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Guidelines for jack-up rigs with particular reference to foundation integrity

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Guidelines for jack-up rigs with particular reference to foundation integrity

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A jack-up rig is typically used at a dozen or more sites during its service life. It can therefore be expected to encounter a range of water depths, environmental load conditions and soil types. For each candidate site, an assessment is conducted to determine the suitability of the unit for the site and to provide data for the installation operation. Such assessments are normally conducted using the SNAME Technical and Research Bulletin 5-5A.

This report addresses the foundations of jack-up rigs, with the overall objective of determining current knowledge and assessment practices. An exhaustive literature search was initially conducted to establish a database of knowledge. A review of the collated data identified case histories of foundation problems, and in turn the major challenges associated with foundation assessment. Various foundation topics (e.g. punch through, bearing and sliding capacity, existing footprints, rack phase difference, etc.) were then examined in depth and recommendations made. In many cases the recommendations are a reaffirmation of existing guidance; in other cases some improvements to the existing guidance are suggested. In the case of rack phase difference, there is no guidance in SNAME and therefore specific provisions have been developed herein.

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1 EXECUTIVE SUMMARY

This document summarises a study undertaken by MSL Engineering Limited for the Health and Safety Executive. The study concerns the foundations of jack-up rigs, with the overall objective of determining current knowledge and assessment practices. In addition, some design guidelines on site specific integrity issues have been prepared.

The work began with an exhaustive literature review; over 250 documents were sourced for the study. Case histories were collated from the sourced documents and over 50 incidents relating to jack-up foundation problems were identified. Punch-through (rapid penetration through a stronger soil layer overlying a weaker one) has the highest rate in foundation incident causes, accounting for over 50% of all foundation incidents and the great majority of fatalities that have occurred in jack-up foundation related accidents. The second highest rate in foundation incident causes is for uneven seabed / scour / footprints.

The information contained in the literature, including existing guidance within SNAME and ISO documents, was assessed under various headings or topics:

- punch-through
- bearing failure and settlement
- sliding failure
- footprints
- rack phase difference (RPD)
- scour
- layered soils
- foundation fixity
- jack-up spudcan and jacket pile interaction
- cyclic loading
- debris
- shallow gas
- seafloor instability
- liquefaction.

For each topic and as appropriate: definitions are given, the effects on foundation behaviour are described, analysis/assessment approaches are summarised, mitigation methods are presented, and recommendations are made. In many cases the recommendations are a reaffirmation of existing guidance; in other cases some improvements to the existing guidance are suggested. In the case of rack phase difference, there is no guidance in SNAME or ISO and therefore specific provisions for RPD have been developed.

A number of flowcharts have been prepared. The flowcharts serve the following purposes:

- A visual overview of potential problems that may be encountered during the jack-up installation phase
- A visual overview of potential problems that may be encountered during in-service operations
- A summary of the jack-up foundation design process
- Detailed design process for selected topics.

2 INTRODUCTION

This report has been prepared by MSL Engineering Limited (MSL) for the Health & Safety Executive (HSE) following various HSE/MSL discussions, and relates to the preparation of guidelines on the safety and integrity of jack-up rigs, with particular reference to foundation integrity.

2.1 BACKGROUND AND NEED FOR STUDY

Over the past few years, a large number of research and technology development projects have been carried out in the UK and elsewhere on jack-up integrity. One of the primary focuses has been the integrity of jack-up foundation systems and associated risks.

Whilst a significant amount of useful information has been generated, there is an increasing recognition that this information is diverse and not easily accessible by the practising engineer. A proposal was therefore prepared for HSE's review and consideration, with the following target objectives:

1. To review/capture all information on the integrity of jack-up rigs, with specific focus on foundation and soil-structure interaction, including case histories.
2. To prepare a document on current knowledge and assessment practices.
3. To prepare engineering guidelines on site specific integrity issues.

2.2 SCOPE OF WORK

The following scope of work was proposed to meet the above stated objectives:

i. Capture all documents related to the integrity of jack-up rigs, including standards and recommended practices.

ii. Undertake review of the present-day state-of-the-art and state-of-practice.

iii. Identify and catalogue all relevant case histories.

iv. Undertake a critical appraisal of all the foundation related matters, including the following:

- Installation of jack-up rigs in close proximity to jacket structures, i.e. jack-up spudcan/jacket pile interaction
- Initial penetration and punch-through
- Settlement under storm loading/bearing failure
- Foundation fixity
- Sliding failure
- Scour
- Seafloor instability
- Shallow gas pockets

- Existing debris
- Sloping foundation and eccentric loading, and significance of Rack Phase Difference (RPD)
- Previous footprints
- Layered soils
- Effects of cyclic loading
- Liquefaction.

v. Prepare guidelines with specific focus on foundation related matters.

vi. Prepare report covering all work carried out and all findings.

3 AVAILABLE INFORMATION

3.1 GUIDANCE DOCUMENTS

There are two principal sets of guidance documents covering design and assessment engineering of jack-ups, and these are discussed below. In this report, they are generally referred to as SNAME [(1)] and ISO [(2)] for simplicity.

3.1.1 SNAME

The primary guidance used by the jack-up industry is "Society of Naval Architects and Marine Engineers Technical and Research Bulletin 5-5A". The first edition of SNAME was issued in May 1994 and was based on a Joint Industry-Funded Project involving all sections of the industry. The background to the project was disseminated at a seminar [(3)] held at City University, London, in September 1993. There have been two revisions since the first edition. The first revision was issued in May 1997 and the second in January 2002. Unless noted otherwise, in this report a reference to SNAME implies the latest version (2002).

The SNAME T&R Bulletin 5-5A 2002 contains four documents:

- T&R5-5—"Guideline for Site Specific Assessment of Mobile Jack-Up Units" (First Edition – May 1994)
- T&R5-5A—"Recommended Practice for Site Specific Assessment of Mobile Jack-Up Units" (First Edition – Rev 2, January 2002)
- Commentaries to Recommended Practice for Site Specific Assessment of Mobile Jack-Up Units (First Edition – Rev 2, January 2002)
- Example ("Go-By") Calculation Using Recommended Practice For Site Specific Assessment of Mobile Jack-Up Units (Preliminary Issue – May 1994).

The stated purpose of the guideline (T&R5-5) is to identify the factors that are likely to be the main concerns for any site assessment of a jack-up unit. It is not intended to be used as guidance for design or construction. The Recommended Practice document (T&R5-5A) provides specific provisions for use with the T&R5-5 Guideline. Each assessment should cover the areas of the Recommended Practice as appropriate for the particular jack-up and location. The Recommended Practice does not intend to impose calculation methods or procedures and leaves the engineer freedom to apply alternative practices within the framework of the accompanying Guideline. The Commentaries to the Recommended Practice provide background information, supporting documentation, and additional or alternative calculation methods as applicable.

A summary of the coverage within SNAME for the topics investigated in this study is included in Table 1.

3.1.2 ISO

The working draft 'C' of International Standard ISO, Part 1, entitled 'Petroleum and Natural Gas Industries—Site specific assessment of mobile offshore units (ISO/WD 19905-1.4)', was issued in October 2003 for review and comment. Part 2 of this document is the associated commentary. These documents are to be published as international standards in the future. They are very largely based on the SNAME documents, the main difference being purely one of format. Unless noted otherwise, in this report a reference to ISO implies the working draft 'C', ISO/WD 19905-1.4 (2).

There is also an ISO standard "ISO 19901-4:2003(E)" (4) on geotechnical and foundation design. However, this document does not cover all investigated topics in this study.

A summary of ISO coverage for the topics investigated in this study is included in Table 1.

Table 1 SNAME and ISO coverage of foundation issues

No	*Topic*	*Relevant Clause in SNAME*	*Relevant Clause in ISO/ WD 19905-1.4*	*Level of Detail (SNAME)*	*Level of Detail (ISO)*
1	Punch-Through	6.2.6;8.3.5	A.9.3.2.7	Detailed	Detailed (mainly follows SNAME)
2	Settlement under Storm Loading	6.3.3.4	A.9.3.3.2.4	Mentioned	Mentioned
3	Sliding	6.3.3; 6.3.4; 8.3.1	13.7.2; A.9.3.3.2; A.9.3.3.3	Detailed	Detailed (mainly follows SNAME)
4	Previous Footprints	6.4.2	9.3.4.2	Partly detailed	Partly detailed (mainly follows SNAME)
5	Rack Phase Difference	-	-	No mention	No mention
6	Scour	6.4.3	9.3.4.3	Mentioned	Mentioned
7	Layered Soils	6.2.6	A.9.3.2.7	Detailed	Detailed (mainly follows SNAME)
8	Foundation Fixity	5.3; 6.3; 8.3	13.7.4; A.8.6.3; A.9.3.3.3; A.9.3.3.4	Detailed	Detailed (mainly follows SNAME)
9	Spudcan/Pile Interaction	6.4.6	9.3.5; A.9.3.5	Mentioned	Mentioned
10	Cyclic Loading	6.4.4	9.3.4.5	Mentioned	Mentioned
11	Existing Debris	3.13.1	A.9.2.1.2	Mentioned	Mentioned (mainly follows SNAME)
12	Shallow Gas Pockets	6.4.5	9.3.4.6	Mentioned	Mentioned
13	Instability of Seafloor	6.4.4	9.3.4.4	Mentioned	Mentioned
14	Liquefaction/Pore -pressure	6.4.4	9.3.4.5	Mentioned	Mentioned

As suggested by Table 1, ISO is very similar to SNAME since it is based on SNAME. However, ISO has more guidance than SNAME in some areas.

3.2 PUBLIC DOMAIN LITERATURE

An extensive literature search was conducted to obtain comprehensive information regarding jack-up foundations. Data has been collected from the early years of jack-up use to the present-day. The following methods to identify and subsequently source documents were followed during this stage of the work:

(a) Internet Searches by keyword

The World Wide Web was searched for information using keywords in Internet search engines. A number of keywords were used in the searches including:

- Spud-can, spud can, spudcan
- Jack-up, jackup, jack up
- Footing
- Fixity
- Initial penetration
- Debris
- Scour
- Footprint
- Shallow gas, etc.

(b) Internet searches of specific sites

Also, a broad search has been done within specific websites that were thought likely to contain relevant information. These sites include:

- Health and Safety Executive (HSE) website for OTO, OTH, OTI, RR reports, safety notices and more.
- Minerals Management Service (MMS) website
- International Association of Drilling Contractors (IADC) website
- United Kingdom Offshore Operators Association (UKOOA) website
- Centre of Offshore Foundations Systems (COFS) website, University of Western Australia. A large amount of relevant information and papers are available for downloading from this website.

(c) MSL Engineering in-house library

(d) Conference papers

- Offshore Technology Conference (OTC), 1969-2003
- Jack-Up Platforms Conference, 1987-2003
- International Society of Offshore and Polar Engineers Conference (ISOPE), 1991-2003
- Offshore Mechanics and Arctic Engineering Conference (OMAE), 1994-2003
- Behaviour of Offshore Structures conference (BOSS), 1976-2003

These conferences were checked for all years they have been held.

(e) Journals and magazines

Journals/magazines listed from citation and Internet searches were examined. The journals and magazines included: Oil and Gas Journal, Offshore Magazine, Journal of Offshore Mechanics and Arctic Engineering, Marine Structures, Japanese, Canadian and British Geotechnique magazines. Marine Structures and British Geotechnique journals were looked at with a greater effort as these appeared to have a greater proportion of relevant articles.

(f) Authors were contacted directly for further information on tracked papers and research.

(g) Cited references
Further documents were found from the reference lists of sourced papers and documents.

Over 350 references were initially identified. Following a screening process, based mainly on document title, a total of 250 documents were thought to be of particular interest and were therefore sourced to form a "project library". The documents in the project library were then reviewed, categorizing each document according to one or more of the various topic areas discussed in this report, and entered into a database. This stage of work is summarised in Table 2, which shows the breakdown by topic area and publication year. (Note, some documents cover more than one topic area.)

Table 2 Classification of references by topic area and publication date

Topic Area	*Before 1980*	*1981-1990*	*1991-2000*	*After 2001*	*Total*
Spudcan / Pile Interaction	0	8	8	7	23
Punch-Through	0	15	13	11	39
Settlement	1	4	14	10	29
Sliding	1	3	13	6	23
Scour	0	3	2	6	11
Instability of Seafloor	0	3	2	2	7
Shallow Gas	0	0	4	1	5
Debris	0	1	1	1	3
Rack Phase Difference	0	5	1	7	13
Footprints	0	3	2	10	15
Layered Soils	1	6	3	3	13
Cyclic Loading	0	7	20	8	35
Liquefaction / Pore-Pressure	1	1	9	2	13
Fixity	2	17	54	23	96
Fatigue	0	0	1	4	5
Risk of Impact with Jacket	0	0	4	4	8
Case History	0	10	13	10	33
Unclassified	3	2	13	9	27
Total No. of Documents	7	44	108	71	230

3.3 CASE HISTORIES

During the collation and classification of obtained literature, those documents identifying case histories of failure incidents were tagged. Some of these incidents are discussed in detail in these references but the majority of them are just merely mentioned as having occurred.

A table of the available data including jack-up name, location, date and cause of accident was prepared. Further investigation was carried out using the Internet to acquire more information (where available) about missing data for each case.

It should be noted that in reality there might be many other incidents that have never been reported in the public domain. Therefore, the following set of data is not a complete database and only reflects a limited experience. However, it is a complete collection of all available data on case histories in the project library. The data is compiled into Table 3.

One third of jack-up accidents have been associated with foundation problems [5 & 6]. Causes are classified to categories such as punch-through (during preloading or hurricane/storm events), uneven seabed / scour / footprint, seafloor instability / mudslide / seabed slide / volcanic activity, sliding of mat foundation, unexpected penetration and others.

Over fifty incidents have been tracked down from records, giving rise to a wide range of damage, from no damage to major damage / total loss. An examination of Table 3 and the derived Figure 1 provides the following observations.

- Punch-through has the highest rate in incident causes, representing 53% of all incidents. Punch-through problems can be further subdivided: 8% of all incidents are associated with punch-through caused by hurricanes, 14% with punch-through during preloading, and 31% with no stated underlying punch-through cause.

- The second highest rate in incident cause is for uneven seabed/ scour/ footprint. Some 15% of all incidents are covered by this category.

- A comparison of legged and mat foundation jack-ups seemed appropriate and interesting. Sliding of foundation has been the major problem with mat foundation jack-ups while punch-through is restricted to jack-ups with spudcan type foundations. In five of the six mat foundation incidents, the jack-ups shifted in position in hurricanes. The remaining incident was due to a mudslide and this case is included in the seafloor instability category.

- Other incident causes include soil failure, overturning and tilting and these account for 8% of all failures.

- From 24 fatalities reported in all 51 incidents, 19 are due to punch-through failure. The other 5 occurred in 1983 on the “60 years of Azerbaijan” jack-up, due to volcanic activity.

- Of the 6 recorded incidents in the North Sea, 5 are due to seabed instability/ scour/ footprint. (The other one incident is due to punch-through.) There has been no fatality attributed to jack-up foundation problems in the North Sea according to the data captured.

Table 3 Compilation of reported incidents involving jack-up foundation issues

Ref. No.	*Name*	*Foundation Type*	*Location*	*Designer*	*Year*	*Comment*	*Coverage*	*Notes*
7	Kolskaya	Legged	Norwegian North Sea	-	1990	Scour	Mention	-
7	West Omicron	Legged	Norwegian North Sea	-	1995	Punch-through, one leg sank 1.5 m	Mention	-
8	Monarch	3 Legged	Southern North Sea	Friede & Goldman L780 Mod V	2001-02	Scour, Eccentric Loading, RPD	Detailed	-
9	Monarch	Legged	Southern North Sea	Friede & Goldman Mod V	2002	2 legs damaged from uneven seabed/scour	Mention	-
9	Monitor	Legged	Central North Sea	Friede & Goldman Mod V	2000	1 leg damaged from uneven seabed	Mention	-
9	101	Legged	Central North Sea	KFEL's modified Mod V	2000	1 leg damaged from adjacent footprint	Mention	-
10	-	Legged	Gulf of Mexico	-	-	Punch-through	Partly detailed	-
10	-	-	Gulf of Mexico	-	-	Footprint	Partly detailed	-
11, 12 & 13	Dixilyn Field 81	3 Legged	Gulf of Mexico	-	1980	Additional penetration in Hurricane Allen	Mention	1
11, 12 & 13	Penrod 61	3 Legged	Gulf of Mexico	Le Tourneau, Inc	1985	Additional penetration in Hurricane Juan	Mention	1
6 & 13	Maverick I	Legged	Gulf of Mexico	-	1965	Overturned in hurricane BETSY(220' WD) / Punch-through	Mention	1

Ref. No.	*Name*	*Foundation Type*	*Location*	*Designer*	*Year*	*Comment*	*Coverage*	*Notes*
13	Pool Ranger 4	-	Gulf of Mexico	-	1997	Break-through or slide into crater	Mention	1
13	Pool 55	-	Gulf of Mexico	-	1987	Soil failure when drilling	Mention	1
6	Zapoteca	Legged	Gulf of Mexico	-	1982	Punch-through while jacking up	Mention	-
9	John Sandifer	Legged	Gulf of Mexico	Levingston 111S	2002	1 leg damaged from extra penetration	Mention	-
13	Transgulf Rig 10	Legged	Gulf of Mexico	-	1959	Capsized when preparing to move/ punch-through	Mention	1
13	Mr Gus 1	Legged	Gulf of Mexico	-	31 Mar 1957	Punch-through- Tilted 9 degrees. Later capsized in Hurricane Audrey (1 Fatality)	Mention	2
13	Penrod 52/ Petrel	Legged	Gulf of Mexico	-	9 Sep 1965	Punch-through & Capsized moving on then bit by Hurricane Betsy	Mention	2
13	Bigfoot 2	Legged	Gulf of Mexico	-	20 Oct 1987	2 Bow legs broke through while preloading- 21 deg list. 1 corner of hull 10' underwater. CTL	Mention	2
13	Keyes 30	Legged	Gulf of Mexico	-	23 Feb 1988	Bow leg punch-through 2 m while preloading legs bent-listed. Constructive total loss.	Mention	2
13	Western Triton 2	Legged	Golf of Mexico	-	8 Jan 1980	1 Leg break through 22'- Jacking up-Damaged Jacking System and chord-crew in sea	Mention	-
13	Dresser 2	-	Gulf of Mexico	-	28 Apr 1968	Overturned due to soils failure	Mention	2
5, 11, 12 & 13	Harvey Ward	Mat	Gulf of Mexico	-	1980	Mudslide (Total Loss) Mat Foundation	Mention	1
14	-	Legged	Gulf of Mexico	-	-	Punch-through	Mention	-

Ref. No.	*Name*	*Foundation Type*	*Location*	*Designer*	*Year*	*Comment*	*Coverage*	*Notes*
15	-	Legged	Gulf of Mexico	-	-	Punch-through	Mention	-
5	Triton II	Legged	High Island, Off Texas	-	1980	1 leg Punch-through during preloading, 2 other legs buckled	Partly detailed	-
10	-	Legged	Brazil	-	-	Punch-through	Partly detailed	-
10	-	Legged	Brazil	-	-	Punch-through	Partly detailed	-
6	High Island V	Legged	Brazil	-	1982	Punch-through while jacking up	Mention	-
16	Hakuryu 9	Legged	Bay of Bengal, North Sumatra	-	1987	Punch-through	Detailed	-
10	-	Legged	Brazil	-	-	Punch-through	Partly detailed	-
10	-	Legged	Brazil	-	-	Punch-through	Partly detailed	-
6	High Island V	Legged	Brazil	-	1982	Punch-through while jacking up	Mention	-
16	Hakuryu 9	Legged	Bay of Bengal, North Sumatra	-	1987	Punch-through	Detailed	-
16	Hakuryu 7	Legged	Bay of Bengal, North Sumatra	-	1987	Punch-through	Detailed	-

Ref. No.	***Name***	***Foundation Type***	***Location***	***Designer***	***Year***	***Comment***	***Coverage***	***Notes***
13	Dixilyn Field 83	Legged	Indian Ocean	-	20 Nov 1986	Starbd Leg broke through/capsized preloading@4m air gap off Bombay	Mention	2
9 & 13	60 Years of Azerbaijan	-	Caspian Sea	-	1983	Seabed failure/ volcanic action (5 Fatalities)	Mention	1
13	Baku 2	Legged	Caspian Sea	-	1976	Capsized and sank while drilling/ jacking up?/Punch-through? First location, age 0	Mention	1
13	Marlin 4	Legged	South America	-	1980	Jack house split, 3 legs damaged due to seabed Slide- Hull dropped-30ft down bow leg-when jacking	Mention	1
5 & 6	Rio Colorado I	Legged	Argentina	-	1981	Punch-through of one leg offshore	Partly detailed	-
6, 9 & 13	Gemini	Legged	Gulf of Suez	-	1974	Punch-through/ Leg failure whilst in-situ (18 Fatalities)	Mention	-
9	Victory	Legged	South Australia	Modec 300C-35	1996	3 legs damaged from Punch-through	Mention	-
9	Harvey Ward	Legged	Indonesia	Friede & Goldman Mod II	1998	3 legs damaged from Punch-through?	Mention	-
5	Gulftide	4 Legged	Sable Island, Canada	-	1977	Damaged due to scour	Partly detailed	-
9	57	Legged	South China Sea	Friede & Goldman Mod II	2002	Rapid Leg Penetration	Mention	-
9	Ekhabi	Legged	Persian Gulf	Lev MSC CJ50	2001	Punch-through	Mention	-

Ref. No.	*Name*	*Foundation Type*	*Location*	*Designer*	*Year*	*Comment*	*Coverage*	*Notes*
13	Bohai 6	-	West Pacific	-	1981	Slipped while on location	Mention	1
13	Roger Buttin 3	Legged	West Africa	-	9 Feb 1966	Legs penetrated faster than jacking due to weak clay, then capsized and sank	Mention	2
5, 6 & 13	Gatto Salvatico	Legged	East Africa, Off Madagascar	-	1974	Deeper Leg Penetration during a storm	Mention	-
12	Salenergy 1	Mat	-	-	1980	Shifted position in Hurricane Allen	Mention	-
12	J. Storm 7	Mat	-	-	1980	Shifted position due to scour in Hurricane Allen	Mention	-
12	Teledyne 17	Mat	-	-	1980	Shifted position due to scour in Hurricane Allen	Mention	-
12	Fjelldrill	Mat	-	-	1980	Tilted during Hurricane Allen-Damaged	Mention	-
12	Mr Gus 2	Mat	-	-	1983	Slide 2.5m off location in 8 m seas. No damage	Mention	-
12	Pool 50	Mat	-	-	1985	Slide off location in Hurricane Danny, Leaned towards a fixed structure, couldn’t jack down	Mention	-

Notes:

1: Total or major loss of jack-up on location
2: Total or major loss of jack-up while moving

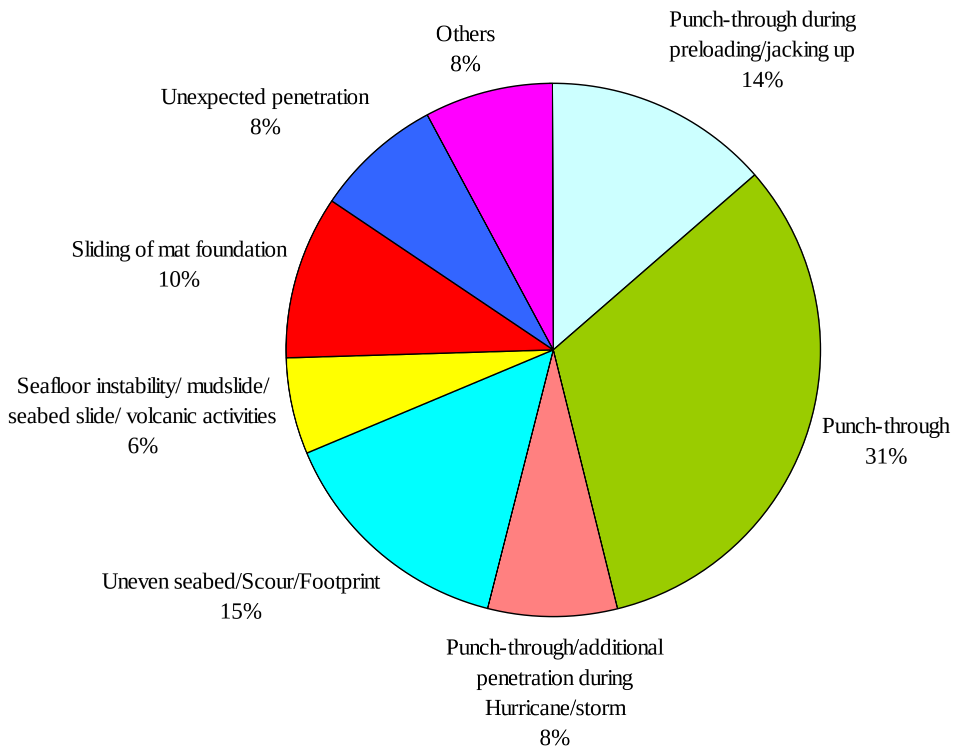


Figure 1 Case histories classified according to the cause of failure

4 ASSESSMENT OF INFORMATION

This Section 4 presents an assessment of the information collated in Section 3. The assessment has been conducted under a number of topics as presented in the following subsections. The interrelationship between the topics and where they fit in within the foundation design/assessment process is indicated in Figure 2.

