposition rates during the design life of the facility.

6.3.5 Scour and sediment mobility

Scour is the removal of seabed soils by currents and waves. Such erosion can be due to a natural geological process or can be caused by structural components interrupting the natural flow regime above the seafloor.

From observations, seafloor variations can usually be characterized as some combination of the following.

a) local scour

   Steep-sided scour pits around foundation components such as piles and pile groups, as seen in flume models.

b) global scour

   Shallow scoured basins of large extent around a structure, possibly due to overall structure effects, multiple structure interaction, or wave-soil-structure interaction.

c) overall seabed movement of sand waves, ridges, and shoals that would also occur in the absence of a structure

   Such movements can result in seafloor lowering or rising, or repeated cycles of these. The addition of man-made structures often changes the local sediment transport regime that can aggravate erosion, cause accumulation, or have no net effect.

Scour can result in removal of vertical and lateral support for foundations, causing undesirable settlements of shallow foundations and overstressing of foundation components. Where scour is a possibility, it shall be taken into account in design and/or its mitigation shall be considered (see A.8.1.4.2.7.8 and A.8.5.2).

6.3.6 Shallow gas

The presence of either biogenic or petrogenic gas in the pore water of shallow soils is an important consideration to the engineering of the foundation. *In situ* natural gas can be either gaseous (present as gas dissolved within the pore space or free gas) or bound with water to form a solid (known as hydrate). In addition to being a potential drilling hazard during both site investigation soil borings and oil well drilling, the effects of shallow gas can be important to foundation engineering. The effect of dissolution and expansion of gas in recovered soil samples shall be taken into account in developing geotechnical parameters for design.

6.3.7 Seabed subsidence

The nature of the soil conditions and the reservoir and extraction processes should be investigated to establish whether subsidence of the seabed is likely to occur during the field life. Subsidence, where this is a possibility, shall be taken into account in design.

The potential for seabed subsidence due to reservoir compaction shall be considered. The magnitude of surface settlements will depend on the reservoir dimensions, compressibility of reservoir rocks and the anticipated pressure drop. Where the magnitude of subsidence is uncertain, an increase in air gap shall be considered.

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6.4 Geotechnical investigation

Knowledge of the soil conditions existing at the proposed location of any structure is necessary to develop a safe and economical design. Details about equipments and procedures for marine soil investigations are provided in ISO 19901-8.

Geotechnical surveys and geotechnical investigation data should be combined in an engineering geological model of the region so as to develop the required design parameters.

6.5 Carbonate soils

When performing site investigations in frontier areas or areas known or suspected to contain carbonate material, the investigation should include diagnostic methods to determine the existence of carbonate soils. Particularly in sands and silts that contain in excess of 15 % to 20 % carbonate material, foundation behaviour can be adversely affected. In these soils, a carefully developed field and laboratory testing programme can be warranted (see A.6.5 and ISO 19901-8).

7 Design of shallow foundations

7.1 General

Shallow foundations in the context of this document include foundations placed directly on the seabed without embedment and embedded foundations with a maximum embedment of half the shortest plan dimension, i.e. an embedment ratio of 0,5. The guidance provided can be applicable to foundations with embedment ratios up to 1,0 but specialist advice should be sought in these cases.

All activities shall be performed by competent personnel with the qualifications and experience necessary to meet the objectives of these international standards.

General equations to be considered when evaluating the stability and serviceability (displacement) of shallow foundations are given and are applicable to idealized conditions. Additional equations that can be used for design are provided in A.7 together with discussion of limitations and alternative approaches.

The following is noted.

— It is primarily intended that these recommendations be applied to the design of temporary foundations, such as jacket mudmat foundations with shallow skirts used for temporary support during installation, under idealized conditions.

— The methods presented are intended for combinations of dead action, variable live action and environmental action (i.e. wind, wave and current); where these arise from transient or cyclic actions they are considered as quasi-static actions. Specialist advice shall be sought where more complex dynamic analysis is required, such as where the inertia of the structure or foundation soils is important (e.g. seismic loading).