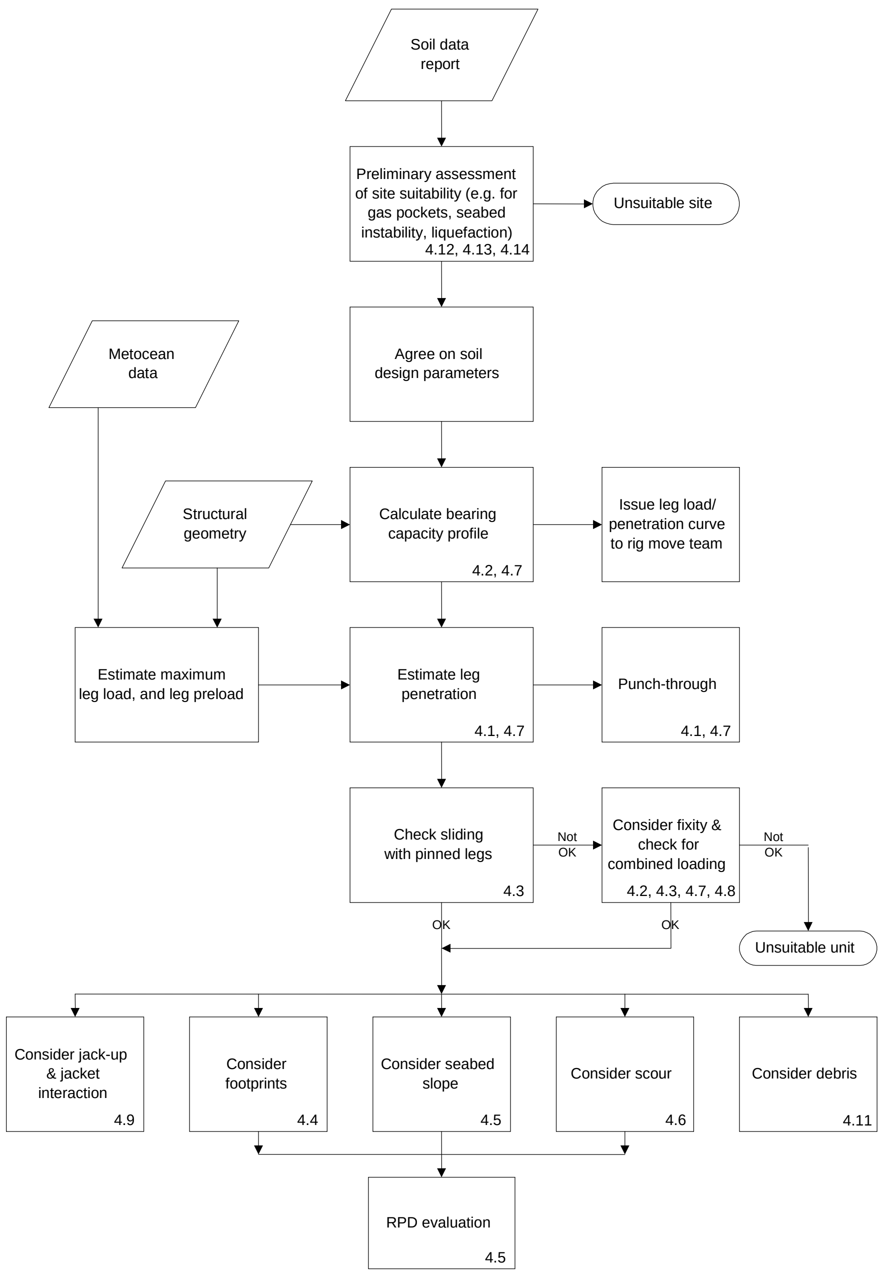


Figure 2 Overview of jack-up foundation design/assessment process (numbers refer to report sections)

4.1 INITIAL PENETRATION AND PUNCH-THROUGH

4.1.1 Definition

After initial set down of the legs on the sea bed and loading of the legs, unless founded on very hard clay or sand, the spudcans will penetrate the surface soils. Penetration will continue whilst the preloading operation is on-going until equilibrium is reached between the capacity of the soil and the forcing load. Mirza et al [17] comment that penetration depths in sand are small, but that in soft clays such as the Mississippi delta, penetration depths as much as 55m have been recorded. If the soil strength decreases with depth, or if a soft layer exists below a stronger surface stratum, the profile of capacity with depth may reduce over a range of depth, increasing again at greater depth. When such a profile exists, an unstable load equilibrium condition may develop since a small increase in load will cause the spudcan to penetrate further to try and balance the leg reaction. Rapid displacement ("punch-through") of the soft soils will continue until the required resistance is developed at a greater depth. If the distance through which the spudcan travels is significant, then damage may be caused to the leg both due to the dynamic consequence factor and the increased moments due to out-of-verticality of the jack-up.

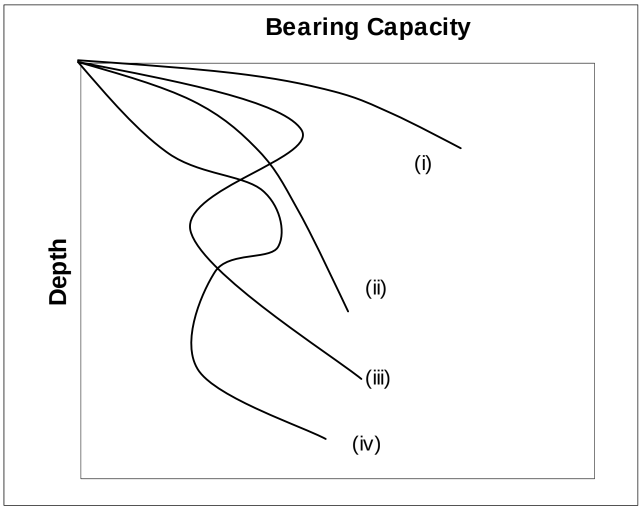
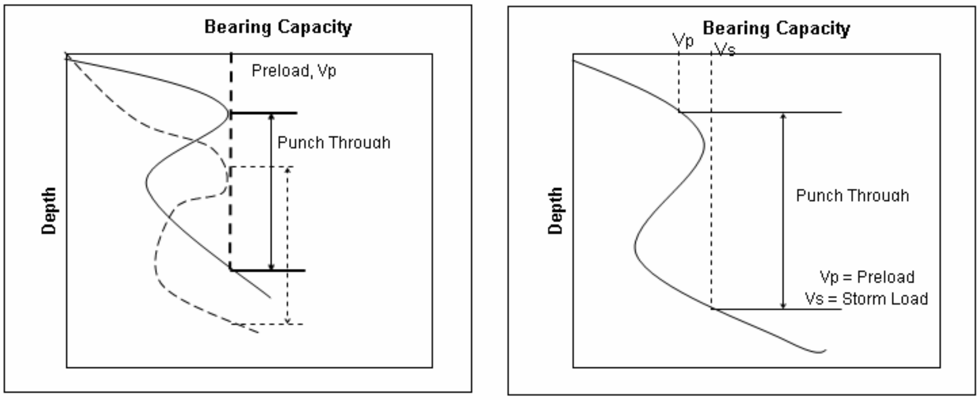


Figure 3 Typical bearing capacity profiles

The figure above show typical bearing capacity profiles. These are for (i) a hard soil profile, sand or clay, (ii) a normally consolidated clay, (iii) layered soils comprising dense sand or hard clay overlying a soft layer of clay, and (iv) a sandwich of hard clay or sand, bounded top and bottom by soft clay. When the hard soil in (i) sand/clay is loaded, penetration is small (and the full spudcan area may not participate); for the (ii) soft clay profile, with increasing strength with depth, the spudcan will penetrate and may bury until equilibrium is reached; for both the non-uniform soils (iii) and (iv) punch-through may occur as shown in the figure below.

Hambly points out [18] that a geometric instability condition can occur where the bearing capacity increases monotonically with depth, if the rate of increase falls below a critical value. In this condition, the leg loading rate due to tilting of the rig exceeds the ability of the leg penetration rate to maintain equilibrium.



(a) Installation punch-through (b) Punch-through in storm conditions

Figure 4 Punch-through scenarios

Punch-through may also occur in the in-place condition. McClelland et al [5] identify a condition of punch-through due to "storm overload". This may result when the storm load exceeds the preload, and this is combined with soils condition comprising a hard founding layer over a weak underlying clay or silt. Figure 4(b) illustrates punch-through under storm conditions.

4.1.2 Effects and causes of punch-through

The effects of an unexpected punch-through, which will occur at one leg, may be very severe, resulting in tilting of the jacket and possible damage to the legs. Since the imposed tilt will cause additional out-of-balance moments, this will lead to an increase in spudcan loading and hence further punch-through deformation.

Punch-through may arise due to any of the following conditions:

- Presence of a hard clay crust over softer soils which may stay uniformly soft or decrease with depth.
- Existence of sand over soft clay strata.
- Founding in a clay stratum which decreases in strength with depth
- Firm clay with sand or silt pockets (Rapoport and Young [10])
- A very soft clay where rate of increase of capacity does not match loading rate (Hambly[18]).

All of the above apart from the final are examples of layered soils, where a strong soil overlies a weak soil. It should be noted that soil strength may not be solely due to past geological process, or loading histories, but may occur due to the loading regime imposed by the jack-up prior to preloading. For instance, according to Rapoport and Alford [19] an artificial crust may be developed due to delays in preloading, and consolidation under self weight (lightship load).

4.1.3 Punch-through analysis

Punch-through analysis is carried out by determining the soil ultimate capacity profile with depth. For the condition of sand over a soft clay, the conventional analysis is to determine the lesser of the capacities of the equivalent flat footing in sand (e.g. by the Brinch-Hansen method given in Reference 20) and in sand overlying clay. Usually the lower capacity is developed in clays, and the clay capacity is determined using Skempton's equation, as described in Section 4.2. When checking the capacity of a two layer strong over weak profile McClelland et al [5] recommend that one of the following is satisfied:

- The bearing capacity of the strong layer is weak enough to allow the spudcan to penetrate completely through that layer
- The bearing capacity of the strong layer is strong enough to support the footing with safety.

The first criterion will be satisfied if the upper bound capacity estimate is less than the variable weight and light ship weight. If the lower bound bearing capacity of the strong layer exceeds the preload by 50% the second criterion will be met.

It is usually assumed that there is no backflow over the spudcan. For the condition of backflow, the spudcan capacity is reduced. In layered soils the appropriate layered soil capacity is determined, as described in Section 4.7.

Svano and Tjelta [21] comment that a skirted spudcan will follow a different load penetration curve. Due to the low tip area of the skirts, the resistance will initially be low. Once the base touches the mudline the resistance will immediately increase. Therefore if the geometry of the skirts is designed so that the hard overlayer is penetrated during jacking, the full area of the soil plug will become available in the softer soils to allow safe preloading to proceed without risk of sudden punch-through. If the overlying soil is very hard, suction could be used to penetrate to ensure that the base makes full contact. Typical application of this is shown in Figure 5.

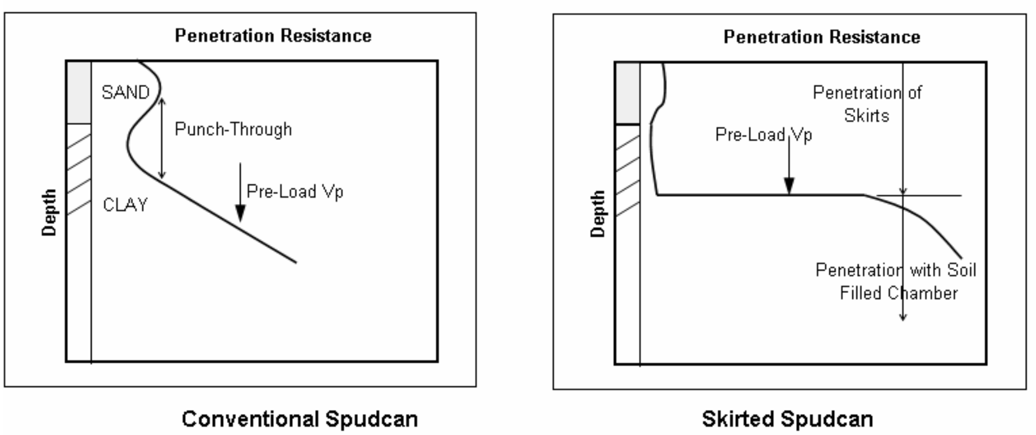


Figure 5 Punch-through profile for skirted spudcan

4.1.4 Load factors for preload

The resistance factor for preload bearing capacity (ϕ) as provided in SNAME is 0.90. This should be used in combination with load factors of 1.0 for the variable load and dead weight, and load factor of 1.15 for environmental and inertia load effects. The preload check is carried out as follows, using SNAME terminology:

$$Q_v = \gamma_1 V_D + \gamma_2 V_L + \gamma_3(V_E + \gamma_4 V_{Da})$$

In this equation γ_1 ,γ_2 and γ_3 are dead load, variable load and environmental load factors and γ_4 is the inertia load dynamic factor. The required preload V_{Lo} is then given by:

$$V_{Lo} = Q_v / \phi_p$$

where ϕ_p is 0.9.

This results in a global safety factor of between 1.11 and 1.27 depending on the ratio of load types. Historically a load factor of 1.0 has been used. The assumption has been that if the design storm load is exceeded, then additional capacity can be provided by additional settlement and enlargement (hardening) of the yield curve. However where the soils are such that punch-through could occur with a small overload during the assessment condition, then this should be incorporated by increasing the factor of safety as for instance recommended by McClelland et al [(5)]. The latter recommend a global factor of safety of 1.5 to be applied to the calculated preload to ensure that foundation failure does not occur, for instance in layered soils. To provide this load factor in susceptible soils would require a resistance factor ϕ_p of between 0.67 and 0.76 to be applied to the SNAME checks.

4.1.5 Mitigation of punch-through effects

Procedures for mitigating the possible consequences of punch-through have been described by Rapaport and Alford [(19)]. Approaches for reducing the sudden vertical displacement associated with punch-through include:

- Using a small air gap, so that any large vertical displacement is prevented by the buoyancy of the hull as it penetrates the water line and produces a draught.
- If required, preload in water so that the leg loads are reduced whilst penetrating the soft layers.
- Preloading by one leg at a time. This will also reduce the overall topsides load and therefore lessen the individual leg loads, and leaves the other legs free to jack if required.
- Use of jetting in the hard layers to allow penetration into the soft stratum under minimum load.
- Provide a guide distance as great distance as possible, to reduce the brace reactions during overload (see Section 4.5). This is part of the rig design and hence may not be an option.

4.1.6 Recommendations to avoid punch-through

If the soil strength includes a reduction with penetration depth, then there is a potential for punch-through. However the potential effects of a sudden reduction in spudcan support can be mitigated by the following procedures:

- Carry out a detailed soils survey of the site, including borehole sampling and CPT testing to obtain good quality soils data. The soils sampling should include both shallow borings and at least one deep borehole with a depth equal to 30 meters or the anticipated footing penetration plus 1.5 to 2.0 times the footing diameter, whichever is the greater. The borings should cover the expected footprint area of the jack-up rig.
- If spudcan data from previous experience in the location is available use this to back analyse and confirm the prediction methods for bearing capacity.

- Have in place procedures for reducing the spudcan loads during the potential punch-penetration phases including the use of buoyancy and zero air gap and preloading one leg at a time.
- Consider the use of jetting to penetrate the harder soils.

Finally, consideration may be given to the use of a different jack-up, with different spudcan geometry, to reduce bearing loads.

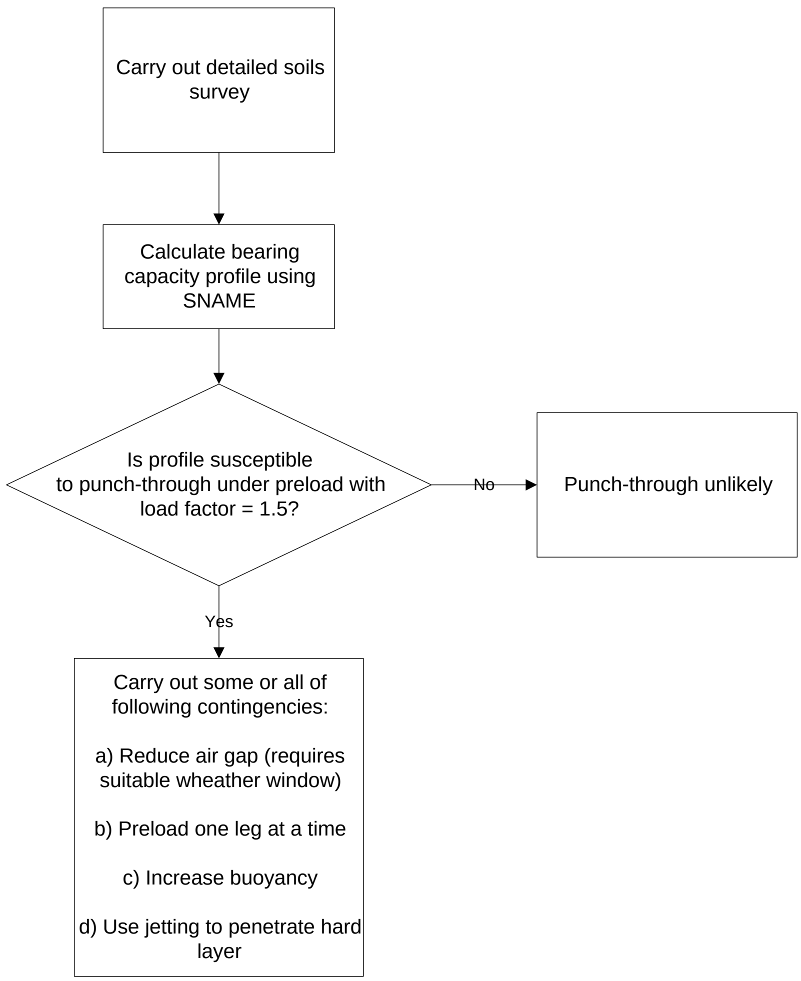


Figure 6 Punch-through assessment

4.2 BEARING FAILURE AND SETTLEMENT

4.2.1 Definition

The aim of preloading is to proof load the soils so that when subjected to the design storm conditions, the load envelope remains within the failure surface, and that settlements are minimal and acceptable. Whilst the preloading operation is a controlled soil failure which provides a test and measure of the capacity of the soil under vertical loading conditions, the foundations will be subjected to combined loading during storm conditions. This combined loading may include interacting vertical load, shear and moment. If during the in-place condition the vertical load for instance equals or exceeds the preload (“proof capacity” load), there will be no additional reserve to accommodate combined vertical load with shear and moment and bearing failure will ensue, accompanied by further penetration. This settlement will continue until the soils capacity balances the imposed load. The yield surface will harden and expand so that the combined load vector is on or inside the new failure surface.

Settlements can also occur due to long term consolidation or creep of the soils under load. These long term settlements are not considered here.

4.2.2 Bearing capacity

In the SNAME site specific assessment of mobile jack-up units, a three level procedure is specified for the foundation stability assessment. This is based on the Van Langen and Hospers [22] proposals:

- Step 1 **-** Bearing capacity and sliding resistance based on maximum preload leg reactions. A pinned footing is assumed.
- Step 2 (a) **-** Stability analysis of individual footings assuming pinned conditions.
- Step 2 (b) **-** Stability analysis of individual footings assuming soil springs.
- Step 3 **-** Displacement check based on a non-linear soil structure interaction allowing for load redistribution between the footings.

The Step 1 checks assume bearing capacity assessment based on vertical load only with no interaction from shear loads. Therefore traditional methods for calculating capacities in clays and sand are used. In determining the initial penetration depth under preload, the procedure as described in SNAME is as follows:

- Model the spudcan as an equivalent diameter flat circular plate
- Draw a plot of bearing capacity versus depth
- Using this curve, enter the required preload and determine the predicted penetration depth (Figure 7).

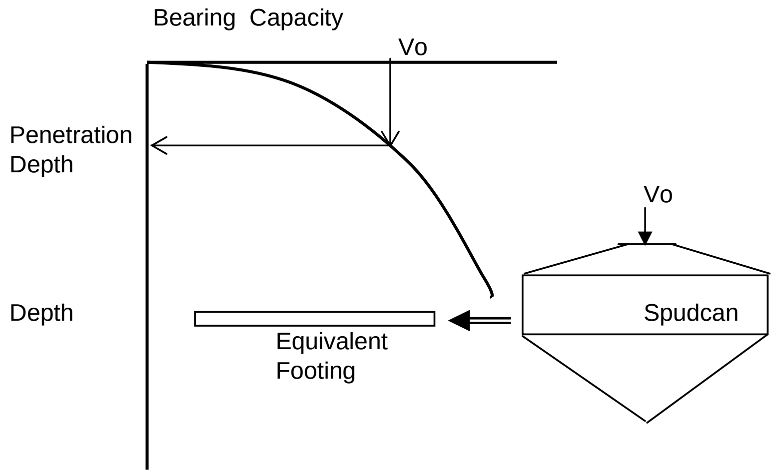


Figure 7 Bearing capacity profile

Clays

Bearing capacity in clays may be determined using the general ultimate bearing pressure equation:

$$q_u = N_c S_u s_c d_c + \gamma' V/A$$

The dimensionless bearing capacity factor N_c is given by established methods, for instance using Skempton or Vesic's equation or the Brinch-Hansen formulae [20]. The shear strength S_u is the average over a depth B/2 below the spudcan. Factors s_c and d_c are the shape factor and the depth factor respectively. Allowance for adhesion to the sides of the spudcan is not usually provided.

In Skempton's equation the modification is as follows, for unit bearing capacity:

$$q_u = N_c\, S_u\, (1 + 0.2D/B)\, (1 + 0.2B/L) \quad \text{where } N_c = 5.0$$

For a circular footing this reduces to:

$$q_u = 6S_u\,(1+0.2B/D)$$

For soils with increasing shear strength the Booker and Davis [20] factors are used. The incorporation of this modification is then:

$$q_u = F(N_c S_{uo} + \rho B/4) + \gamma' V/A$$

F is a coefficient which depends on $\rho B/S_{uo}$ and is described in Reference 20; ρ is the rate of shear strength increase; and S_{uo} is the mudline shear strength.

SNAME warns that bearing capacity may reduce due to the effects of cyclic loads. This is considered in Section 4.10.

Sands

In sands the generalized capacity is given as:

$$F_v = 0.5B\gamma N_\gamma s_\gamma d_\gamma + p_o N_q s_q d_q$$

Shape and depth effects are incorporated by factors s_γ and d_γ, and overburden is represented by the last term. For a circular spudcan this equation reduces to the following:

$$Fv = 0.3B_\gamma N_\gamma + \gamma D N_q$$

The coefficient s_γ and d_γ in the previous equation are replaced by 0.6 and 1.0 respectively, whilst s_q and d_q are both 1.0.

According to Craig and Chua [23] centrifuge testing indicates that there is a scale effect in extrapolating from small scale testing to the geometries associated with spudcans. This results in a lower bearing capacity, as calculated using the N_y term. SNAME notes that the failure mode in large spudcan foundations may include punch-through or local shear failure. Therefore SNAME recommends that the sand friction angle is reduced by about 5 degrees to account for this reduction.

Silts

Silts are considered by bounding the solutions using an upper bound solution as a drained material, loose sand and a lower bound solution as soft clay. The capacity may be reduced significantly by cyclic loading.

Conical spudcans

In assessing bearing capacities conical spudcans are usually modelled as flat plates. SNAME provides adjusted bearing capacity factors for different geometry of spudcan cone, and for rough or smooth footings, when founded in clay. These factors result in a lower capacity than those found using the classical N_c factors.

Although SNAME does not provide a design method for conical footings in sand, Cassidy and Houlsby (24) have carried out analyses assuming a range of roughness coefficients varying from 0.0 to 1.0. The resulting capacity factors (N_γ) are given in Reference 24. As with spudcans in clay, the resulting factors are somewhat lower than the equivalent flat footing, the effects being more pronounced as the roughness factor reduces.

4.2.3 Backflow

If the void created by the penetrating spudcan collapses so that the soil flows back over the spudcan, the capacity is reduced by the weight of the collapsed soil mass. Various approaches are used to determine the stability of the foundation hole. The usual criterion is to calculate the stability number. Hossein et al (25) suggest limiting the stability number to 6, for hole stability. The stability number as given by Britto and Kusakabe (26) (referenced in SNAME) is as follows:

$$N_s = \gamma H / S_u$$

The volume of soil replaced by the spudcan should be accounted for, so that the net value of backflow reduced by the volume replaced is given as:

$$\text{Backflow} = A\gamma H - V\gamma$$

4.2.4 Other considerations

Predictions of penetration depth may be inaccurate not only due to the analysis method employed, but due also to difficulties in assessing an adequate shear strength for use in the equations. According to Reference 27, anisotropy, yielding and remoulding of clay deposits cause a reduction in S_u. It is suggested (27) to use a reduction factor of 0.85 to account for this. Samples cut at 45deg have only 60% stress ratio, compared with vertical specimens. If soils yield, then strength post yield is lower for stiffer soils. Su may therefore be lower and the equations should reflect this. In addition, due to the preloading which causes plasticity beneath the base, remoulded soils may exist below the base and set up of these soils may not occur sufficiently soon to resist the storm loads.

In clays or mixed soils back analysis of preload penetration depths may allow a more accurate model to be created of the soils conditions and bearing capacity methods employed. Therefore preload penetration and leg loads should be monitored accurately, particularly if there is uncertainty about soil conditions, and the procedure to be halted if the loads are substantially different to those expected. The collected data will allow more accurate modelling of the in-place condition to be made.

4.2.5 Combined loading of spudcans

For combined loading of footings the industry practice has been to use the Brinch-Hansen equations which allow for the interaction of horizontal and vertical loading. Moment loading is incorporated by the use of Meyerhof's effective area concept. This approach is outlined for instance in DNV recommendations for foundations [20]. The Brinch-Hansen equations in Reference [20] are as follows:

$$q_u = N_c S_u (1 + s_{ca} + d_{ca} - i_{ca}) + p'_o \qquad \text{for Clay}$$

$$q_u = 0.5g'BN_\gamma s_\gamma d_\gamma i_\gamma + (p'_o + a)\, N_q\, s_q d_q i_q \quad \text{for Sand}$$

Where $s_{ca,}$, s_γ and s_q represent shape factors; d_{ca}, d_γ and d_q are depth factors and i_{ca}, i_γ and i_q are inclination factors for the effect of horizontal loads.

It is recognized that the Brinch-Hansen interaction equations may be inadequate. For instance the tensile capacity of the base due to suction, or skirts, is not included and coupling between horizontal loading and moment is not allowed for. However, they form the basis of the SNAME Level 2(a) checks.

Experimental programmes have been conducted using both centrifuge testing and gravity testing. A full 3-D yield surface has been proposed by Martin and Houlsby [28 & 29]. The resulting generalised interaction equation is as follows:

$$(M/M_o)^2 + (H/H_o)^2 - 2e(M/M_o)(H/H_o) - \beta^2 (V/V_o)^{2\beta1}(1-V/V_o)^{2\beta2} = 0$$

$$e = e_1 + e_2(V/V_o)(V/V_o-1)$$

$$\beta = \{(\beta_1 + \beta_2)^{\beta1+\beta2}\} / \{(\beta_1)^{\beta1}\,(\beta_2)^{\beta2}\}$$

where e_1, e_2, β_1 and β_2 are coefficients that depend on soil type and loading. V_o, H_o and M_o are the uncoupled vertical, horizontal shear and moment capacities.

This results in a "cigar" shaped surface, which is parabolic in V-H space, V-M space and is elliptical in M-V space.

In SNAME (Step 2 (b)), the interaction equation using the above terminology is defined as follows:

$$(M/M_o)^2 + (H/H_o)^2 - 16\,(V/V_o)^2\,(1-V/V_o)^2 = 0$$

According to Martin and Houlsby, the yield surface expands with penetration depth. Therefore the hardening parameter is defined by the plastic displacement with depth. A flow rule is defined by the usual normality conditions.

Under the usual in-place conditions of combined vertical and horizontal load the preload surface may be exceeded if the preload level is insufficient. This may result in expansion of the yield surface under storm loadings to a depth where equilibrium is achieved.

Skirted spudcans have the advantage of providing tensile capacity, and increased horizontal capacity due to the passive resistance. The skirt tips can transfer the load through to the pile tip where the soils may be stronger and therefore the response can be stiffer in a uniform strength soil or normally increasing strength.

Bransby [30] provides a form of yield surface for skirted foundations, which includes the effect of soil non-homogeneity.

$$(V/V_o)^{2.5} - (1 - H/H_o)^{0.33}.(1\text{-}M^*/M_o) + 0.5\,(M^*/M_o)(H/H_o)^5 = 0$$

The footing is assumed to be founded at the skirt tip level where the shear strength is S_{uo} and the increase in strength with depth is given a linear relationship, kz. M* is the reduced moment given by:

$$M^* = M - LH$$

where L is the height of the scoop mechanism centre of rotation above the footing interface. The uncoupled capacities V_o, H_o and M_o may be determined from figures given in Reference 31.

4.2.6 Load Factors

SNAME adopts a load factor approach to the calculations such that the factored loads including dead loads, live loads and environmental loads are less than the bearing resistance factored by a partial resistance factor. For bearing capacity, load factors vary from 1.0 for dead and live loads and inertia effects to 1.15 for environmental load.

The equations for bearing capacity checks of the leeward leg under pinned conditions are as follows:

$$Q_{VH} = \gamma_1 VH_D + \gamma_2 VH_L + \gamma_3(VH_E + \gamma_4 VH_{Da})$$

where VH_D, VH_L, VH_E and VH_{Da} are vectors of vertical and horizontal leg reactions due to dead load, variable load, environmental load and dynamic inertia load respectively. The required foundation capacity F_{VH} is:

$$F_{VH} = Q_{VH} / \phi_{VH}$$

SNAME recommends a capacity reduction factor (ϕ_{VH}) of 0.9 where the maximum bearing area is not mobilised and 0.85 where the maximum area is mobilised. The use of a larger ϕ_{VH} value will apply to sands where additional capacity is mobilised by a small increase in penetration depth. Therefore the global factor of safety will vary between 1.11 and 1.27 for sands and between 1.17 and 1.35 for clays.

Under full or partial fixity conditions, the leeward and windward legs are checked for the following condition:

$$Q_{VHM} = \gamma_1 VHM_D + \gamma_2 VHM_L + \gamma_3(VHM_E + \gamma_4 VHM_{Da})$$

Here VHM_D, VHM_L, VHM_E and VHM_{Da} are vectors of combined vertical, horizontal and moment leg reactions due to dead load, variable load, environmental load and dynamic inertia load respectively.

To check the foundation capacity the factored vector Q_{VHM} {Q_V, Q_H, Q_M} is compared with the yield locus:

$$16[F_{VHM}/V_{Lo}]^2 .[1-F_{VHM}/V_o].|[1- F_{VHM}/V_o]| – [F_{HM}/H_{Lo}]^2 – [F_M/M_{Lo}] = 0$$

where F_{VHM}, F_{HM} and F_M are capacities under combined loading. In the equation V_{Lo} is the preload capacity, and the other terms are defined as:

$$H_{Lo} = 0.12V_{Lo} \text{ and } M_{Lo} = 0.075V_{Lo}B \qquad \text{for sand}$$

$$H_{Lo} = c_{uo} + (c_{uo} + c_{u1})A_s \text{ and } M_{Lo} = 0.1V_{Lo} B \qquad \text{for clay}$$

It is recognised that embedded footings in clay achieve greater moment and sliding capacity and therefore the yield locus can be modified when $F_{VHM}/V_{Lo} < 0.5$:

$$1 – [F_{HM}/(f_1H_{Lo})]^2 – [F_M/(f_2M_{Lo})] = 0$$

The factors f_1 and f_2 are defined in SNAME. For soft clays and if uplift can be included then f_1 and f_2 are both equal to 1.0.

SNAME does not apply a reduction factor to these fixity bearing capacity calculations.

4.2.7 Site investigation

A detailed soils investigation should be carried out so that accurate soils parameters may be determined for the site. Reference 32 recommends that the following form part of a site investigation:

- Obtain geological information on the proposed site, perhaps by extrapolation from adjacent sites
- Determine bathymetry and identify obstructions from a sonar survey data
- Carry out a high resolution geophysical survey to obtain good quality data for augmenting the geotechnical investigation
- Obtain geotechnical data from shallow coring. Both CPT data and recovered samples should be included. Site geotechnical coring (this is stated in Reference 32 to extend to 100 ft; elsewhere in this report the boring depth is given as 1 to 1.5 times the spudcan diameter plus an estimate of the leg penetration).
- Obtain jack-up rig installation data for area to allow potential problem to be identified.

The cost of an adequate site investigation may be high. However this will be more economic than the cost of repair damage, and may be more efficient than mitigation methods to reduce the possible consequences of bearing failure and excessive settlement, such as operating with minimal air gap or sequential leg loading.

4.2.8 Recommendations to avoid bearing failure and settlement

Good soils data are essential in order to have confidence in the calculations for bearing capacity and estimates of penetration and settlement. If the soils are very soft, the effects of overload due to storm may be severe, leading to increased penetration depth in order to allow the soils yield locus to expand.

Therefore the following measures may be considered to avoid bearing failure:

- Increase the load factor (γ_e) or reduce the bearing capacity strength factor.
- Finite element analysis can provide confidence in the empirical analyses equations and can be used where good soils data are available.
- At the jack-up rig selection stage, spudcans with larger spudcan areas may result in lower bearing pressures and therefore should be considered for the location.
- Skirted spudcans could provide advantages since the skirts can help resist the horizontal loads and may increase capacity by pushing the loads to a lower depth (corresponding to the skirt tips) where stronger soils exist.

Allowances may be required to allow for other effects such as cyclic degradation, particularly in silts or very soft clays.

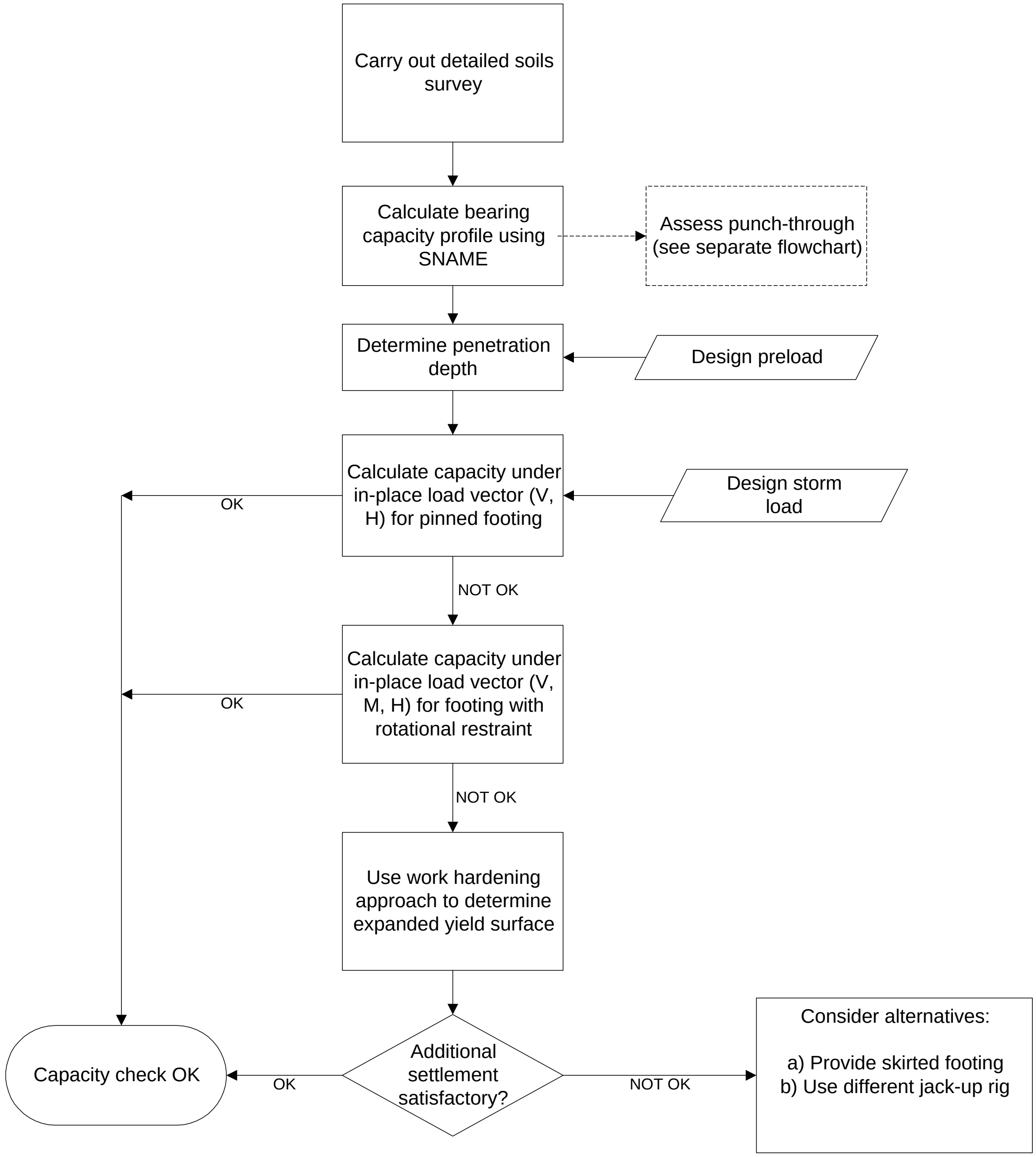


Figure 8 Bearing capacity assessment

4.3 SLIDING FAILURE

4.3.1 Definition

Spudcans may move horizontally if there is insufficient lateral restraint when subjected to horizontal loading, usually from environmental effects in the in-place condition. This sliding can be a first stage in the overall failure of the jack-up, since load is then redistributed to the other legs (leeward) which are then further stressed and pushed towards their yield surface under increased combined loadings. It should be noted that sliding can also occur during installation, particularly into footprints as discussed in Section 4.4.

4.3.2 Sliding resistance

The problem of sliding is most likely to occur in sands, where penetration is minimal. Clays will usually result in significant penetration so that sliding is unlikely. In sands, sliding of the windward leg is often the governing acceptance criterion. According to SNAME, the lowest level check (Step 1) calls for a simple sliding check on the windward leg. This is not given in SNAME but can be determined using an expression for sliding given in DNV (20):

$$H = \alpha V \tan \delta + P_P - P_A$$

Here $P_P - P_A$ represents the net contribution of passive and active pressure resistance against embedded parts of the spudcan. For surface spudcans this term can be neglected. DNV introduces the coefficient α to account for friction at the soil-steel interface.

According to Allersma et al (33) the sliding model recommended in SNAME for Step 2 has been developed on the basis of a flat foundation plate which is sliding over a sand body. The effect of the spudcan geometry and roughness is empirically included in the model by correcting the internal friction angle of the sand. For a steel-sand interface δ is usually taken as $\phi - 5^o$, where ϕ is the internal friction angle. To account for the conservatism introduced due to the conical shape of spudcan, it is recommended in SNAME to use an interface sliding angle equal to the sand friction angle.

In clay, the sliding resistance for a flat plate resting on the seabed is computed as:

$$H = AS_u + P_P - P_A$$

Here it is assumed that the adhesion factor A is 1.0, applied to the base interface shear strength S_u. In clays usually encountered, the spudcan penetration is significant and hence passive resistance and active resistance on embedded surfaces and bearing failure are more significant.

SNAME in Step 2a gives sliding checks for sand and clay which are similar to the above expressions:

Using SNAME terminology, in sands:

$$F_H = F_{VH} \tan \delta + 0.5\, \gamma' \,(k_p - k_a)(h_1 + h_2)\, A_s$$

In this expression, F_{vh} is the Brinch-Hansen axial capacity and h_1 and h_2 represent the embedment depth of the top and bottom of the spudcan. The interface angle δ is used, which can be increased to the soil friction angle for a rough conical spudcan. The second term is the net passive resistance.

In clays, the resistance is:

$$F_H = Ac_{uo} + (c_{uo} + c_{u1})\, A_s$$

In this expression c_{uo} and c_{u1} are the shear strength at the maximum bearing area and the can tip respectively. The second term is the passive resistance, which apparently conservatively neglects any contribution from the active resistance.

4.3.3 Failure mechanisms

Results of tests conducted using centrifuge modelling have been reported by Allersma et al [33]. At low vertical loads the failure mechanism under combined loading is a lateral displacement of the spudcan accompanied by uplift. At high vertical loading, the failure mode combines lateral displacement with further penetration of the spudcan. The two modes are distinguished by a transition point, which is dependent on footing roughness, geometry and penetration depth. Allersma et al comment that the sliding model (see Section 4.3.2) is strictly only applicable for low vertical loads and that it would be more appropriate to use a bearing capacity model, as described by the Brinch-Hansen equations presented in DNV [20]. Hence, on the basis of centrifuge tests, they conclude that the SNAME Step 1 model (simple sliding check) is non-conservative and that the Step 2 models should be implemented.

Hambly [34] reports on a series of model tests which confirm that a partially penetrated spudcan can mobilize greater resistance from sand than is often deduced from the Brinch-Hansen formulae. Test results confirmed that the bearing capacity under a partially penetrated spudcan is considerably greater than the preload and hence the sliding resistance as computed by Brinch-Hansen is an underestimate. When a conical spudcan is used with apex angle 150^{o} the preload should be factored by 1.8 to generate the appropriate sliding resistance. The resistance is increased by passive sliding resistance as well as the increased penetration compared to a flat plate of the same area, used in the formulae. This therefore agrees with the empirical increase in interface fiction angle when using the simple frictional sliding model.

4.3.4 Safety margins

Typical failure envelopes for spudcans on undrained soils and drained soils are shown in Figure 9. Sliding capacity is shown by the sloping lines at the bottom of the drained conditions envelope. Also identified are the load paths for the leeward and windward legs. Loading comprises horizontal loading and vertical loading. Under the leeward spudcan, both vertical and horizontal loading increase during storm loading. For the windward foundation, the vertical load reduces and horizontal load increases with increased environmental conditions. The capacity for sliding is governed by the margin given by the resistance factor ϕ_e in LRFD design or by a safety factor in WSD design. Historically jack-ups have used a safety factor of 1.0 for the vertical leeward reaction, since the preload was restricted to the 50 year storm loading, whilst the windward leg was assessed using a factor of safety of 1.1[35]. This is because the rig would be able to jack-down after the assessment event since its inertia would prevent structural or mechanical overload.

Using SNAME the safety levels are significantly increased. Resistance factors of 0.9 and 0.85 are used for preload bearing capacity and in-place bearing capacity. However, these factors are made more onerous for sliding, being set to 0.8 for sand and 0.64 for clay. For sliding therefore total safety factors of 1.56 for sand and 1.95 for clay are required. According to Reference 35, this departs from historical practice which assumed that safety was determined by limiting displacements.

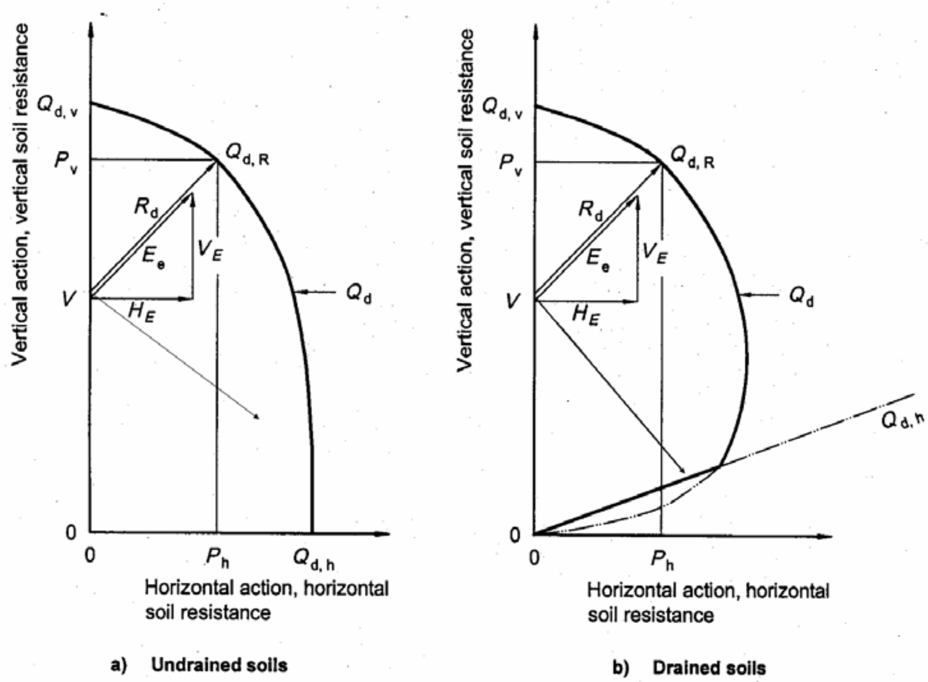


Figure 9 V-H interaction diagrams

In an assessment of failure probabilities, Hoyle et al [36] found that windward leg sliding failure occurs at a load factor greater than suggested by SNAME.

4.3.5 Recommendations to increase sliding resistance

Sliding resistance can be increased by utilising passive resistance in addition to the adhesion or frictional component on the base. Therefore any procedure which increases the spudcan penetration depth or provides skirt resistance will enhance lateral resistance. The methods may include therefore:

- Provision of skirted spudcans
- Use of jetting or other techniques to increase penetration depth
- Increase the vertical loading on the spudcans

Sliding failure of one leg is not an indication of overall failure (i.e. sliding of three legs and movement of the whole jack-up unit) and there is still reserve capacity before overall foundation failure. However the migration of base shear to the leeward legs following sliding of the windward legs, sometimes referred to as the “Hambly failure mode”, might lead at the least to a change of orientation and therefore is unacceptable.