— These recommendations can also be considered for foundations that are to be used for permanent support. This is applicable only where permanent foundations are subject to simple loading and soil conditions, for which the recommendations remain valid; or for situations where foundation failure would result in displacements and rotations that could be shown to have minimal environmental, safety and economic consequences.

— Alternative design approaches can be more applicable for design of shallow foundations used in more complicated and critical situations, such as complex seabed conditions or complex multi-directional cyclic actions (including cases in which cyclic uplift actions can largely offset the foundation dead weight). Advanced analysis techniques such as finite element modelling can also be used to confirm that design requirements are met for more complex situations.

— Ultimate limit state under uplift requires special attention with due regard to negative pore pressure (suction), adhesion, permeability of the soil, drainage paths, duration of the action and geometry of the foundation. Unless other failure modes are more critical, such as where separation of the foundation from the seabed occurs due to drainage, uplift resistance can be analysed as a reverse bearing problem applying the relevant equations for general shear failure in compression presented below and in A.7.

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— For large concrete gravity base structures and mobile offshore units, the requirements in this clause shall be supplemented and/or modified by requirements given in ISO 19903, ISO 19905-1 and ISO/TR 19905-2.

For many situations, it is advantageous to equip the foundation plate with vertical skirts around its periphery that penetrate into the seabed. If the foundation area is large, interior skirts should also be provided to form skirt compartments under the plate foundation. The presence of the skirts will in most cases (i) increase foundation stability, (ii) decrease foundation deformation, and (iii) reduce susceptibility of the foundation to seabed scour.

For shallow foundations equipped with seabed penetrating skirts, the following is noted:

— care should be taken to appropriately transfer the seafloor actions to skirt tip level, as described in A.7.2.3;

— when assessing stability based on loading at skirt tip level, it is important to ensure that the skirt depth to spacing ratio is sufficient to avoid internal failure mechanisms forming within the skirted compartment, which might lower the overall capacity. A maximum ratio of between 5 and 10 is typically required to achieve this, although this depends on the site specific soil profile and gravity load. A case-specific analysis is required to confirm an acceptable ratio.

7.2 Principles

7.2.1 General principles

The following general principles shall be considered in assessing stability of shallow foundations:

— bearing failure constitutes any failure mode that could result in excessive combinations of vertical displacement, lateral displacement, or overturning rotation of the foundation; while pure sliding or torsional failure corresponds to a failure mode where the foundation translates or twists only in a horizontal plane;

— foundation stability shall be analysed by limit equilibrium methods ensuring equilibrium between design actions and design resistance. The shape and location of the critical failure mechanism shall be determined, which will depend on the design actions, soil stratification and foundation geometry. Due consideration shall be given to the possibilities of excessive displacement and deformation of the foundation soil, and where these are critical, more complex analysis approaches are required;

— calculations using alternative methods of analysis shall include an explanation of any possible differences due to the method adopted;

— design actions shall be determined with due consideration of the design life of the foundation;

— undrained calculations shall be adopted where no drainage, and hence no dissipation of excess pore pressure, occurs during loading. This can occur as a result of the rate of loading or the impermeable nature of the soil. In contrast, drained calculations shall be adopted where no excess pore pressures arise during loading. Analysis of foundations subject to partial soil drainage during the loading event is complex, and specialist advice shall be sought in these cases;

— design can be based on serviceability (rather than stability) criteria, whereby the deformation of the foundation is assessed against allowable movement criteria. The appropriateness of adopting this approach will depend on the type of structure and its installation. The selection of appropriate soil moduli (especially considering strain dependency) is essential in calculation of serviceability limit states;

7.2.2 Sign conventions, nomenclature and action reference point

Vertical (Q), horizontal (H), overturning (M) and torsional (T) actions are centric and act at a reference point (RP) taken as the midpoint of the foundation at seafloor level, see Figure 1. This is the point of structural action transfer. H and M may be co-planar

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