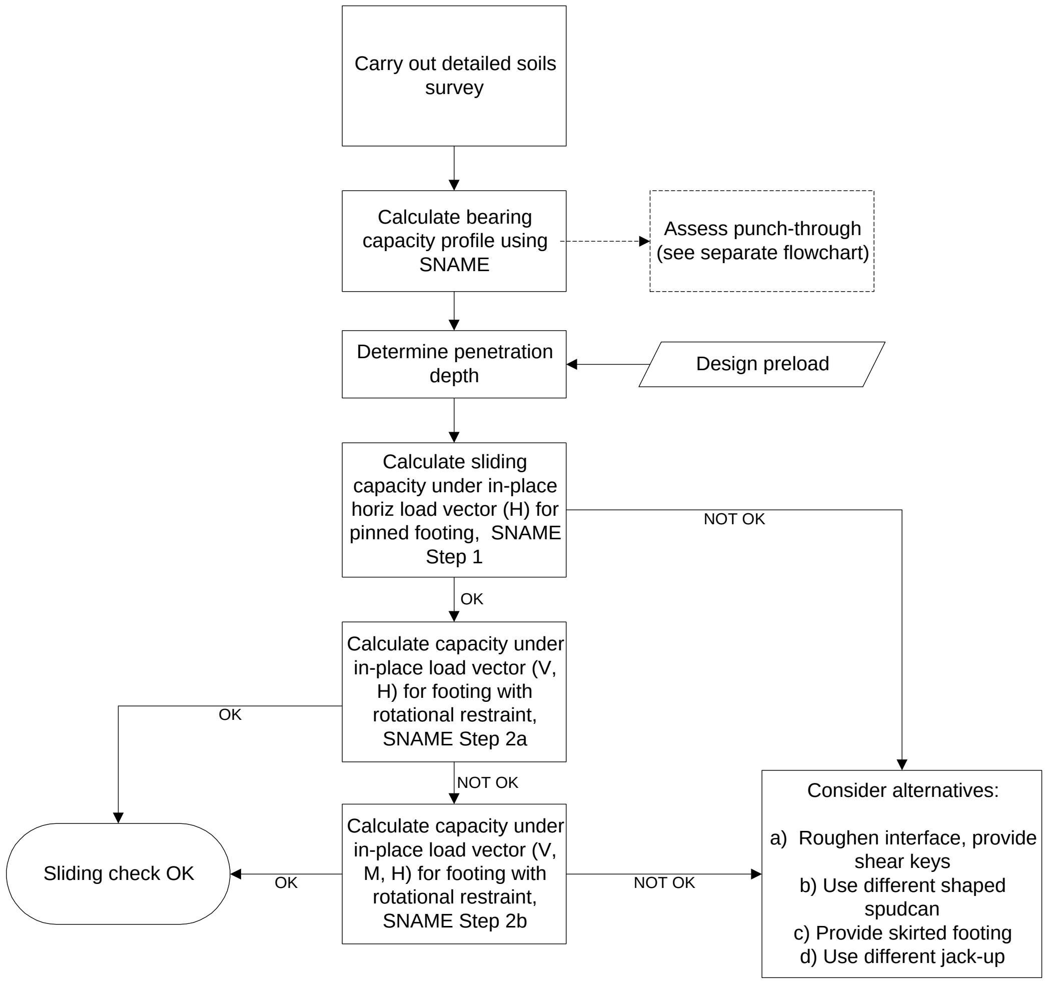


Figure 10 Sliding assessment

4.4 PREVIOUS FOOTPRINTS

4.4.1 Definition

The seabed depressions that remain when a jack-up is removed from a location are referred to as ‘footprints’. The form of these features depends on factors such as the spudcan shape, the soil conditions, the footing penetration that had been achieved and methods of extraction. The shape, and the time period over which the form will exist, will also be affected by the local sedimentary regime [1 & 2].

4.4.2 Footprint effects on foundations

Installing a jack-up very close to footprints or partially overlapping them may cause the following effects:

- Different leg penetrations due to different resistances in the disturbed and virgin soil [5, 37 & 38] and possibly causing damage to the rig.
- Due to different levels in the original and disturbed soil in the footprint area and/or the slope at the footprint perimeter, the spudcan may slide towards the footprint. The resulting leg displacement could cause severe damage to the structure and, at worst, could lead to catastrophic failure.
- This situation could become exacerbated by the jack-up being close to a fixed platform structure [1].
- Lost drilling days due to a need to re-level the jack-up [39].
- Injury to personnel [39].

4.4.3 Prevention of footprint effects

While ISO gives brief recommendations on how to reduce the effects of footprints on foundations, SNAME is more detailed and suggests the following operational sequence:

a) Installation of an identical jack-up design with identical footing geometry to that previously used at a particular location should not cause any problems provided that it is installed in exactly the same position. (Foo et al [40] suggest that this method generally helps the spudcans to make even better penetrations during preloading in comparison to the original installation.)

b) It is unlikely for two jack-ups of different design to have exactly the same footing geometries. Therefore, it will not generally be possible to locate a jack-up exactly in the footprints of a jack-up of different design. However it may be possible to carefully position the jack-up on a new heading, and/or with one footing located over a footprint with the others in virgin soil, to alleviate the potential for spudcan sliding.

c) Where there is no possibility of carrying out either of the above options to avoid the footprint interaction, special attention is required to minimize the potential sliding problem. Consideration may be given to infilling the footprints with imported materials. The material selection should recognize the potential for material removal, by scour, and the differences in material stiffness compared to that of the existing soils. The SNAME Commentary gives a warning of possible soil failure due to a weaker infill material.

Methods for the evaluation and prevention of footprints according to both ISO and SNAME are:

- Evaluate location records (Bathymetric study)
- Prescribed installation procedures (Seabed surface study)
- Consider filling/modification of holes if necessary.

One other method that is recommended in Reference 41 is stomping the footprints. Stomping is explained as a process where the footings are initially emplaced further from the centre of the footprint than the final intended position. This will displace the soil towards the old footprint as illustrated in Figure 11. The authors state that stomping the footprints, or infilling the holes, represent the only two practical options of overcoming the footprint problem.

However, there are three disadvantages to stomping according to the same reference. Firstly, it could involve significant rig-time, which makes it an expensive task. Secondly, it is only practical in mild weather conditions. Thirdly, the ease of work is limited by the clear distance between the fixed platform and the jack-up.

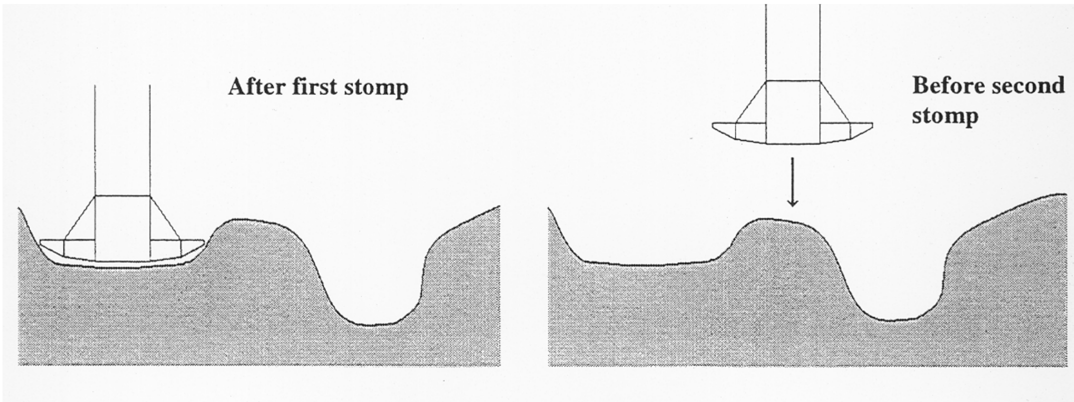


Figure 11 Stomping

Reference 40 introduces a different concept to the problem and suggests that although some of the previously mentioned methods could reduce the probability of the spudcan sliding into the footprint, none of them prevent the leg from bending beyond its limits. So the recommendation of an effective monitoring procedure is stated. According to this reference a good monitoring device to measure Rack Phase Difference (RPD) during the jacking could prevent the leg from bending failure. As long as the RPD limits are not exceeded, rig operators can determine when it is necessary to stop the jacking operation and perform procedures to reduce RPD when necessary. Further discussion of RPD may be found in Section 4.5.

Considering the infilling method, the following points can be made:

1. According to operators' experience, infilling the footprints with materials such as blast furnace slag introduces new problems resulting from a non-uniform pressure distribution on a spudcan resting partly on sand and partly on slag, unless a large area of the seabed is covered with a *level* surface of material.

2. It is also stated by an operator that for shallow footprints, remedial actions may lead to more problems than if no action is taken [38].

3. Finite element analyses have been reported by Jardine et al [41] for a new spudcan installed close to a footprint in cohesive material, but infilled with granular material. The following conclusions were drawn:

    - The overall vertical capacity available to the new installation is significantly downgraded (in this case by about 20%) in comparison with the virgin site.
    - The full capacity can only be mobilized at the cost of sustaining relatively large lateral leg movements, leg forces and leg bending moments.
    - The reduction in capacity is likely to show its maximum value at eccentricity values exceeding B/4 where B is the spudcan diameter.

- Leg lateral forces and bending moments become larger when either the leg is stiffened, or the infill is made softer and weaker.
- Site vessel specific calculations can be made to assess whether infilling provides a viable solution to any particular series of well work-over, or other offshore operations.

4.4.4 Safe distance to avoid footprint effects

SNAME states that it is not possible to advise on a minimum acceptable distance between the proposed spudcan location and existing footprints as this depends on several parameters such as soil conditions, the depth and configuration of the footprint, the degree of soil backfill during and after spudcan removal, the elapsed time since the last installation, the spudcan geometry and foundation loading. As a general rule, SNAME suggests using a minimum distance of one diameter measured at the spudcan bearing area. However, it also warns that this distance varies according to different soil conditions. For example, in soft clay conditions with consequently deep footing penetrations, the situation might be complicated due to the possible larger footprint diameter than the spudcan. Also in dense sand and stiff clay conditions, where shallow footprints are unlikely to influence the integrity of the spudcan foundations, the above guidelines may be conservative.

A series of centrifuge tests [40] were carried out at the Centre for Offshore Foundations Systems (COFS) in The University of Western Australia in 2003. These tests look at rigs installed in different offset instances from the footprint. The experimental results of the tests are illustrated in Figure 12.

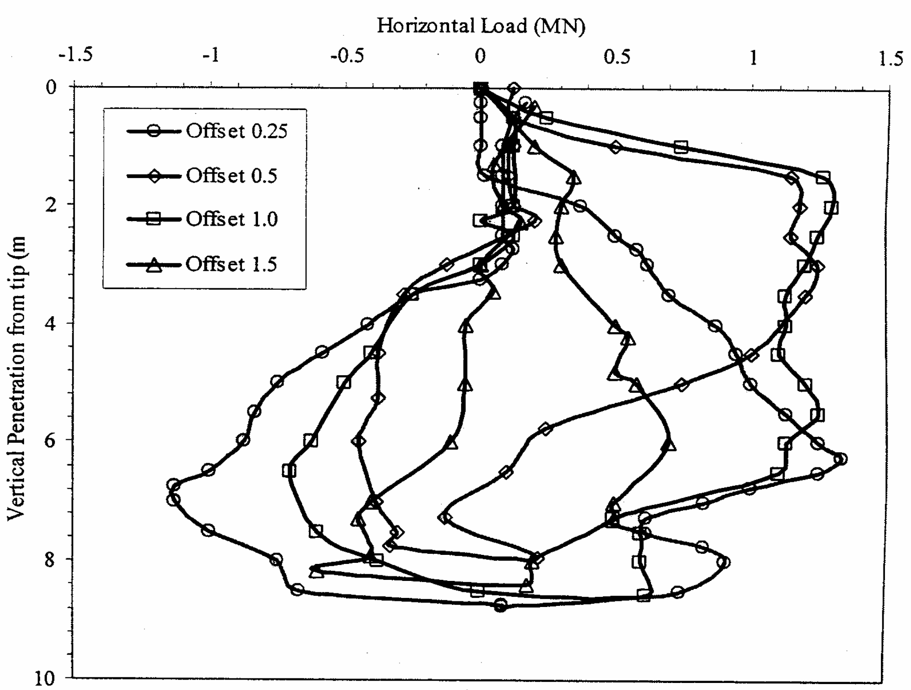


Figure 12 Vertical and horizontal load over depth [40]

The figure shows the horizontal load against penetration, during insertion and withdrawal, for each of four values of eccentricity of the spudcan with respect to the footprint. The plots show that the maximum lateral force was observed at an offset distance of 0.5 or 1.0 times the spudcan diameter. Even though observations suggests that avoiding this peak lateral forces would help in the installation, it is difficult to determine the safe zone in which re-installation can be completed successfully because old footprints may not be of the same diameter and surveying to locate the final spud down position is usually only accurate to a couple of meters [40].

Reference 42 refers to another series of centrifuge tests that were carried out at The University of Western Australia in 2001. The aim of these tests was to assess the loads induced on a spudcan footing, attached to a rigid leg, when penetrating adjacent to an existing footprint. The lateral loads, which may induce significant moment in jack-up legs, were measured. The test results show that the lateral loads are at a maximum when the footing is installed at the distance of ¾ times the footing diameter from the original installation site, as shown in Figure 13.

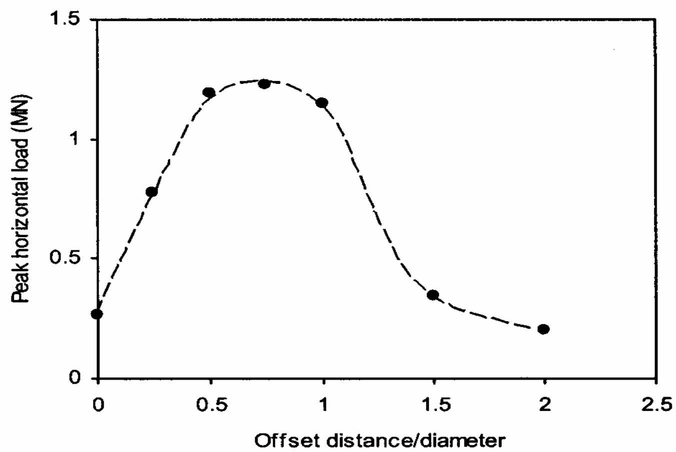


Figure 13 Summary of peak horizontal forces [42]

4.4.5 Recommendations

The following methods for alleviating footprint problems can be considered:

- Install the jack-up in exactly the same footprint locations (SNAME).
- Install the jack-up partly overlapping the footprints, e.g. one leg centralised in footprint and other legs in virgin soil (SNAME).
- Infilling the footprints with similar material to the footprint soil up to the seabed level (SNAME).
- Stomping the footprints as discussed above [41].
- RPD monitoring during jacking to prevent the legs bending beyond their limits [40].
- New installation to be carried out at an appropriate distance from the old footprints.

The general rule within SNAME of using a minimum distance of one spudcan diameter, measured at the spudcan bearing, requires reconsideration as centrifuge tests show that this distance is in the critical range regarding to lateral loads (as indicated in Figure 13).

Global Maritime London and Fugro Ltd. are conducting a Joint Industry Project to investigate problems associated with jack-up installation in locations that other jack-ups previously operated. The work is based on data on past experience of jack-up installations where spudcan footprint interaction has been considered as a potential problem, and on an investigation of current prevention methods (39, 43 & 44). The study aims to produce an optimum set of jack-up installation procedures. This project is due for completion in 2004 and has the potential of being a valuable reference on this matter.

4.5 SIGNIFICANCE OF RACK PHASE DIFFERENCE (RPD)

4.5.1 Definition

Rack Phase Difference (RPD) is simply the difference in elevations between the rack teeth of the chords of any one leg. There are, therefore, as many RPD values as there are leg faces.

RPD may be used as a measure of the inclination of the leg relative to the hull and, in turn, may used to estimate the leg loads (shear and bending). The primary reason for measuring RPD is to ensure that the leg braces are operating within their design envelope, see Figure 14, but the measurements may also be used to determine if intentionally tilting the rig (adjusting horizontal trim) is an option for alleviating RPD.

As explained below, there are a number of loading mechanisms that are manifested as RPD, but not all of these components of RPD induce significant leg brace loads and so are not of concern. For example, guide clearance (gap) and wear of the guide surfaces allow the leg to lean with respect to the hull, producing what some call "free" RPD (45). This free RPD does not give rise to any leg brace loading, nor indeed to leg chord loading.

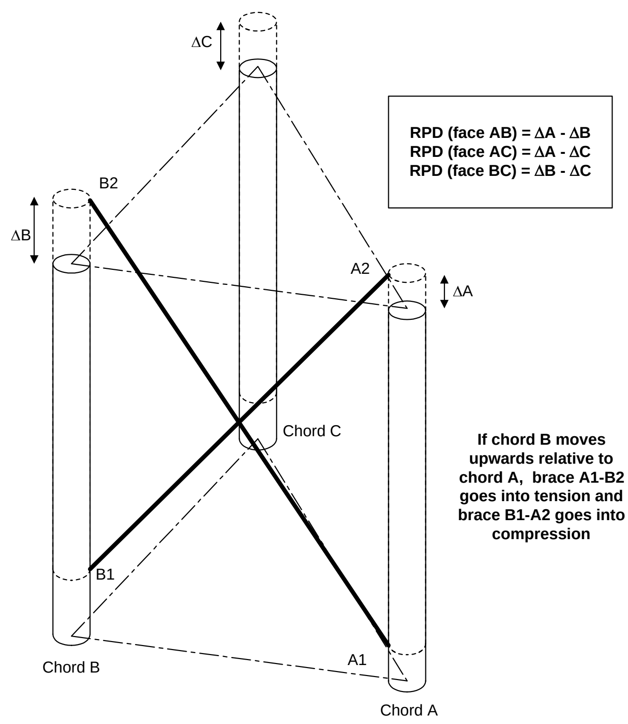


Figure 14 Schematic view of RPD effect (8)

4.5.2 Measurement of RPD and determining plane of leg bending

Traditionally, RPD has been calculated from measurements taken manually. Gauging the vertical offset between corresponding rack teeth on adjacent chords conveniently provides a means for taking suitable measurements. The vertical distance from a reference datum on the hull (typically the top of the jack case or upper guide walkway) to the specified rack tooth position is termed the Rack Phase Value (RPV). The RPV measurement is facilitated by the use of a template that accurately follows the profile of the rack, see Figure 15. Three or four RPVs (depending on the number of chords in the leg) are thus obtained. The RPV measurements are then used to calculate the RPD as explained below.

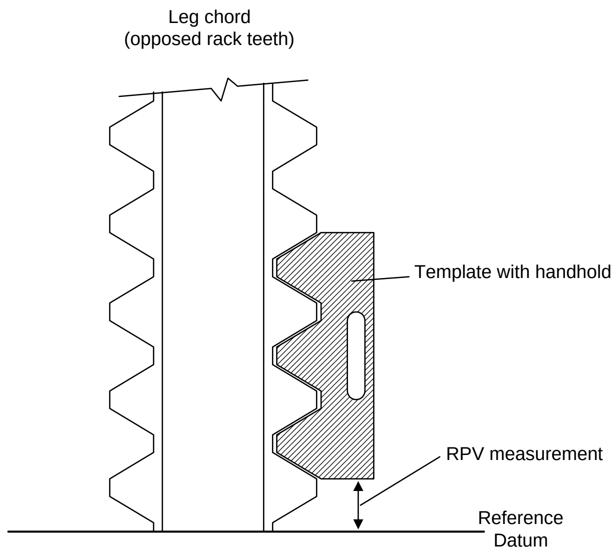


Figure 15 Measuring rack phase value (RPV)

With the growing recognition that monitoring RPD during jacking operations can give an early indication of possible leg overstress, and the difficulties of conducting continuous manual RPV measurements under such conditions, electronic/mechanical systems are beginning to be installed on some jack-ups [40 & 46].

It should be noted that the calculation of RPD from RPV can be carried out in a number of ways, leading to different but related sets of RPD values, and potentially causing some confusion. It is therefore worth exploring this aspect in detail here, so that there is no misunderstanding as to the intent of the practice recommended herein.

The following aspects should be noted at this stage:

- A difference in rack phase between adjacent chords causes the bracing members in that leg face to become stressed, see Figure 14. It follows that RPD should more properly be assigned to a face rather than an individual chord. However, some of the technical literature proposes calculated RPD values that are assigned to individual chords. (For example, some authors in the literature suggest that the RPD for a given chord should be calculated as the difference of that chord's RPV and the largest RPV of all chords of the leg.) This chord assignment is not to be recommended as it may lead to confusion.

- Whatever RPD calculation is adopted, it has to be generally applicable to both 3-chord and 4-chord legs.
- The RPV measurements contain valuable information on the direction of leg bending, and the RPD calculation should also be capable of showing this.

Figure 16 shows example RPV measurements and the derived RPD (face) values for a 3-chord leg and a 4-chord leg. The RPDs have been calculated as the absolute difference between the RPVs of the associated chords. Absolute differences rather than algebraic differences are used to ensure positive RPD values are always returned. It should be noticed that each chord has two RPDs associated with it (e.g. 2” and 1” for chord B in Figure 16), and this is the reason why a single chord assigned RPD may lead to confusion.

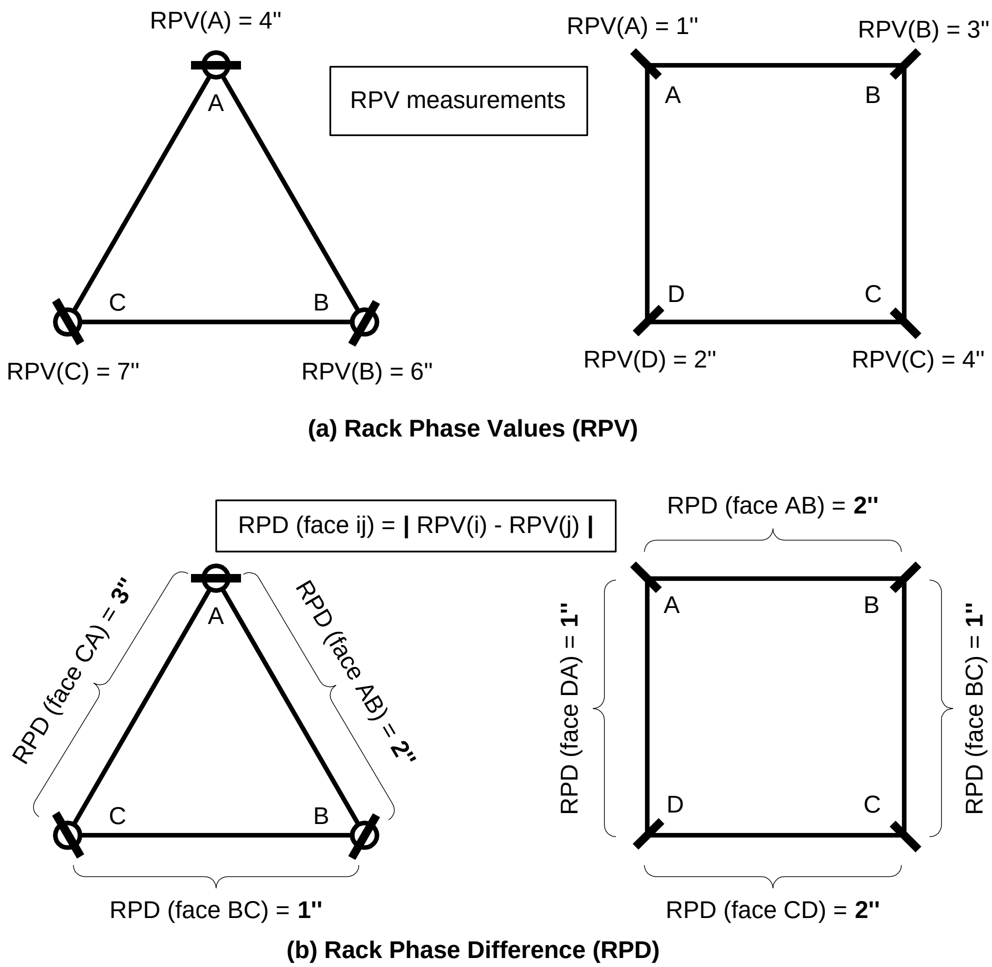


Figure 16 Determination of RPD from measured RPV

During jacking operations, it may be useful to monitor the direction of leg inclination, particularly if the RPD is approaching a safe limiting value and corrective action is contemplated. There are a number of ways to establish the direction of leg inclination, including simple physical observation of where the upper and lower guides are rubbing against the leg, or looking at the tilt of the leg above the jack house. It is recommended that these observational methods are used to confirm leg inclination as calculated from the RPV measurements. Two similar calculation methods are presented here, termed Method A and Method B in Figure 17. Both methods rely on the assumption that the leg is symmetric such that the 'rack phase' of the neutral axis is simply the average of the rack phases of all chords in that leg. The two methods only differ in detail as to how to calculate the rack phases (RP) of the chords. In Method A, the RP of any chord is taken as the measured RPV less the average RPV across all chords in that leg. In effect, this redefines the RPV datum to be zero at the leg centroid and, since the neutral axis passes through the centroid, RP = 0 along the neutral axis. The neutral axis line can then be drawn by finding the 'zero points' on the leg faces by interpolation. In Method B in Figure 17, the RP of the chord is taken as the measured RPV. In this case the neutral axis RP becomes equal to the average RPV of the chords in that leg, and the neutral axis line is found by interpolating for this average RPV on the leg faces. The two methods are equivalent and yield the same neutral axis. Method A is perhaps slightly clearer and is easier for hand computations.

The vector showing the direction of the spudcan is orthogonal to the neutral axis and is directed towards the chord having the largest measured RPV, see Figure 17.

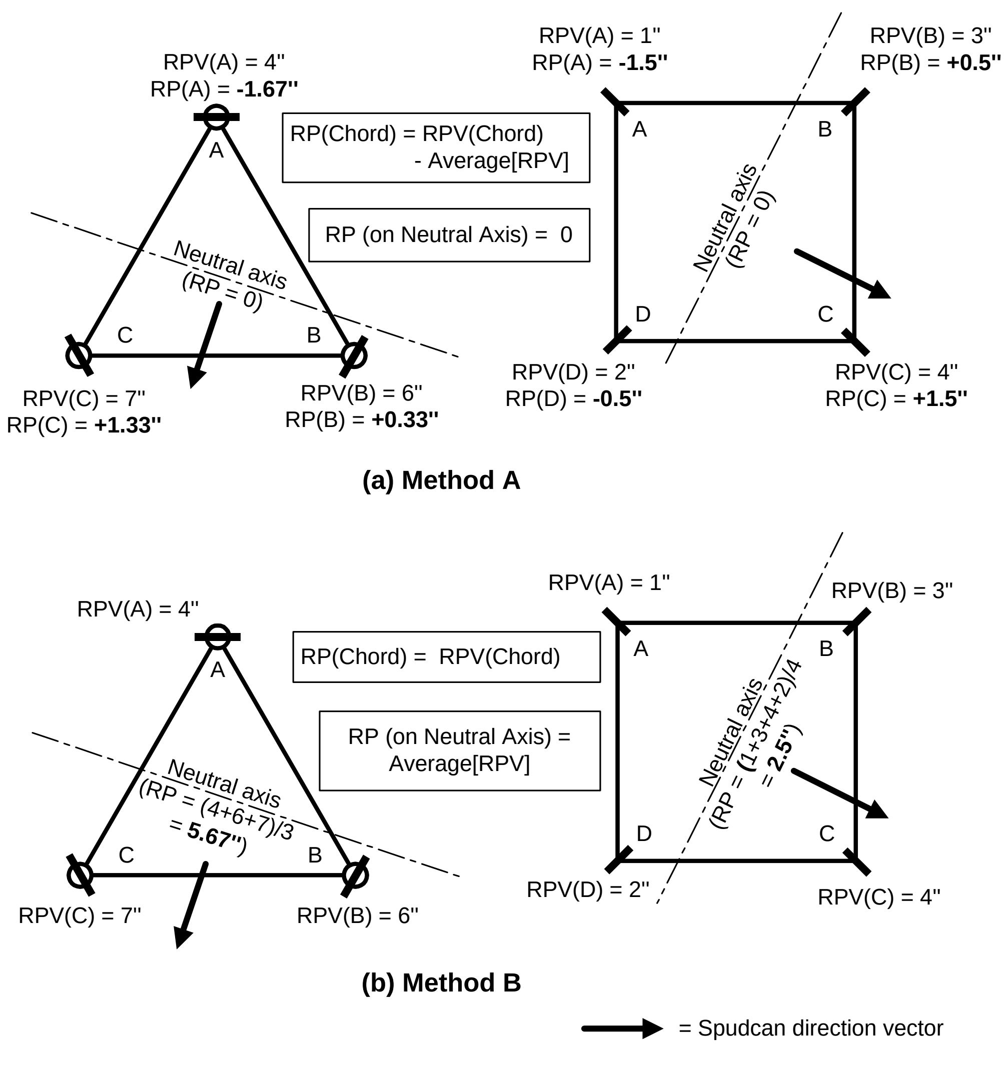


Figure 17 Determining neutral axis and spudcan direction vector

The relative deflection of the leg inclination indicates the direction in which the part of the leg below the hull is pointing or bending. In cases where the inclination of all legs are found to be in approximately the same direction (or the inclination of one or two legs is negligible), RPD can be reduced by lowering that point of the hull which is located in a relative direction opposite to the direction of leg inclination. In Figure 18 for example, lowering the stern will alleviate high RPD. However the most effective method for reducing RPD is to jack on individual chords.

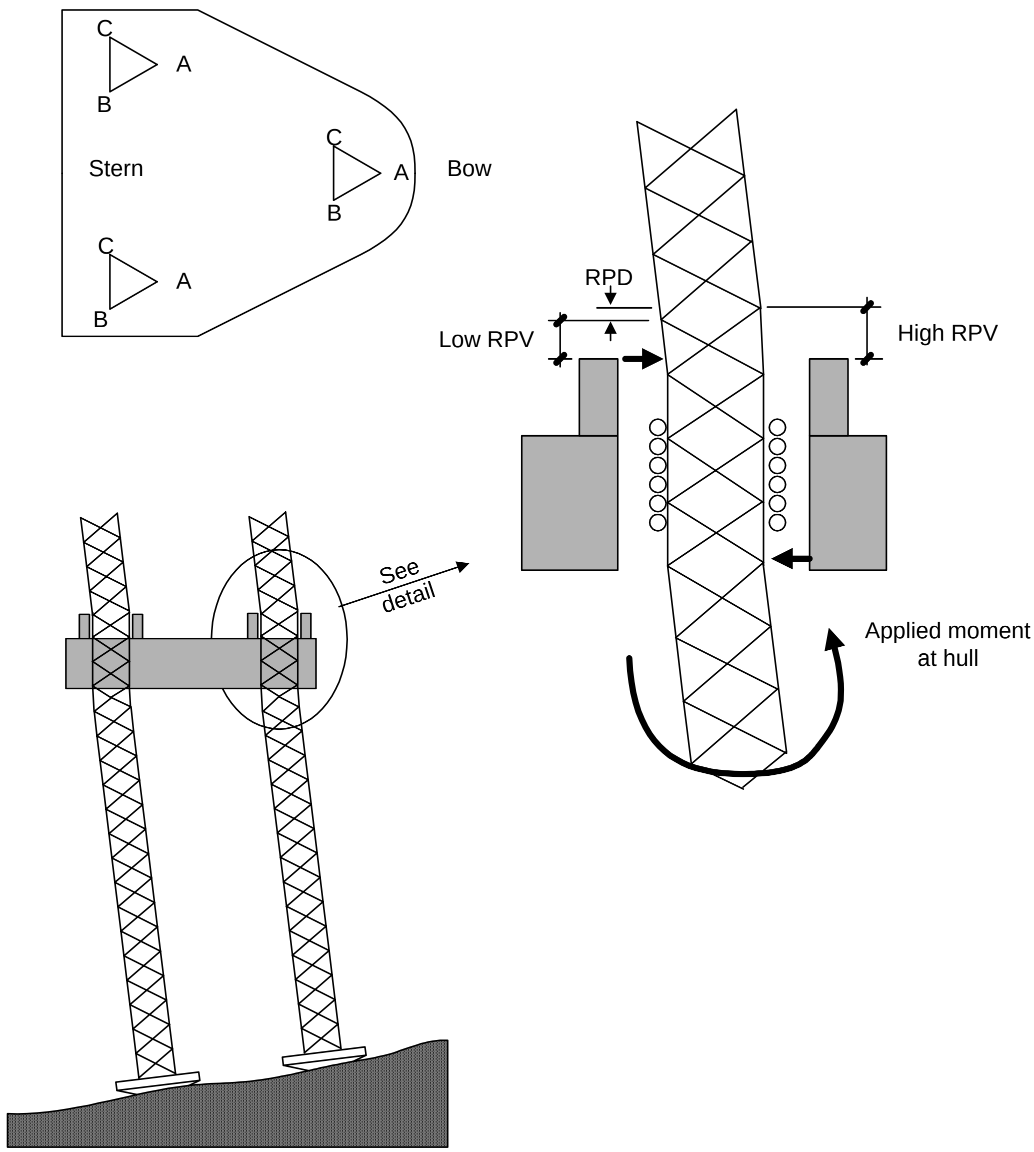


Figure 18 Alleviating RPD by tilting hull

4.5.3 Causes and effects of RPD

The underlying causes of significant RPD are related to moments and shear forces passing through the leg/hull connection, such forces being induced by environmental loads or foundation loads. Shear and moment loads applied to the spudcan are resisted at the leg/hull connection and may give rise to significant RPD. The scenarios that may lead to such spudcan loads include:

- Eccentricity of leg vertical reaction caused by uneven ground conditions, i.e. sloping seabed, previous footprints, and scour. The degree of eccentricity can be affected by hard soil conditions and the shape of the spudcan. According to an operator, during structural analyses and foundation stability it is assumed that the spudcans penetrate into a level seabed and hence the centroid of reaction is the centre of the spudcan. So in areas with sand wave potential, if a leg is placed on the sloping face of a wave, the effect of a non-concentric spudcan reaction on leg moment should be included in the structural analysis of the jack-up. The structural analyses of jack-ups by the same operator have indicated that the adverse moment introduced into the leg by a non-concentric reaction can lead to overstressing of the leg. So they recommend if a non level seabed is identified by the site survey, the jack-up should, if possible, be relocated to avoid a sloping seabed profile [38].

- Sliding of leg (relative to the others), especially into footprints.

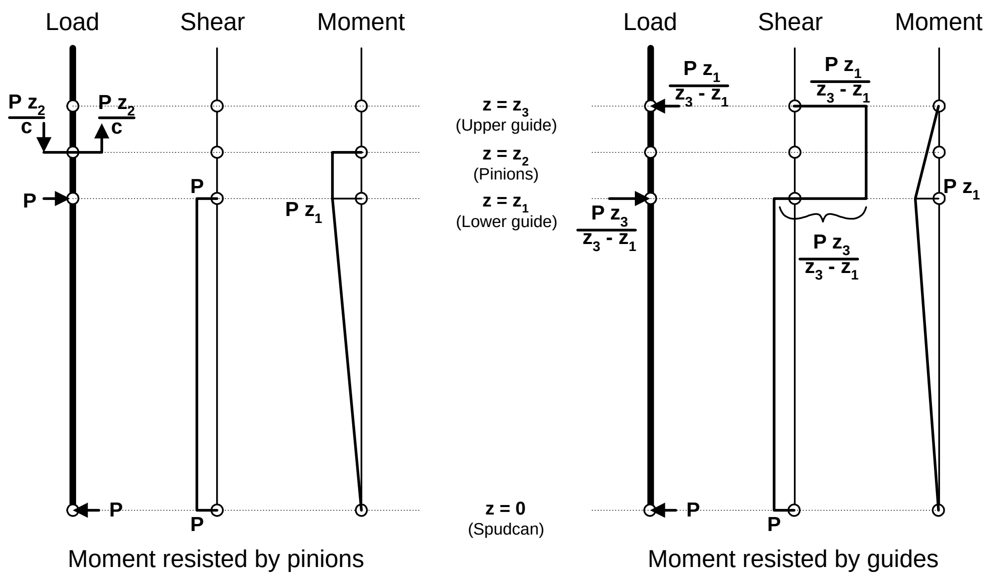
The above scenarios are discussed further in other sections in this document. Here, interest is focused on the effects at the leg/hull connection. There have been several incidents in the last few years where the effects have been manifested as buckling of horizontal and/or diagonal braces in the legs [8, 40, 46 & 47]. The best-documented incident, at least in the public arena, is leg brace failure of a GSF Monarch jack-up operating in the southern North Sea in early 2002. Failure of the braces occurred as the jack-up crew were attempting to re-level the hull, following scour under one spudcan. This incident is reported in Reference 8, although additional relevant information about the jack-up and RPD tolerance can be found in Reference 48.

A useful simplified model that explains the mechanisms by which spudcan loads are reacted at the leg/hull connection is presented in Reference 45. The model is essentially a free body diagram of a single 2D leg, with imposed translation or rotation at the foundation level. Two variants of the model are considered. The first is when the moment at the leg/hull connection is transmitted by a vertical couple due to differential loading exerted by the pinions acting on adjacent chords. The second model variant transmits the moment by a horizontal couple arising from the guide reactions. The resulting shear and bending moment diagrams are given in Figure 19.

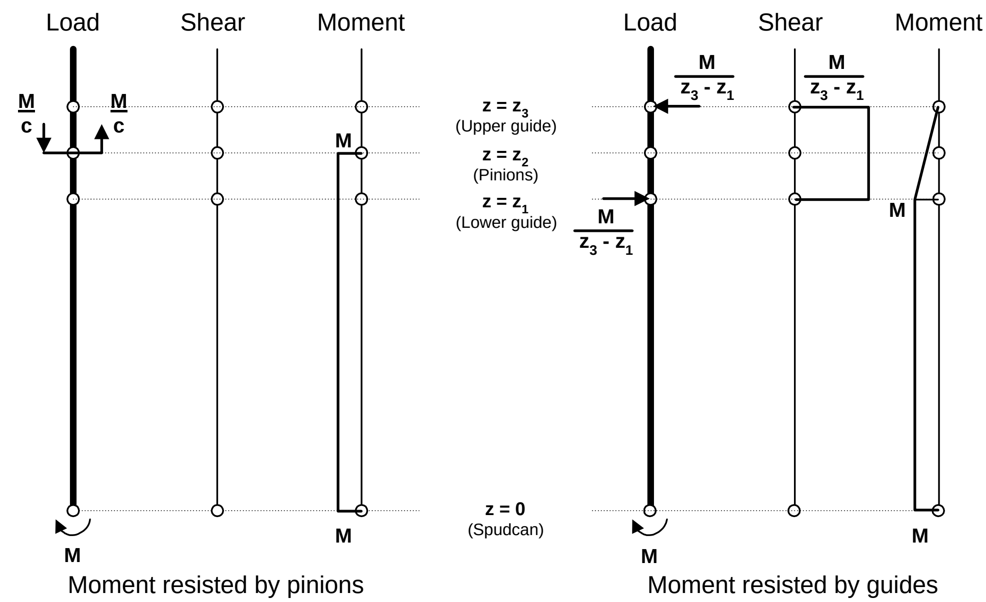
Regarding Figure 19, it is clearly apparent that higher shear loads in the legs, over the distance between the lower and upper guides, arises when the bending moment is resisted by the horizontal couple (guide reactions) rather than by the vertical couple (pinion reactions). Other observations as reported in Reference 45 are:

- The shear and moment diagrams in the leg below the lower guide are identical no matter how the leg moment is transferred to the hull.
- When the moment is transferred by differential pinion load (vertical couple), the shear is zero above the lower guide.

- When the moment is transferred by guide reactions (horizontal couple), the shear direction is reversed at the lower guide. Furthermore, except for very shallow water depth cases, the maximum shear occurs in the region between the lower and upper guides.
- Given typical water depths, chord spacing and guide spacing, the lateral deformation of the leg below the guides is mostly due to flexure, whereas the lateral deformation of the leg within the guides is mostly due to shear.



(a) Force (displacement) applied to spudcan



(b) Moment (rotation) applied to spudcan

Figure 19 Leg shear and bending moment diagrams

Finite element analyses [45] of 2D models of legs having various bracing configurations (X, K and reversed K) confirmed that inter-guide brace loads are relatively low when the leg bending moment is resisted by differential pinion load (vertical couple), but are extremely high when the moment is transferred by guide reactions (horizontal couple). The analyses also demonstrated that legs with K bracing have the largest brace load for the same spudcan loading, and that legs with reverse K bracing have the largest RPD. Leg deformation plots are presented in the paper and these generally show that the leg remains vertical above the upper guide if the moment is carried by differential pinion load, but lean if the moment is carried by guide reactions.

More complex 3D finite element studies exploring RPD effects, some including time domain simulations of the hull jacking process, may be found in the literature [46, 48, 49 & 50]. One important modelling issue, at least as far as obtaining accurate brace loads is concerned, is to consider where guide reactions may arise; there is a fundamental difference between tangential and radial guide arrangements in this respect. However, a particular modelling consideration in the time domain analyses is how the pinion loads are established. Each pinion is driven by one motor. It is important that the torque-rotation characteristics of the driving motors are captured. Electric motors are typically asynchronous, i.e. they tend to slip in relation to one another, as their speeds are a function of the applied torque. It is this feature that allows the RPD to increase during jacking operations because any moment initially carried by differential load in the pinions (vertical couple) is reduced as the motors slip and the pinion loads equalise. The moment is progressively transferred to the horizontal couple and the guide reactions, and the RPD, increase. The reported results in the 3D time domain analyses [46, 49 & 50] are very similar to each other and illustrate:

- The growth of RPD with the transfer of leg/hull bending moment from differential pinion loads (vertical couple) to guide reactions (horizontal couple),
- The comparatively minor role of guide gaps,
- The beneficial effect of foundation fixity,
- That RPD can be modified by selectively jacking on a single chord.

The literature gives the impression that RPD problems have been brought into focus only relatively recently. A clue as to why this might indeed be so is provided in Reference 45. Until about 1980, jack-up designs were based on either fixed jacking systems with radial pinions, or floating jacking systems with opposed pinions. Prior to 1980, the details of construction were such that significant brace loads could be expected under storm loading and other events, and so the braces were accordingly designed to resist these loads. In the early 1980's, rig designs based on opposed pinion fixed jacking systems were introduced, some incorporating chord chock systems for transfer of leg to hull forces once jacking had ceased. This change in design philosophy permitted moments to be carried by vertical couples rather than horizontal couples and led a consequent reduction of brace loads and brace sizes. However, and as discussed above, during jacking up (or jacking down) operations the leg/hull moments may be redistributed from a vertical to a horizontal couple with a consequent increase in brace loads.

As a final comment in this section, it is worth emphasising that RPD and consequent brace loading can occur whenever a moment is being transmitted through the leg/hull connection and the hull is being raised or lowered. Wind and current loading gives rise to moments, although these are unlikely to be severe during jacking operations. Leg sliding into footprints or any eccentricity of the spudcan reaction on a sloping seabed will also impose moments. Note, when a slide occurs in the elevated mode causing the legs to splay, the leg bending moment increases as the hull is jacked down. Even after a jack-up has been successfully installed, subsequent scour can cause difficulties (due to eccentricity of spudcan reaction) when re-levelling or lowering the unit.

4.5.4 Recommendations

Both SNAME and ISO are silent on RPD issues and clearly both would benefit by addressing this topic.

Several recent incidents on the UK Continental Shelf (UKCS) prompted the Offshore Division of the Health and Safety Executive to issue a Safety Notice [47] in August 2002. It may also be mentioned that some Operators issued safety alerts at about the same time, e.g. BP in June 2002 and Shell in October 2002. With respect to the HSE Safety Notice, the required actions by the Duty Holder are: (a) to ensure that on-board operating procedures prescribe appropriate limits to which the installation can be operated, and (b) that suitable monitoring arrangements are in place to ensure that the prescribed limits are not exceeded. These actions are considered to be entirely appropriate. However, the advice that suitable monitoring arrangements may consist of "pinion load monitoring <u>or</u> measurement of RPD" is not sufficient if the former option alone is selected. This is because pinion loads by themselves do not allow the state of stress in the braces to be determined.

A limiting RPD value should be calculated being the maximum acceptable value for use during normal jacking operations. The value should be based on elastic design principles, including factors of safety. Exceeding this limiting value, whilst not necessarily representing a dangerous condition initially, should prompt the jack-up crew to contact the onshore support team for expert advice with respect to continuing jacking operations.

A sufficiently detailed representation of the leg/hull connection should be used in the analysis model for determining acceptable RPD values. Particular care is required in accurately modelling the stiffnesses of guides and pinion systems, and the positions of the active guide restraints. It is not necessary to perform a time domain simulation; although a number of hull positions may need to be considered in the analysis.

Gaps at the guides lead to free RPD and therefore these can be conservatively ignored in the analysis. When they are included, it is suggested that confirmation measurements are made on the actual rig to confirm that the values adopted in the analysis are not too large.

Particular caution should be exercised and frequent RPD measurements taken, at the outset of jacking operations to level or otherwise to adjust the position of the hull of a jack-up that has stood in a region subject to scour.

When the limiting RPD value is approached, the operator may intentionally introduce differential chord loading to reduce RPD. The operator must be aware, however, that this may cause pinion loads to exceed the jacking system rating. If so, it may be possible to reduce pinion loads by decreasing or repositioning the vertical dead load [45].

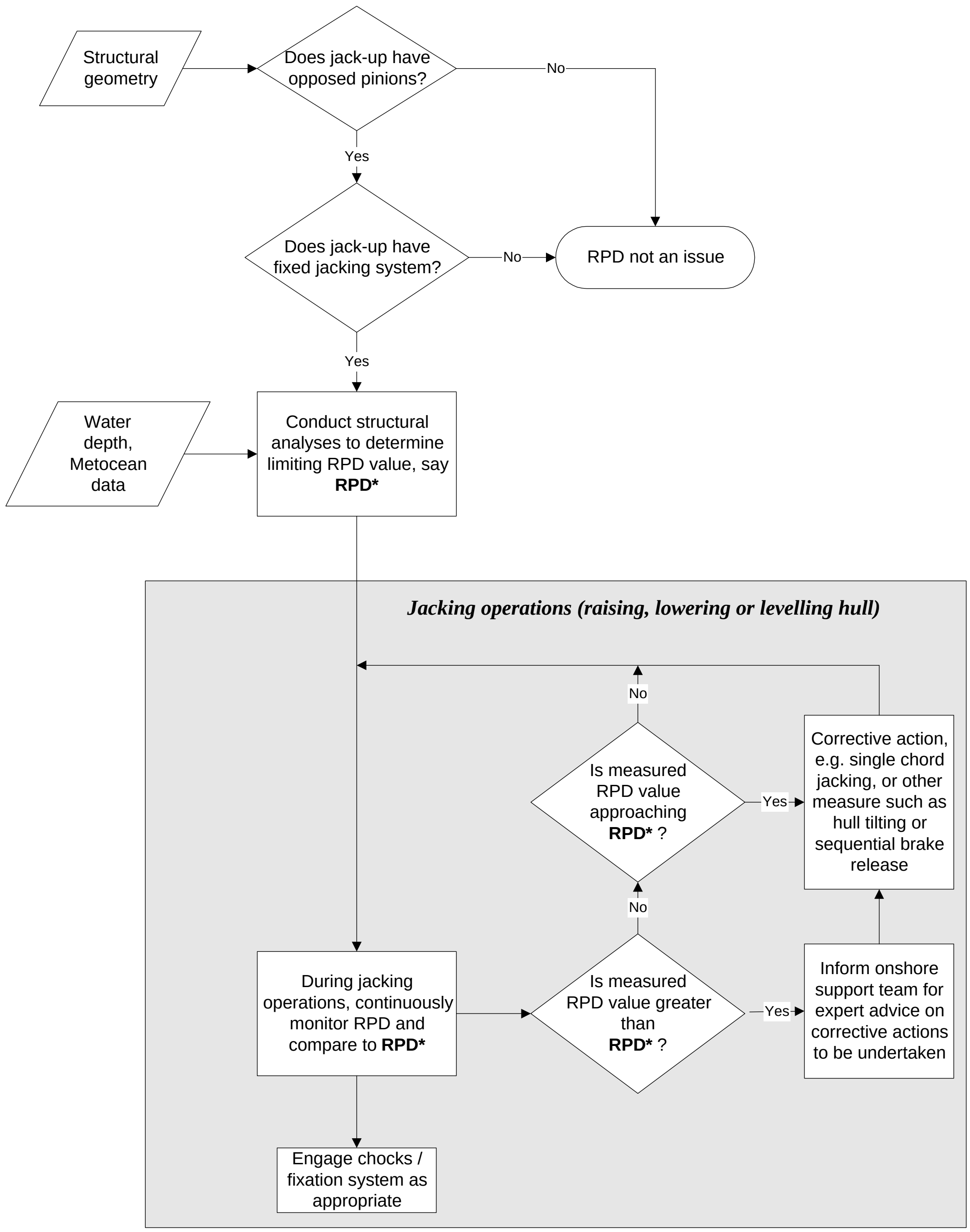


Figure 20 Rack phase difference (RPD) assessment

4.6 SCOUR

4.6.1 Definition

Scour is the removal of seabed soils by currents and waves. This can be due to a natural geological process or can be caused by structural components interrupting the natural flow regime above the sea floor [(4)].

Scour is differentiated into three categories in OTO 93 024 [(51)]:

- Overall scour which would occur even if no platform were there.
- Global scour that represents a general scouring caused by the water flow through the base of the platform.
- Local scour that represents the local cone of depression formed due to the increased local flow around on obstruction such as a pile [or spudcan].

4.6.2 Scour potential

The SNAME Guidelines [(1)] state that scour happens when the shear stresses induced by fluid flow exceed a certain value and/or when the turbulent intensity of the flow is sufficiently large to lift individual grains and entrain these in the flow. But scour phenomenon occurs around spudcans that are embedded to a shallow level in granular materials at locations with high current velocities.

Neither SNAME nor ISO present any definitive procedure to evaluate and assess scour potential. However, previous operational experience should be studied and be taken into account in foundation design. Further guidance is given in the commentary to SNAME and some of important parameters in the assessment of scour potential are are:

- Seabed material: size, shape, density and cohesion
- Flow conditions: current velocity, wave-induced oscillatory velocities and interaction of waves and currents
- Shape, size and penetration of jack-up footing.

The effect of shape parameters is also mentioned in Reference 37, and a warning is given about using cone-shaped footings in areas that are susceptible to scour. Lyons and Willson [(52)] recommend that where scour is a potential problem, a spudcan with a relatively flat profile from the spudcan 'tip' to the 'knuckle' may be more capable of minimizing the effects.

OTO 93 024 [(51)] mentions that the degree of scour depends on the particle size of the bed material, the shape of the obstruction and the stream velocity away from the obstruction. Scour is more likely to happen in shallow waters, i.e. less than about 30 meters.

SNAME emphasizes on the importance of site evaluation for identifying potential scour problems in the Guidelines. The Commentary (section C6.4.3) states that methods are available to determine whether a significant scour is likely to take place. However, no detail about these methods is provided beyond providing a reference [US NCEL, Marine Geotechnical Engineering Handbook].

Both SNAME and ISO recommend that the following data be gathered to assess the potential for scouring:

- An appropriate bathymetric survey should be carried out for an area of 1 Kilometre square centred on the location, to identify sand waves.
- Site investigation, including the characterisation of the surface soil samples.
- Seabed currents.
- Footing and seabed profile to be inspected regularly.

However, it should be noted that the above data could become out-of-date so easily, especially in areas with drilling and construction activity [(4)].

4.6.3 Scour effects on foundations

Both SNAME and ISO state that scour may:

- Partially remove the soil from below the footings and result in the reduction of bearing capacity and any seabed fixity.
- Cause a rapid movement of the leg downward during the storm and affect the foundation severely, especially when there is a potential of punch-through at the location.

ISO also adds that scour may affect the lateral position of the reaction point (i.e. causes an eccentric loading effect).

References 38 and 52 warn about one further effect to the above-mentioned points. Scour around a jack-up foundation can lead to settlement of the jack-up footing and this, in turn, can cause distress to the jacket pile as well as to the jack-up foundation itself.

In one real instance that occurred off Sable Island (Canada) in 1977 [(5)], insufficient initial penetration and a scour exceeding 3 m caused major problems. Odeco's Gulftide has three triangular spudcans 6.7 m long on each side. The expected penetration under preloading was 3.1m, but actual penetration achieved was only 1.4m. Storm-induced scour caused an uneven settlement of the footings and necessitated re-levelling the legs, which interrupted the drilling activity. All the efforts to stabilize the seafloor with anti-scour mats were unsuccessful. In one rig position, after the use of anti-scour mats had been abandoned, a series of four storms caused cumulative settlement ranging from 3 to 3.75m. Because of this unsatisfactory performance, the rig was dry-docked and the footings were equipped with an airlift jet system. At the next location, the jetting method enabled the footings to penetrate the soil to a depth of 5.8 to 6.4m, where the footings were found to be no longer susceptible to scour-induced settlement. The addition of skirts around the periphery of the footing also proved helpful.

In 1990 Kolskaya (Norwegian jack-up) experienced scour around one of its legs [(7)].

In January 2002, in the southern North Sea, two legs of a Mod V Monarch jack-up were damaged from a combination of uneven sea-bed and scour [(8 & 9)].

4.6.4 Scour mitigation methods

Once investigations have been carried out and appropriate studies have predicted scour as being a potential problem, the following measures could be implemented [1]:

- Gravel dumping prior to jack-up installation
- Installation of artificial seaweed
- Use of stone/gravel dumping, gravel bags or grout mattresses after jack-up installation.

ISO recommendations are exactly the same as those of SNAME (above), along with monitoring and adjusting for loss of air gap. This suggestion of monitoring air gap does not appear to be very effective either to prevent the problem happening or reduce its further development if it has already begun. Monitoring of the actual condition of the spudcan and seabed is a better option if applied regularly [38 & 52] and following a storm [52]. This could be carried out by ROVs or divers and can capture a potential problem at an early stage. Some operators practice this method in their current jack-up operations and foundation assessments. In this procedure they record the penetration of each spudcan at the location by divers at regular intervals, and then infer whether scour is happening.

Note that in at least one problematic incident [8], the placement of frond mats has hampered the ability of subsequent ROV surveys to monitor the actual soil/spudcan interface.

If scour is detected the two principal corrective methods for mitigation are:

- Placement of a scour resistant material on the seabed, e.g. blast furnace slag or sandbags.
- Reduce current velocity close to the seabed to prevent removal of material in suspension, e.g. use hanging curtains or artificial reeds.

Consideration may also be given to the removal of the spudcan and, where applicable, the clearing of the seabed [38].

Offshore Technology Report OTO 01 014 [37] mentions about covering the area that scour may occur in, with suitable protective materials to prevent loss of the underlying soils and advises regular inspection of the spudcan to assess the degree of scour.

References 1, 4 and 51 all recommend dumping gravel, rock or geotextiles onto the site to prevent further scour damage.

It should be noted that there is a trade-off to be made between rocks/boulders that are good for scour protection but bad for spudcans, and gravel which is not so effective at scour protection but less likely to damage spudcans. Subsequent rig positioning operations may be jeopardised by rock dumping activities.

4.6.5 Recommendations

All discussed prevention methods are summarized below:

1. Using the most up-to-date geotechnical data during the design and installation

2. Dumping gravel, rock, or sandbags prior/after installation (providing no damage to the footing is caused)

3. Installation of artificial seaweed and reeds to reduce the current velocity

4. Monitoring the actual condition of the spudcan regularly

5. The validity of the preloading operation should be reappraised if significant scour (say more than 1m) is encountered.

6. Deeper spudcan initial penetration using jetting method if needed

7. Using spudcan with relatively flat profile in susceptible area

8. Removing the spudcan and cleaning the seabed if necessary.

It is noted that specific provisions are given in SNAME and ISO with respect to items 1, 2, 3 and 4. Both SNAME and ISO would benefit by including all items.

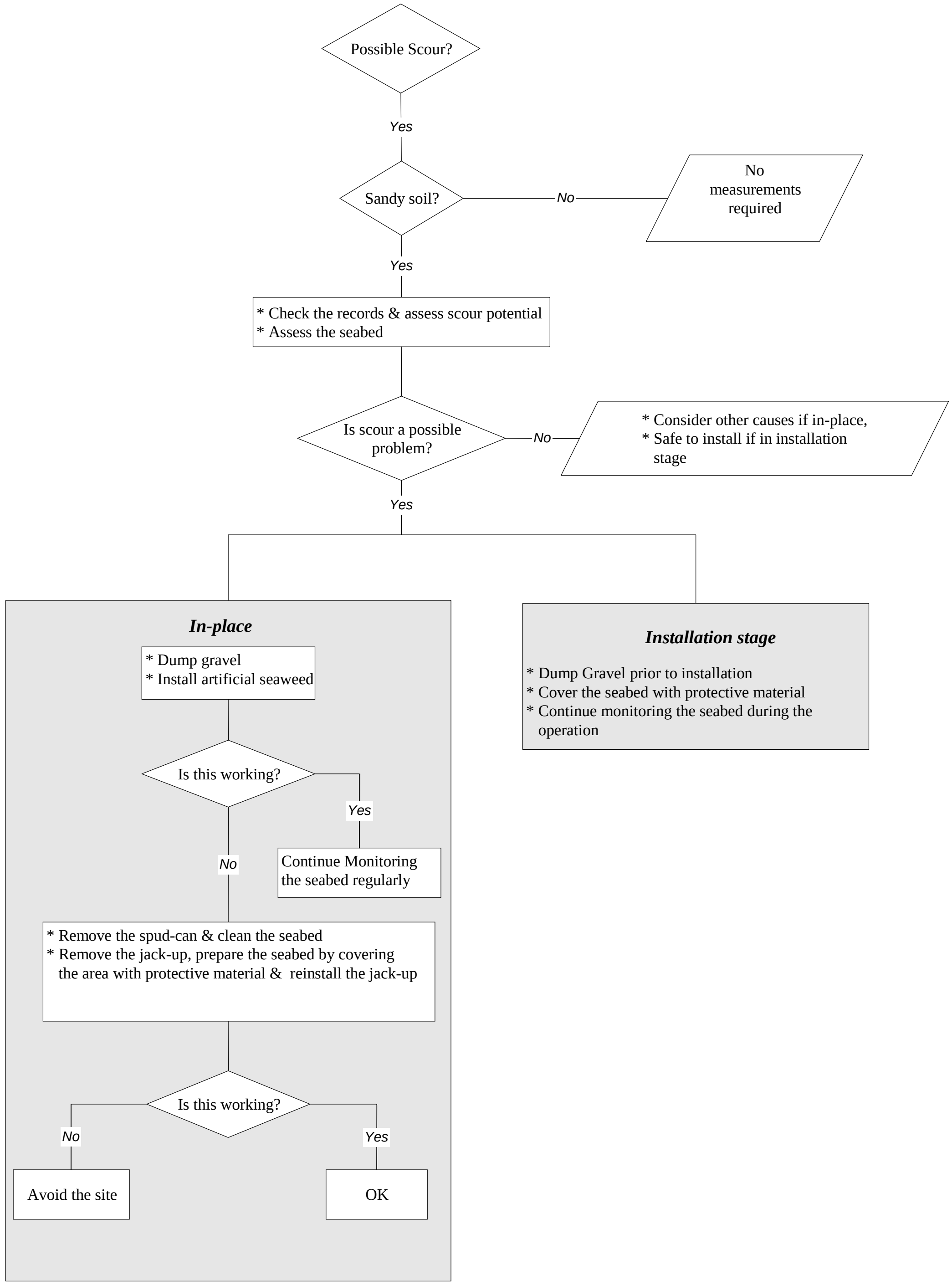


Figure 21 Scour assessment

4.7 LAYERED SOILS

4.7.1 Definition

The surface soils in which the spudcans are founded may include stratified soils in which clays are interlayered with sands. Usually the most significant conditions for spudcan foundations is the case of hard clay or dense sand overlying soft clays, or the incidence of a sandwich of sand between upper and lower strata of soft clays. Different geological processes may have been responsible for these deposits, for instance clay at depth may have been frozen, with the result that the consolidation process did not develop. Overlying sand may be the result of marine or glacial deposition processes. These are significant for the spudcan foundation, since a punch-through could develop in passing from stronger sand into a weaker clay layer, and many cases of punch-through have occurred in such layered conditions.

Layered soils are particularly significant since this condition may lead to punch-through, which is one of the prime causes of jack-up rig distress and personnel injury (see Section 3.3).

4.7.2 Design approaches

Methods for determining the capacity of layered profiles include:

- the squeezing solution, due to Meyerhof (53),

- the punch-through solutions for strong clay overlying a weak clay, due to Brown and Meyerhof (54), or for a dense sand overlying a soft clay, as proposed by Hanna and Meyerhof (55), and

- various load spreading methods.

Only the first two methods are described in the SNAME and ISO documents.

Squeezing

The equation for squeezing, which is appropriate for a thin soft clay layer overlying a strong layer, is:

$$F_v = A(5.0 + 0.33B/T + 1.2D/B)S_u + p'_o \quad \text{for no backflow}$$

and

$$F_v = A(5.0 + 0.33B/T + 1.2D/B)S_u + V\gamma' \quad \text{with backflow}$$

The limiting value of the strength of the soft layer is the Skempton capacity. D is the depth of bearing surface, B is effective diameter, T is thickness of soft layer and V is the volume of the backflow material.

Strong clay over weak layer

The strength at the surface of the strong layer is:

$$F_v = A\,(3\,H/B\,S_{ut} + N_c\,s_c\,S_{ub}) <= A\,N_c\,s_c\,S_{ut}$$

Punch-through checks are as follows:

$$F_v = A\,(3\,H/B\,S_{ut} + N_c s_c\,(1 + 0.2\,(D + H)/B\,S_{ub} + p'_o) \quad \text{for no backflow}$$

and

$$F_v = A\,(3\,H/B\,S_{ut} + N_c s_c\,(1 + 0.2\,(D + H)/B\,S_{ub})) + \gamma' V \quad \text{with backflow}$$

S_{ut} and S_{ub} are the strength of the upper and lower clays; s_c and d_c are shape and depth factors.

Sand overlying clay

The capacity of a spudcan on sand overlying clay is determined as follows, for no backflow:

$$F_v = F_{v,b} - AH\,\gamma' + 2H/B\,A(H\gamma' + 2p'_o)K_s \tan(\phi)$$

$F_{v,b}$ is the capacity of the lower clay. If backflow occurs, as shown in the right hand diagram of Figure 22, the capacity is reduced by the weight of backflow soil. Values for K_s, the coefficient of lateral pressure, are given in the Hanna and Meyerhof [55] paper. According to SNAME it can be approximated by the following lower bound expression:

$$K_s \tan(\phi) = 3C_u/(B\gamma')$$

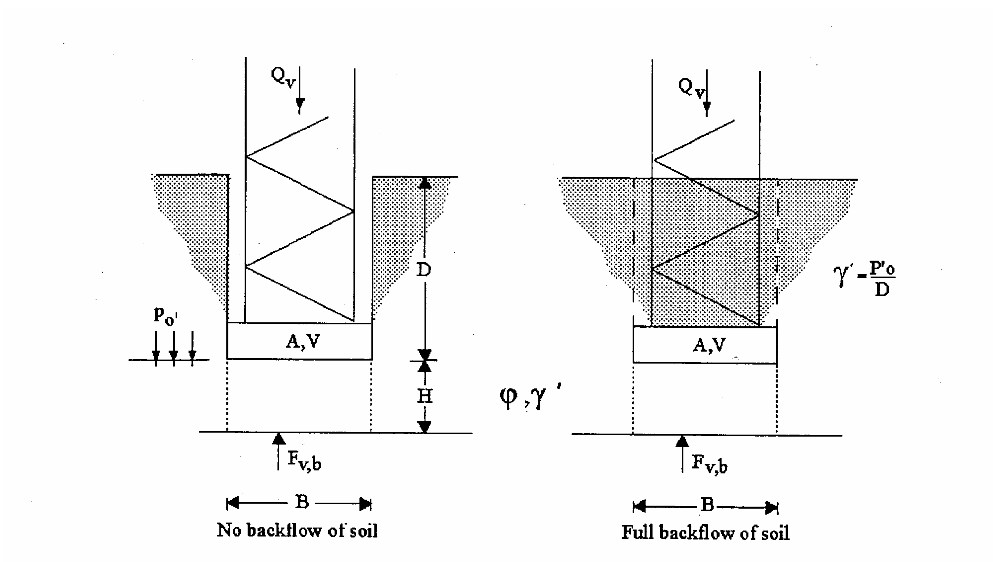


Figure 22 Spudcan penetration geometry

Load spreading methods

As well as the above approaches to the capacity of layered soils with a stiff stratum overlying a soft clay layer, load spreading methods have also been proposed. These methods assume that the load is spread through the stronger over-layer at a spreading "angle" so that the softer layer is subjected to an equivalent reduced bearing pressure. Various methods are suggested for the estimation of this spread. Baglioni, Chow and Endley [56] suggest that the angle of spread should be the friction angle, ϕ, of the stronger over-layer. Other suggestions are to use a 3:1 spread, as described by Young et al [6] whilst Jacobsen et al [57] use an empirical relationship to determine the dispersion angle. Baglioni et al [56] carried out a study of case histories and concluded that the Hanna and Meyerhof method [55] was conservative for thick and dense upper layers, with the load spreading method providing better predictions. It was also argued that thin layers of weak clay or silts in the sands may cause local shear failure. Craig and Higham [58] carried out centrifuge testing and concluded that the behaviour of a foundation during punching shear failure is governed more by the transmission of the applied load through a failure zone in the upper layer to a fictitious interface bearing area rather than by the consideration of punching shear in the upper layer. These authors also note that if the upper layer is weaker than the lower stratum, lateral squeezing may result, as identified in SNAME.

Multi-layered systems

According to ISO and SNAME where the spudcan rests on three layers, the two lower layers are treated as one layer and the bearing capacity assessed based on squeezing or punch-through as appropriate. These two layers are then treated as the bottom of a two layer system with the first upper layer. It would appear that this methodology could, in principle, be extended to systems with even more layers. However, in such cases no guidance is given and recourse to finite element analysis should be considered.

Finite element analysis

Finite element analysis can be carried out to augment the classical bearing capacity solutions and may be useful particularly when multi-layered soils are present. Generally, finite element analysis, as shown by Kellezi and Stromann [59], yields larger capacities than conventional explicit formulae based solutions.

4.7.3 Recommendations to mitigate effects of layered soils

If variable or layered soils are expected at the jack-up site, detailed soils design parameters are required to formulate the bearing capacity for layered soils. The methods for design shown in SNAME should be used to predict the bearing capacity profile. Finite element models could be used to confirm bearing capacities if the soils are very variable. Plasticity models, for example the Martin and Houlsby plasticity models [29], are useful in predicting the capacity in homogeneous soils. As with other aspects of site conditions, it may be necessary to consider the use of an alternative jack-up, with different spudcans which could reduce the average bearing pressure. Skirted spudcans will provide additional resistance to horizontal load which will help improve the interaction response between horizontal and vertical loading.

4.8 FOUNDATION FIXITY

4.8.1 Definition

Foundation fixity of a jack-up normally refers to the rotational stiffness of the footings. The fixity will range from zero for pinned conditions to fully fixed. Foundations, of course, also have translational components of stiffness that may be modelled in conjunction with rotational stiffness.

Assessment of fixity is significant for jack-up structural and foundation design during the following phases:

- Under fatigue seastates
- Under developing storm conditions
- At foundation yield conditions.

For structural fatigue analysis and when subject to storm conditions which are less than the design assessment condition, fixity conditions will significantly affect dynamic response. For instance Osborne et al [60] state that the range of fixity conditions may result in a change in natural period by a factor of 2.0. The guide moment will change by a similar magnitude.

Fixity conditions at yield will influence the structural analysis and therefore the design of foundations, and distribution of forces to the spudcans. For instance, fixity gives rise to foundation moments, and therefore the governing equations (Step 2b), with moment included, are more appropriate than pinned footing conditions. Therefore fixity may result in a less conservative loading regime for the footing than the assumption of a pinned condition.

4.8.2 Existing guidance

SNAME provides a method for assessing foundation fixity. As a starting point, elastic stiffnesses are calculated using Boussinesq formulae. The rotation stiffness is given as:

$$K_3 = G_r\, B^3/3(1-\nu)$$

For clays the shear modulus G_r is given as $I_r\, S_u$, where I_r varies between 50 and 200 depending on the degree of consolidation. The modulus for sand is recommended to be assumed as:

$$G_r = 600\,(V_{Lo}/A)^{0.55}$$

where V_{Lo} is the maximum applied preload and A is the maximum bearing area.

If the load vector lies within the yield surface, SNAME recommends that the initial stiffness should be reduced by a factor f_r given by:

$$f_r = (1-r_f)0.5 + 0.1e^{100(rf\ -1)}$$

where r_f is a failure ratio which depends on the load vector position within the yield surface, and equals 1.0 when the loads are on the yield surface. Applying the equation for this yield condition, results in a reduction factor of 0.1.

When the load vector is outside the yield surface, the factor is not applicable.

4.8.3 Comparison with recent findings

In 2003 the initial findings of a study for the International Association of Drilling Contractors (IADC) were reported [61]. The study considered three broad areas: initial stiffness, yield and cyclic stiffness. Initial stiffness is affected by assumptions about soil moduli which are load dependent and therefore vary both during the loading cycle and across the location of the foundation. The study noted that SINTEF recommended improvements in shear modulus assessments, by up to a factor of 5.0, and that Noble Denton recommended an increase in rotational spring stiffness values by a factor of 2.0 for deep burial. At yield the rotation is determined by a factor, which is dependent on the vertical capacity, which is proof loaded by the preloading process.

SINTEF recommended the adoption of a single formula for clay and sand. However the study also notes that the yield function is unduly conservative particularly at high vertical loads. For small rotations, cyclic loading increases stiffness as recognised by Osborne et al [62].

Increased stiffness also results when cyclic loading is combined with sustained loads. Field measurements made during storm loading in the Gulf of Mexico confirmed fixity levels in soft clays well above SNAME calculated values. The reported findings indicated that degradation of spudcan fixity is not as significant as suggested by present design guidance. Parallel numerical analyses for buried spudcans in clay suggest that moment capacities are about 75% greater than SNAME values. Instrumentation of Adriatic III during a tropical storm identified fixity levels of 90%, and suggested an increase in the soil-modulus level from 200 as given in SNAME to about 600.

Hambly et al [63] measured jack-up performance in a maximum seastate corresponding to a maximum wave height of about 14.9m, slightly smaller than the annual return period storm. They concluded that the spudcan fixity was related to the elasto-plastic theory for homogeneous clays. Additionally cyclic loading during storms caused relaxation ("shakedown"). The foundations behaved as pinned during the average static component of load and were fixed elasto-plastically for the dynamic component of load. The combination of the applied moment and vertical load resulted in yielding of the foundation and increased penetration.

In calibration of a jack-up on sand in the Silver Pit location in the Southern North Sea [64], measurements were made of spudcan fixity under seastates up to a maximum wave height of 20.3ft (6.18m). The results confirmed the fixity did not reduce between wave heights of 5.6ft (1.67m) and the maximum wave height of 6.18m.

Centrifuge testing was carried out for a number of North Sea jack-up rigs and is summarised in HSE Research Report 037 [65]. This confirms the approach of SNAME is conservative. At load levels up to and beyond the 50 year assessment storms, significant stiffness remains as shown in Figure 23.

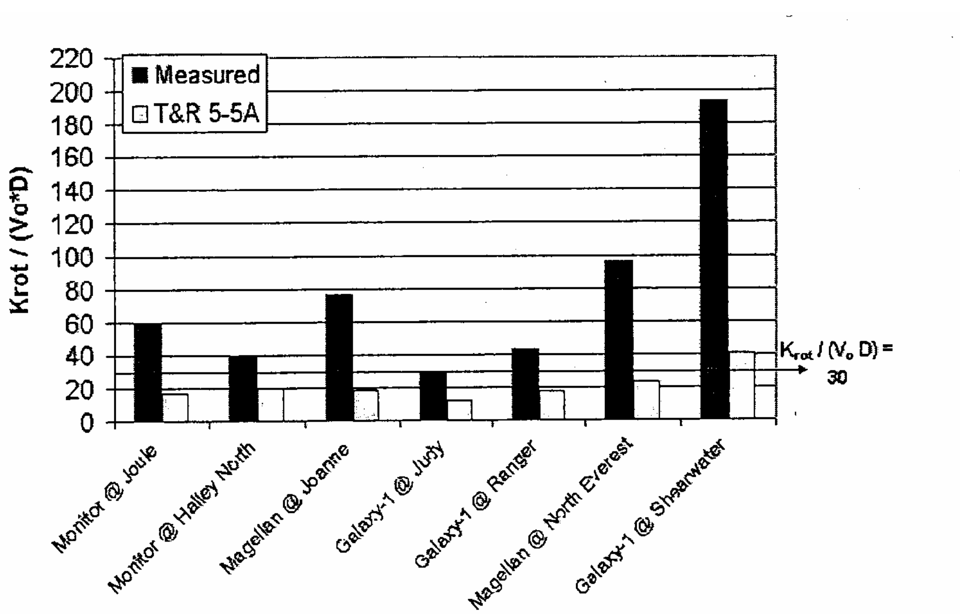


Figure 23 Measured rotation stiffness [65]

It is suggested that a limiting criterion to determine stiffness could be given by:

$$K_{rot} / (V_o D) = 30$$

This provides a lower bound to the measured stiffnesses which can be used in lieu of a (SNAME Step 3) full non-linear interaction study.

4.8.4 Conclusions

Measurements from the field and from centrifuge testing have confirmed that the SNAME Level 2(b) analysis underestimates rotational stiffness. This may not always be conservative. Although increased foundation stiffness leads to smaller moments at the guides, it results in larger moments at the spudcans under the design storms. Increased stiffness under developing storm results in a change of natural period and the dynamic response will be affected.

A study on fixity in clay (61) suggest that the initial stiffness in clays could be increased to provide a shear modulus, given as $G = 600S_u$. This is significantly greater than the maximum SNAME value of $200S_u$. Other studies have confirmed that there is significant rotational stiffness still available under the design assessment storm. For sands, a lower bound value of $K_{rot}/(V_o D) = 30$ is suggested based on centrifuge tests (65).

4.9 JACK-UP SPUDCAN/JACKET PILE INTERACTION

4.9.1 Description of problem

Jack-up rigs are often used to carry out drilling or work-overs on a fixed jacket installation. The jack-up is usually positioned close to the jacket so that the draw works can be provided with an adequate reach over the wells. Depending on the jacket footprint and the positioning of the jack-up rig, the spudcans may be close to the permanent piled foundations of the jacket resulting in stressing of the piles due to the lateral deformations caused by the penetration of the spudcans.

4.9.2 Factors influencing piled foundations

According to Lyons and Willson (52) the principal factors affecting the interaction between the spudcan and adjacent piling include the following:

- Soil movements around the pile caused by the penetration of the spudcan
- Changes in the stress and strain field around the spudcan as the spudcan loadings vary
- Spudcan sliding into previous footprints
- Scour around the spudcan.

In addition, the effects on the piles will depend on pile geometry (diameter and thickness) and the load level.

Soil movements

The penetration depth of the spudcan can be determined using the bearing capacity/depth profile. From the estimated depth, the lateral and axial soil movements may be estimated using slip-circle analysis, cavity expansion theory or finite element analysis. These displacements may then be in input to a conventional sub-grade reaction program (beam-column analysis) using p-y and t-z curves, as Y or Z shifts. The additional stresses in the piles due to penetration of the spudcan may then be estimated.

According to Mirza et al [17], experimental data on homogeneous soils indicates that in dense sand the failure zone may extend up to 2 to 2.5 times the width on each side and up to one footing width in depth. The authors state that other data points to effects up to 4 to 4.5 times the width laterally and up to 2 times the width vertically. It was also noted that the effects of conical footing tips can be to reduce the capacity by up to 75% for a cone tip angle of 120 degrees in sand, and therefore the interaction effects will be more onerous. Mirza et al [17] confirm that the problem of determining soil movements, particularly in stratified soil, is difficult. Surface footings effects may be determined using established elastic solutions. However it is often difficult to assess an appropriate soil modulus due to anisotropy, non homogeneity and inelastic response. For penetrating spudcans, spherical cavity expansion can be used, although results tend to overestimate movements. However there is no reliable method of predicting soil movements due to a penetrating object such as a spudcan, apart from finite element analysis with non-linear and large displacement capabilities to model this problem.

Pile loads are difficult to predict in the absence of soil load measurements. Methods are available to compute ultimate soil loads, for instance under gross soil movements such as mudslides. Mirza et al note that the conventional procedure is to use p-y analysis, and conclude from case histories comparisons that the Matlock [66] criteria for soft clays leads to conservative results, whilst the Reese et al [67] criteria for sand provides reasonable predictions.

Stresses in the pile will be dependent on load levels in the pile due to jacket and on spudcan reactions, as well as pile wall thickness. Therefore analysis of the interaction of pile loads and spudcan displacements may lead to a requirement for increased thickness in the pile wall at mudline and to a sufficient depth to resist the developed stress increases.

Effect of operational and storm loadings

Finite element modelling has been used to assess the soil displacements due to loads on the spudcans from operational and storm conditions. These displacements were then input into a beam column model of the piled foundation to determine the additional stresses. Lyons and Willson [52] have analysed a profile comprising a dense sand incorporating a clay layer, and a profile of dense sand alone. Resulting axial deflections are increased by about 12% for both the mixed profile and the all sand profile. Corresponding lateral displacements are increased by about 10% for the mixed profile and 1% for the sand profile. When the profile is wholly sand, the increase in bending moment is only 1%. Pile ultimate capacities were not affected by the spudcan.

Sliding into previous footprints

Additional stress may be caused to adjacent piling by the sliding of the spudcan into previous footprints.

Scour

The effects of scour on piled foundations are to reduce the lateral support to the pile near the heavily stressed pile head, and to reduce the ultimate capacity of the piles. This is usually incorporated in the pile design. However, it is possible that local velocity increases near the adjacent spudcan can cause increased scour around the pile.

The effects of scour may also be to reduce the bearing area of the spudcan and therefore lower the bearing capacity of the spudcan, thereby resulting in sudden penetration of the spudcan to a deeper equilibrium position. This may cause additional stresses in the adjacent piles, and therefore measures should be taken to prevent this.

4.9.3 Centrifuge testing

Siciliano et al [(68)] carried out scaled testing using model spudcans and instrumented piles located at spacings varying from 0.25 x spudcan radii to 2.0 x spudcan radii from the spudcan edge. Displacement profiles of the piles were measured and bending stresses were calculated using a beam column model. The soils used were normally consolidated laboratory clay. Results were presented as maximum normalized displacements and normalized mudline displacements. Measured pile displacements were much smaller than expected considering the volume of soil displaced by the spudcan. Maximum lateral pile displacement was about 120mm, suggesting that soil flowed back above the spudcans rather than being pushed laterally. The zone of influence of the spudcans extended to about one diameter depth below the footing. Surface displacements were very small at a distance of one spudcan diameter and greater from the edge.

4.9.4 Conclusions

When spudcan clearance from the pile edge is greater than one diameter, the effect of spudcan loadings on the pile is minimal. Thus SNAME concludes that no consideration need be given for this condition.

When pile to spudcan spacings are closer than one diameter, then for surface footings, analysis can be carried out using established elastic solutions for soil movement which can be used as input displacements to a beam-column model or a soil structure interaction program such as SPLICE. When full or partial embedment occurs, as in soft soils, cavity or spherical expansion models can be used to assess the soil movements.

Finite element analysis can be used to determine the effects of spudcan surface footings or embedded foundations, using von Mises plasticity for clays and Mohr Coulomb models for sands. This may be the best approach, particularly for non-uniform soils.

In lieu of more detailed analysis it can be assumed that maximum pile stresses increase by 10% due to the effect of adjacent spudcans.

4.10 EFFECTS OF CYCLIC LOADING

4.10.1 Background

Due to wave and current effects, the spudcan foundations will be subjected to cyclic loading. This may cause distress with increased displacements, settlement or, at the extreme, liquefaction of the soil under the foundation. The effects of cyclic loads will depend on both the magnitude and frequency of loading as well as the nature of the soil deposits. It has been common to assume that the degradation due to cyclic load effects from wave frequencies would be balanced by enhancement due to rate effects caused by the same wave loading [(61)]. Both SNAME and ISO note that bearing capacity may be reduced due to cyclic action. In the SNAME commentary it is concluded that there are uncertainties due to cyclic loading, as well as consolidation and creep and rate effects. According to SNAME, based on recommendations by Andersen [(69)] the cyclic degradation effects on clay are significantly greater on the leeward leg than for the windward leg. A reduction of 20% should be applied to the windward spudcan. This is due to greater uncertainty in the sliding capacity part of the interaction equation than in the bearing capacity, where greater capacity can be generated by increased penetration. ISO comments that cyclic environmental loading or operational vibrations may induce additional settlements and that special attention should be given to cyclic loading in silty sand or silt. Cyclic loading may also involve soil strength reduction, which may induce settlements due to bearing failure.

Poulos [(70)] differentiates between the behaviour of contractive and dilative soils. Under low stress cyclic loading, strains increase to a limiting value and then stabilise. With high stress cyclic loading, strain increases steadily until failure occurs at a stress less than the static strength. Pore pressures increase in high cyclic stress until the effective stress level within the soil fabric is reached. With low cycle stress, there is time for the pore pressure to dissipate. In critical state soil mechanics description, there is a steady state limit state (SS) and a cyclic limit state (CLS), marking the boundaries between contractive and dilative conditions. The contractive soil is defined by its response to cyclic loading in which pore pressures increase or void ratios decrease until failure. The dilative soil is characterised by a reduction in pore pressure or an increase in void ratio.

Unless the soils can be classified as contractive it is unlikely that liquefaction or cyclic failure will occur. However, some very weak silts might fall into this category. The most usual scenario is for some additional straining to occur, leading to settlements in either clays or sands. The magnitude of these settlements will depend on the soil characteristics. Strength reduction may also occur, although this needs to be balanced against possible increases due to consolidation effects under load.

Anderson [(69)] comments that plate loading tests confirm that bearing capacity under combined static and cyclic loads may be significantly smaller than under static loads, and shows examples with cyclic strength reduced to about 65% of the static load.

4.10.2 Cyclic loading of shallow foundations

The most onerous effects occur in sand deposits. Seed [(71)] comments that under certain conditions of density, loose saturated sands having a contractive structure and subject to shear deformations, might develop very high pore pressures and lose all resistance to deformation, and be said to have liquefied. Under cyclic loading spudcans may develop similar high pore pressure which could be high enough to overcome the effective stresses in the soils. Finnie and Randolph [(72)] report on the liquefaction induced failure of shallow foundations in calcareous sediments. A series of centrifuge tests was carried out on flat bottomed model foundations ranging from 30mm to 150mm diameter. Under conditions of constant vertical load and horizontal cyclic loading high pore pressures developed and liquefaction resulted, when the bearing medium was calcareous silt. This was in contrast to sand, in which some settlement occurred but no liquefaction occurred. In the silt, liquefaction occurred under a cyclic horizontal load, some 4% of the vertical bearing pressure. Improved performance to liquefaction at 10% of the vertical pressure could be attained by preloading by a factor of 2.0.

Dean et al [73] carried out centrifuge testing on spudcans in partially drained silica sand. They found that settlement under positive vertical load helped prevent the development of pore pressures and associated liquefaction. In a report on skirted and non-skirted spudcans, Dean et al [74] carried out centrifuge testing and concluded that the concern for liquefaction of soil beneath a footing may have been over-emphasised, and no cumulative pore pressure were found in testing. However, the vertical displacements after a series of "event" loadings, comprising a maximum 50 cycles of horizontal loading, was a maximum 300mm for a prototype non-skirted spudcan of diameter 4.68m i.e. a displacement/diameter ratio of 0.065.

Dean et al [75] also carried out tests in clay. Re-constituted speswhite kaolin clay powder was used. This was normally consolidated to give a vane strength of about 20 kPA at a depth of 2 x spudcan diameter. Horizontal cyclic loading was applied with a static vertical load. The results indicated that there may be thresholds below which significant vertical displacements do not take place. These thresholds depend on the footing size, and soil characteristics. Test data results are shown for conical footings and flat bottomed footings. Threshold bearing stresses of 20 kPa and 46 kPa were found for the conical and flat bottomed foundation respectively. Significantly it was also found that vertical re-loading effects may erase effects of prior cyclic loading and therefore pore pressure build up is prevented.

The effects of cyclic loading are therefore similar to a reduction in bearing strength, with increased penetration depth required to support the same vertical load.

4.10.3 Recommendations

A thorough description of the subsurface soils along with a suitable laboratory investigation programme is necessary in order to appreciate the possible strength degradation or settlements which could result from in-place loads. If cyclic effects are considered to be a problem, then skirted spudcans may provide some protection against settlements or strength degradation.

Therefore the following recommendations should be included:

- Carry out a detailed soils investigation.
- Gather CPT data to a depth at least equal to the diameter of the spudcan plus penetration.
- If the soils are very sensitive to cyclic load, carry out cyclic testing, either triaxial testing or DSS testing and use a finite element or limiting equilibrium model to determine response to cyclic loading. The model developed by Andersen [69] for gravity structure foundations can be used.
- In lieu of detailed analysis, or testing, include strength reductions for the static shear strengths. A maximum reduction of 35% can be applied.

4.11 DEBRIS AND OTHER OBJECTS ON SEABED

There are a number of objects that possibly may be found, both at seabed surface and just below, which could interfere with the placement of a jack-up rig. The objects include:

- Pipelines
- Wellheads
- Wrecks
- Anchors
- Cables

- Dropped objects (from adjacent jacket platform)
- Glacial dropstones
- Ordnance.

Both SNAME and ISO provide for such objects by recommending appropriate bathymetric, magnetometer, side scan sonar and diver/ROV surveys to be conducted.

The literature search revealed no further issues except that glacial dropstones are encountered as far South as the Central North Sea and are often many metres in size [(51)].

It is necessary to repeat suitable surveys even where a jack-up is returning to a location because of the possibility of dropped objects, as well as changes in seabed condition (e.g. sand waves). If possible this should be done immediately before positioning the rig, but not more than 6 months previously.

4.12 SHALLOW GAS POCKETS

4.12.1 Definition

Shallow gas is a biogenic or petrogenic gas in the pore water of shallow soils. In situ natural gas could be either gaseous or bound with water to form a solid, known as hydrate [(4)].

4.12.2 Shallow gas pockets effects on jack-up foundations

SNAME and ISO identify the following effects due to shallow gas pockets:

- Unpredictable foundation behaviour due to seabed depressions or gas accumulations under the spudcan
- Reduced bearing capacity
- Hazards during site investigation soil boring
- Complications with shallow drilling operations, including blowouts,
- Potential of gas migration from depth to the surface outside the casing (uncommon).

Reference 51 mentions liquefaction as another effect of shallow gas charges. The release of shallow gas to the seabed during drilling for the conductor or the main well casing could give rise to liquefaction of the formation and a consequent loss of support of foundations. The result of this effect could be catastrophic.

4.12.3 Prediction of presence of shallow gas pockets

The following activities could be undertaken to predict shallow gas potential at a location:

- ISO and SNAME state that the presence of the shallow gas may be identified by geophysical digital high resolution shallow seismic surveys using attribute analysis technique. Due to qualitative nature of seismic surveys it is not possible to conduct analytical foundation appraisals based on seismic data alone. This requires correlation of the seismic data with soil boring data in the vicinity through similar stratigraphy. The effects of dissolution and expansion of gas in recovered soil samples shall be taken into account in developing geotechnical parameters for design [(4)].

- The presence of pockmarks could be a sign of shallow gas existence both within and immediately outside their areas. Pockmarks are shallow, crater shaped depressions which are believed to have been formed by escape of gas from seabed sediments over geological time periods. They are generally circular or elliptical in plan with diameters 50-100 m and 2-3 m deep (51). At their perimeter, slopes of up to 10° are common.
- A review of all available records to assess the potential of shallow gas pockets including any local shallow gas database (51).

4.12.4 Prevention of damage due to shallow gas pockets

During jack-up operations, any of the following could be an indication of shallow gas pocket existence:

- Reduction in standpipe pressure,
- A sudden increase in the rate of penetration,
- A change in bit torque and weight,
- The activation of gas alarms.

The following measurements could be taken to prevent the damage caused by shallow gas:

(a) The well site could be moved to avoid shallow gas pockets, or the rig can work with expensive and time consuming gas safety precautions (76).

(b) If the presence of the shallow gas is detected during drilling, two routes could be used to escape the gas to seabed (51):

  - Through the annulus surrounding the bit and pipe
  - Up the inside of drill string.

(c) If any gas concentration is located above the primary casing shoe level or the conductor pipe shoe level (the levels being determined during the drilling program design), it should be avoided. This is because neither of these holes are drilled under BOP control and, therefore, there is a risk of seabed cratering around the well which could result in the undermining of the footings in the event of a blow out (1 & 2).

4.12.5 Possibility of preparing a database on shallow gas

The issues regarding to the possibility of producing a database on shallow gas in relation to offshore hazards has been studied by the British Geological Survey and reported in OTH 504 (77). The report concludes that shallow gas is not a big problem for foundations and that there is little support from the operators consulted for such a shallow gas database.

4.13 SEAFLOOR INSTABILITY

4.13.1 Definition

Both ISO and SNAME state that seafloor instability can be caused by a number of mechanisms which may be interactive or act independently. The most frequent types of instability result in large scale mass movements, in the form of mudslides or seafloor failure. This instability cause continued foundation settlements or large scale failure of the soil mass as described above [11, 17]. It is recognised [37] that seafloor movements can impose significant lateral and vertical forces as well as reducing bearing capacity.

According to ISO [2], seafloor instability is caused by:

(a) Ocean wave pressure:
Weak, under consolidated sediments occurring in the areas where wave pressures are significant at the seafloor are susceptible to wave-induced movements and can be unstable under very small slope angles.
(b) Earthquakes:
Earthquakes can induce failure of seafloor slopes that are otherwise stable under the existing soil self-weight and wave actions.
(c) Soil self-weight
(d) Hydrates
(e) Shallow gas (see Section 4.12 of this report)
(f) Faults
(g) Or other geological processes.

SNAME considers liquefaction as another reason for seafloor instability and explains it within the seafloor instability section. However, ISO has a separate section on this issue (Section 9.3.4.5). Liquefaction is discussed in Section 4.14 of this report.

4.13.2 Prevention of problems due to seafloor instability effects

Both ISO and SNAME recommend:

(a) To obtain advice of local experts, where instability phenomena is associated with deltaic deposits.
(b) Seabed survey using sidescan sonar or high-resolution multibeam echosounder techniques. Special care should be taken in using these data due to the fact that they become out-of-date quickly, particularly in those areas with mobile sediments or with construction/drilling activities.
(c) Seismic survey to determine near surface soil stratigraphy and to reveal the presence of shallow gas concentrations. Due to qualitative nature of seismic surveys it is not possible to conduct analytical foundation appraisals based on seismic data alone. This requires correlation of the seismic data with soil boring data in the vicinity through similar stratigraphy.
(d) Soil sampling
(e) Other geotechnical testing and analysis. A site-specific geotechnical testing and analysis plus a report is recommended where:
    - the shallow seismic survey can not be interpreted with any certainty, or
    - no geotechnical data is available in nearby area, or
    - significant layering of the strata is indicated, or
    - the location is known to be potentially hazardous.

    A geotechnical investigation should include:
    - A minimum of one borehole to a depth equal to 30 meters or the anticipated footing penetration plus 1.5 to 2.0 times the footing diameter, whichever is the greater.
    - Undisturbed soil sampling and laboratory testing and/or in-situ cone penetrometer testing.
    - Other recognized type of in-situ soil testing such as vane shear and/or pressure meter tests.

- Adequate investigation of all the layers and coring the transition zones at a sufficient sampling rate.

All the above data should be analyzed by an expert to make the appropriate decisions and a geotechnical report should be produced.

ISO states that 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 effect 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/sea floor interaction and the effects of sediment erosion and/or deposition on structure performance.

The scope of site investigation 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.

Reference 5 states that severe mass movements of sea sediments due to seafloor instability are relatively unique and infrequent. So this hazard is seldom considered with respect to mobile drilling units, which usually occupy a location for a short period of time. However, in an incident in August 1980, the un-manned mat-foundation type jack-up Harvey Ward was overturned in the wake of Hurricane Allen near the Mississippi delta and which was attributed to a mudslide.

4.13.3 Recommendations

Both SNAME and ISO have sufficient recommendations to assess the potential of instability problems on site, but give no clear guidance to deal with the problem if there is a potential (they leave it to local experts).

4.14 LIQUEFACTION

4.14.1 Definition

The term liquefaction has been used to describe a number of different though related phenomenon [78]. As originally used, the term has historically been used in conjunction with a number of phenomena which involve soil deformations caused by monotonic, transient or repeated disturbance of undrained saturated cohesionless soils. The generation of excess pore pressure is the common event linking all instances of liquefaction. According to Reference 78 liquefaction can be divided into two main groups, flow liquefaction and cyclic mobility. Flow liquefaction occurs when the shear stress required for static equilibrium is greater than the shear strength of the soil in its liquefied state. Cyclic mobility describes the build up of excessive deformations during cyclic loading, and is driven by both static and cyclic stresses.

Both SNAME [1] and ISO [2] state that liquefaction, or cyclic mobility, occurs when the cyclic stress within the soils cause a progressive build up of pore pressure. The pore pressure within the profile may build up to a stage where it becomes equal to the initial average vertical effective stress. Foundation failure may result depending on the extent of pore pressure developed in the soil.

4.14.2 Some factors affecting liquefaction

The simplest and most widely used criterion for evaluating liquefaction potential is the grain characteristics of the soil. A simple grading criterion from the National Research Council [79] is shown below. The boundaries for most liquefiable soil are shown by "A", and the limits for potentially liquefiable soils are shown by "B". This clearly shows that grain size affects the behaviour. Fine and uniform sands are more prone to liquefaction than coarse grained ones.

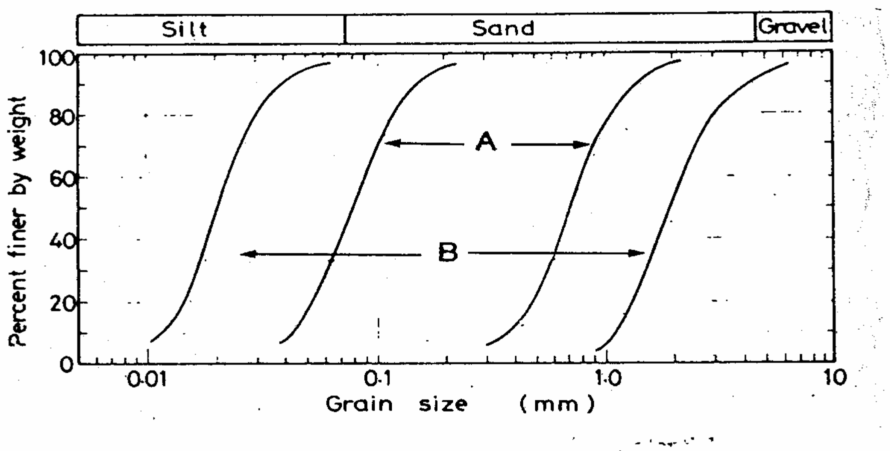


Figure 24 National Research Council grading criteria [79]

According to Kramer [78] the first step in liquefaction hazard evaluation is the assessment of liquefaction susceptibility as decided by historical, geological, compositional and state criteria. Historical assessment is useful for seismic studies, since it is known that susceptibility is dependent on magnitude and proximity to the earthquake epicentre. Geological assessment is useful since soils that are loose and of uniform grain size are most susceptible. Therefore fluvial soils of Holocene age are more susceptible than Pleistiocene deposits. In general older deposits are less at risk. With regard to composition, for many years liquefaction was thought to be limited to sands. However, liquefaction of nonplastic silts has been observed and therefore plasticity characteristics are of significance. Coarse silts which are nonplastic and with bulky particles are at risk of liquefaction; clays are non-susceptible. Well graded soils are generally less susceptible than poorly graded soils and soils with rounded grains are more likely to densify and therefore liquefy under cyclic loading. State criteria are also significant and as described in the section on cyclic loading of soils, soils on the dry side of critical are less susceptible to cyclic loading and to the accumulation of strains and displacements.

Commentaries to Recommended Practice for Site Specific Assessment of Mobile Jack-up Units in SNAME T&R 5-5A indicates that the rate and degree of pore pressure build up will depend on three factors:

a) The loading characteristics; that is, the amplitude, period and durations of the different cyclic loading components

b) The cyclic characteristics of the soil deposits
c) The drainage and compressibility of the strata comprising the soil profile.

4.14.3 Methods to evaluate liquefaction resistance

The term 'liquefaction potential' refers to the possibility of a soil undergoing continued deformation due to the build-up of high pore water pressures and hence low effective stresses. Evaluation of liquefaction potential generally involves the determination of the combinations of cyclic stress and number of cycles which will cause initial liquefaction (a peak cyclic pore pressure ratio of 100%).

Several approaches to evaluate the liquefaction potential have been developed. The commonly employed are cyclic stress approach and cyclic strain approach to characterize the liquefaction resistance of soils both by laboratory and in-situ (field) tests. The cyclic stress approach to evaluate liquefaction potential characterizes both earthquake loading and the soil liquefaction resistance in terms of cyclic stresses. But, in the cyclic strain approach, earthquake loading and liquefaction resistance are characterized by cyclic strains. For laboratory testing, both cyclic triaxial test and cyclic simple shear test are used for cohesive soils to determine strain accumulation and cyclic degradation. Field tests have become the state-of-practice for investigations of liquefaction resistance on site. Several field test based methods have been used for evaluation of liquefaction resistance, including the standard penetration test based method (SPT), the cone penetration test based method (CPT), shear-wave velocity measurements based method (Vs), and the Becker penetration test based method (BPT). The in-situ (field) tests provide a rapid, reliable, and economical means of determining liquefaction potential, soil stratigraphy, relative density, strength and hydro geologic information. Historically the SPT test has been used to derive liquefaction criteria. However, the SPT (N) blowcounts can usually be related to CPT q_c values by correlation factors and charts.

4.14.4 Simplified evaluation of wave-induced liquefaction

In general, two different criteria for the liquefied state have been used in the past. The first criterion is the Nataraja and Gill analysis as reported in Reference 70 based on the data obtained from the in-situ (field) tests for soil. The procedure involves the following steps:

a) Selection of design wave data for input into the analysis; required are significant wave height, significant wave period, largest wave height, wave length, and still-water depth.

b) Computation of the wave-induced bottom pressure, p_0

$$p_o = \gamma_w \frac{H}{2} \frac{1}{\cosh(2\pi h / L)}$$

where H is the wave height, γ_w is the unit weight of water, and h is the water depth.

c) Computation of the amplitude of the wave-induced shear stresses

$$\tau_{vh} = p_0 \lambda z \cdot \exp(-\lambda z) \sin(\lambda x - \omega t)$$

where z is the distance below mudline, x is the horizontal co-ordinate, L is the wavelength, $\lambda = 2\pi / L$, t is time and the circular frequency $\omega = 2\pi / T$ (T is the wave period). If the depth of interest is less than 10% of the wavelength, the following linear approximation may be used for τ_{vh}

$$\tau_{vh} \approx 3.25 p_0 z / L$$

d) Estimation of the cyclic shear strength from SPT test data, or other data, that can be translated into SPT values. This is done by first selecting a design profile of values of *N* (the SPT number) and then converting the *N* values into modified penetration resistance values, N_1, by using the following equation:

$$N_1 = (1 - 1.25 \log_{10} \frac{\sigma_v'}{\sigma_1'}) N$$

where σ_v' is the effective overburden pressure, and σ_1' is the unit pressure. The cyclic shear stress ratio required to cause liquefaction is then calculated, either from available test data, or from the simplified empirical relationship

$$(\tau / \sigma')_l = 0.009 N_1$$

Thus, the cyclic shear stress to cause liquefaction can be found, as

$$\tau_l = 0.009 N_1 \sigma_v'$$

e) Finally, the factor of safety is calculated, as a function of depth, as the radio τ_l / τ_{vh}

The second criterion is based on the concept of excess pore pressure, as suggested by Zen and Yamazaki (80) for a two-dimensional case. This was modified by Jeng (81) to a three-dimensional case as

$$1/3(\gamma_s - \gamma_w)(1 + 2K_0)z + (P_o - p) \leq 0$$

in which γ_s and γ_w are the unit weight of soil and water, K_0 is the coefficient of earth pressure at rest and P_o is the wave pressure at the seabed surface and p is the wave-induced pore pressure.

4.14.5 Evaluation of earthquake-induced liquefaction potential

Simplified methods are presented by Seed (82) and by Robertson and Campanella (83), both methods employ data from in-situ penetration tests. In general, these methods involve three main steps:

1. Estimation of the cyclic shear stress induced at various depths within the soil by the earthquake, and the number of significant stress cycles.
2. Estimation of the cyclic shear strength of the soil, i.e. the cyclic shear stress ratio which is required to cause initial liquefaction of the soil in the specified number of cycles.
3. Comparison between the induced cyclic shear stress and the cyclic shear strength; at locations where the induced shear stress exceeds the shear stress required to cause initial liquefaction, there is a potential for liquefaction.

The application of these methods and the correlation between penetration testing and the cyclic stress to cause liquefaction is given in Reference 70 and Reference 78, for instance.

4.14.6 Preventing liquefaction failure

It was found that liquefaction failure can be avoided through alternative strategies of:

1. Preloading of the foundation [72]
2. Improvement of soil drainage and densification
3. Using hazard zone mapping, which is possible to identify areas potentially subject to liquefaction
4. Using suction caissons as the jack-up foundation.

ISO [11] indicates that in areas where liquefaction is known to be a hazard, its potential shall be assessed.

4.14.7 Recommendations

The risk of liquefaction due to seismic activity in the UK North Sea Sector is low. There are insufficient recommendations in both SNAME [1] and ISO [2] to evaluate liquefaction. However, if it is recognized that there may be a potential hazard, due to seismic activity or due to wave action, it may be evaluated by the methods described above.

5 KEY FINDINGS AND CONCLUSIONS

5.1 KEY FINDINGS

The major causes of jack-up foundation problems that may occur during the installation phase and during in-service are identified in Figures 25 and 26 respectively. The left hand side of both figures refers to the preventative controls that may be employed either to reduce the risk of foundation problems occurring or even to negate the risk entirely. Possible mitigation measures appear beneath the boxed entries for the causes and effects. Inspection of Figures 25 and 26 reveals the following:

- There are few mitigation techniques for alleviating either a cause or its effect. The only option in the majority of the scenarios depicted in the figures is to ensure that the scenario is prevented from happening in the first instance.
- The best approach for the prevention of foundation problems is the careful assessment of site and soil conditions. This may indicate a need to relocate the jack-up unit.
- Whereas mitigation techniques exist to allow for the possibility of punch-through during the installation phase, there is none for the in-service condition. It is vital, therefore, that soils data is assessed carefully and that actual penetration behaviour is used to verify predicted behaviour.
- RPD monitoring and comparison against specified safe limits should be used to ensure leg integrity during any jacking operation, including hull relevelling.

The following specific key findings have been made during this study. They are organised according to topic heading.

Foundation related incidents

- It is estimated that approximately one third of all jack-up accidents are associated with foundation problems. Over 50 incidents involving jack-up foundations have been identified from the literature.
- Punch-through represents more than half of the noted foundation problems, and also has been the cause of the majority of associated fatalities. This high punch-through incident rate was noted more than a decade ago [(84)].

Punch-through

- Punch-through may occur either during jack-up unit installation (i.e. when preloading) or in-service during severe storms.
- Punch-through is particularly associated with failure of hard layers in multi-layered soils, but it can also result from artificial crusts formed by the loading regime imposed by the jack-up prior to preloading.
- There appears to be sufficient availability of design/assessment approaches. SNAME recommends the consideration of load factors against potential punch-through based on the merits of each case. The factors of safety in SNAME however, do not seem large enough to prevent punch-through problems from occurring.
- There are practical steps that can be, and are, undertaken to mitigate against the effects of punch-through should it occur.

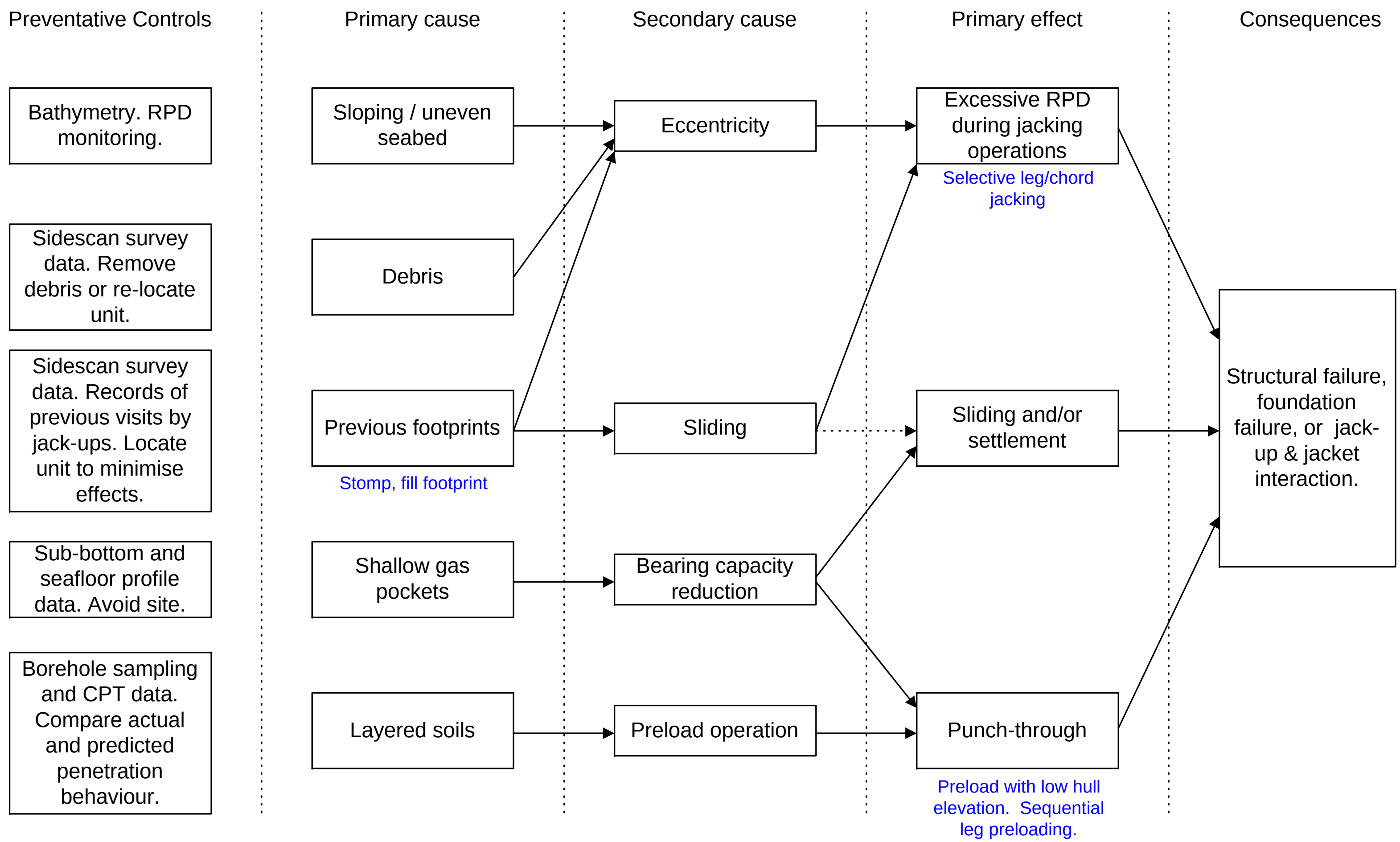


Figure 25 Jack-up problems during installation, their preventative controls and mitigation techniques

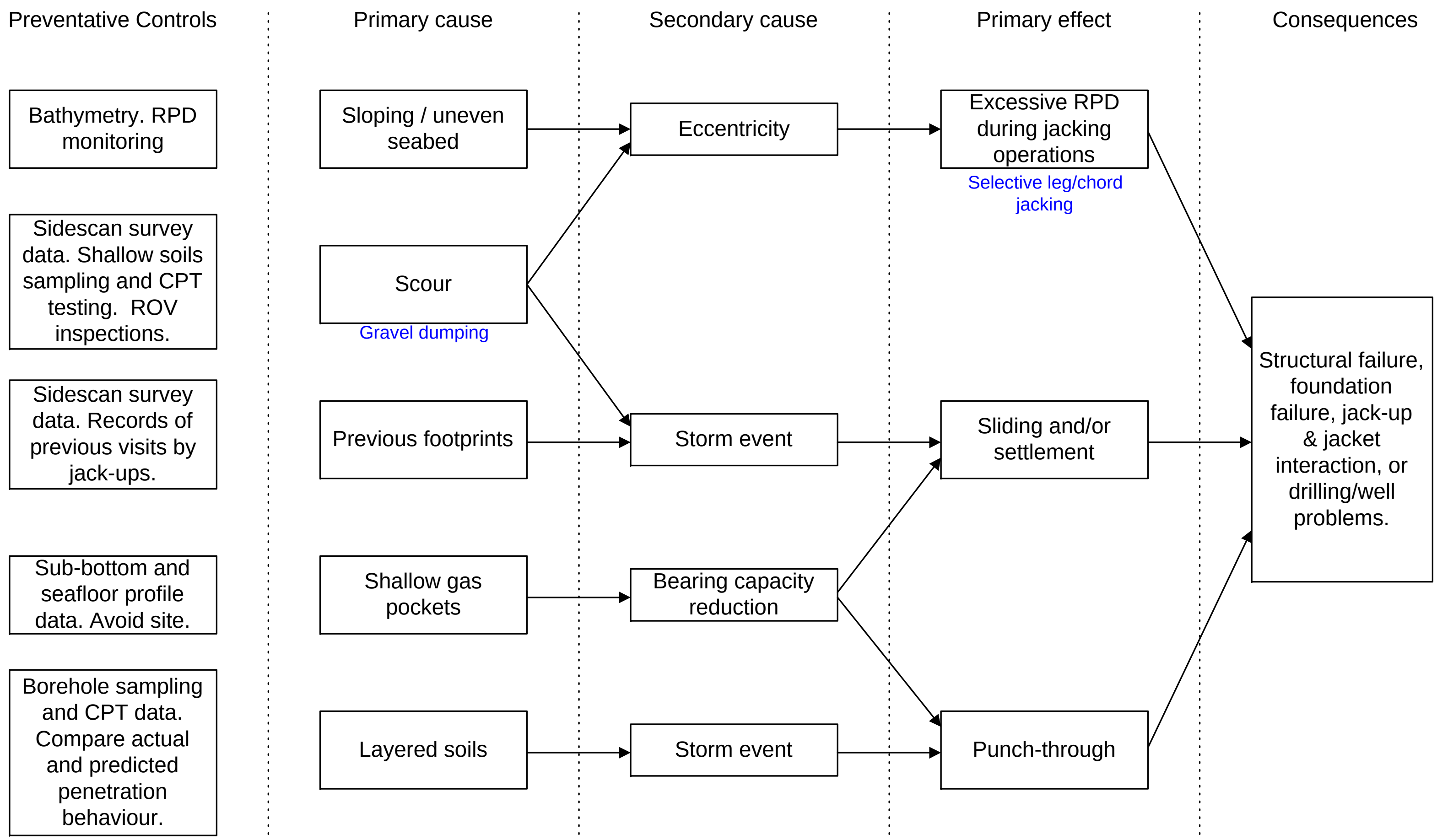


Figure 26 Jack-up problems during in-service, their preventative controls and mitigation techniques

Bearing failure and settlement

- Controlled bearing failure occurs during spudcan preloading and is part of the proof loading procedure of the soils. Monitoring of spudcan penetrations can allow an in-situ assessment to be made of soil parameter values or capacity calculation methods.
- Conical spudcan shapes may result in reduced bearing capacity which can be assessed for clays using SNAME. Methods for sands, although not given in SNAME, can be determined as referenced in Section 4.
- Effects of combined moment and shear loading with axial load reduce the capacity. A SNAME level 3 check may be required to determine settlements particularly in soft soils.

Sliding Failure

- Sliding is more likely to occur on sands, especially where the spudcan does not penetrate completely. In hard clays penetration may be shallow and sliding may also be a risk. SNAME recommends higher factors of safety for sliding in such clays than for sand due to possibility of cyclic strength degradation.
- In soft clays the spudcan will penetrate more deeply and sliding is less likely to occur, as adhesion and passive resistance will oppose it.
- At low vertical loads failure is characterised by sliding associated with uplift. On leeward legs combined axial and shear loading may lead to increased penetration of the spudcan.
- Sliding resistance can be improved by site measures such as jetting to allow deeper penetration depth or providing higher loads on the windward leg. Alternatively the spud can could be reconfigured and provided with skirts.

Previous footprints

- The highest lateral load applied to a leg when setting down on a footprint is when the eccentricity of the leg (with respect to the centre of the footprint) is in the range of half to a full diameter of the spudcan. This range is not the same as suggested by SNAME and ISO.

Rack Phase Difference (RPD)

- Neither SNAME nor ISO have anything to say on this issue, despite several recent incidents where excessive RPD values have caused damage to leg bracing.
- There is a good understanding on the reasons giving rise to RPD and on how it may be controlled.
- It is recommended that acceptable RPD values are calculated for every jack-up unit to permit safe operation during jacking and during service.

Layered Soils

- Where soils comprise strata of sand over clay or clay over sand or mixed strata, then methods for analysis of layered soils should be adopted. Punch-through may be a risk in these soils.
- SNAME provides methods for analysis of layered soils using punch-through models and load spreading methods. The punch-through models are considered conservative.
- Skirted spudcans could be used to resist lateral load and to carry load through to lower strata at the skirt tips so that the load bears on the lower stratum only.

Foundation Fixity

- Measurements of rotational stiffness indicate that SNAME calculations underestimate stiffness under storm conditions. Initial stiffness values are also under-estimated.
- Monitoring of jack-ups in clay suggest that fixity of buried spudcans is 75% greater than SNAME. From the results of centrifuge testing in sands a limiting value for stiffness has been determined for use under storm conditions.

Interaction of Piles and Spudcan

- Spudcan loadings may increase lateral and axial stresses in adjacent piles. This is more onerous in soft soils where both lateral displacements and vertical penetrations may be larger.
- Where the spudcan edge to pile centreline distance is greater than one spudcan diameter, effects on jacket piles can be neglected.

Cyclic Loading

- It has been common to assume that cyclic loading strength degradation may be mitigated by rate effects. However over-consolidated clays with OCR ratios greater than 4 may be subjected to strength reduction which should be incorporated in design.
- For these soils a strength reduction of up to 35% should be included for bearing capacity calculations.

5.2 CONCLUSIONS

This study has reviewed current design practices for jack-up foundations with regard to overall integrity and soil-structure interaction. It is considered that both SNAME and ISO address most of the issues relating to jack-up foundation behaviour in an appropriate manner. However, there are some issues highlighted in this study, which do not appear to be adequately covered:

1. Punch-through is the main cause for concern and has been the cause of the majority of foundation related incidents for jack-ups. An adjustment to the factors of safety used in assessing the bearing capacity of susceptible soils is one approach that can be used to reduce the risk of punch-through occurring during in-service operations.

2. There have been several recent incidents during jacking operations leading to damage of leg bracing members. Such damage would have been preventable if the loads at the leg/hull connection had been controlled by monitoring the rack phase difference (RPD).

Fixity levels are lower in SNAME than suggested from field measurements. Initial stiffness is low and reductions due to applied loading are higher than seem justified. Use of SNAME values could be non-conservative as for instance in calculating dynamic response or in design of spudcans for pinned soil reactions under storm conditions.

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NOTATION

The following symbols have been used in this report.

A	Area of spudcan
A_s	Projected spudcan lateral area
a	Interface friction reduction factor
B	Effective spudcan diameter at uppermost part of bearing area
β	Exponent for interaction equation
$\beta 1$	Exponent for interaction equation
$\beta 2$	Exponent for interaction equation
C_n	Correction factor for SPT value
c_{u1}	Undrained shear strength at spudcan tip
c_{uo}	Undrained shear strength at maximum bearing area
D	Depth from mudline to maximum bearing area
d	Interface friction angle
d_c	Depth factor
d_{ca}	Depth factor in Brinch Hansen equation
d_γ	depth coefficent for drained bearing
d_q	Depth coefficient of surcharge
D_r	Relative density
ER_m	Ratio of actual energy to theoretical energy (%)
E	Eccentricity
f_1, f_2	Factors applied to deep clay combined loading failure locus
F	Strength increase factor
F_{VH}	Load capacity under combined vertical and horizontal load
F_{VHM}	Load capacity under combined vertical, horizontal and moment load
F_{HM}	Load capacity under combined horizontal and moment load
F_M	Load capacity under moment load
ϕ	Drained angle of friction
ϕ_p	Resistance factor for vertical load
F_H	Total sliding resistance
fr	Reduction factor for initial stiffness
F_v	Vertical bearing capacity
F_{vb}	Capacity of lower clay layer
F_{VH}	Axial load capacity with horizontal load
γ'	Unit submerged weight of soil
γ	Load factor
γ_1	Load factor for dead load
γ_2	Load factor for variable load
γ_3	Load factor for environmental load
γ_4	Load factor for inertia load due to dynamic response
G_r	Shear modulus

γ_s	Unit weight of soil
γ_w	Unit weight of water
H	Depth of excavation
H	Horizontal load
H	Distance from spudcan maximum bearing area to weak layer below
H	Wave height
H_{Lo}	Maximum horizontal load capacity with preload
h	Water depth
h_1	Depth of top of spudcan
h_2	Depth of bottom of spudcan
H_o	Max Horizontal load capacity, no axial load or moment
i_{ca}	Inclination factor in Brinch Hansen equation
K_3	Rotational stiffness
k_a	Active resistance coefficient
Ko	Coefficient of pressure at rest
k_p	Passive resistance coefficient
K_s	Coefficient of lateral pressure
L	Long dimension of spudcan
L	Height of scoop mechanism centre of rotation
L	Wave length
λ	Wave coefficient ($2\pi/L$)
M	Applied moment
M*	Moment at top of bucket
M_o	Moment capacity under moment alone
M_{Lo}	Limiting moment with vertical preload
ν	Poisson's ratio
N	Standard penetration resistance
N_1	Modified standard penetration resistance
$(N_1)_{60}$	Corrected standard penetration resistance
N_c	Clay bearing capacity factor
N_γ	Bearing capacity factor for sand
N_q	Bearing capacity factor due to overburden
N_s	Stability number for open hole
P_a	Active resistance force
p_o	Overburden pressure (surcharge)
p_o	Pressure due to wave action
P_p	Passive resistance force
q_u	Ultimate bearing capacity
Q_v	Factored vertical load reaction
Q_{VH}	Factored combined vertical and horizontal load vector
Q_{VHM}	Factored combined vertical, horizontal and moment load vector
ρ	Rate of shear strength increase
r_f	Ratio of load to ultimate load

σ_1	Unit pressure
σ'_{vo}	Effective overburden pressure
s_c	Shape factor
s_{ca}	Shape factor in Brinch Hansen equation
s_g	Shape coeffient for drained bearing
s_q	Shape coefficient for overburden
S_u	Undrained shear strength
S_{ub}	Undrained shear strength of bottom layer
S_{uo}	Undrained shear strength at surface
S_{ut}	Undrained shear strength of top layer
s'_v	Effective overburden pressure
T	Thickness of layer
τ_l	Cyclic stress to cause failure
τ_{vh}	Shear stress due to wave loading
V	Volume of soil displaced
V	Applied vertical load
V_D	Vertical leg reaction due to self weight
V_{Da}	Vertical leg reaction due to dynamic inertia effects
V_E	Vertical leg reaction due to environmental load
V_L	Vertical leg reaction due to variable load
V_{Lo}	Maximum applied preload
V_o	Max axial load capacity, no shear or moment
VH_D	Combined vertical and horizontal load vector due to dead load
VH_L	Combined vertical and horizontal load vector due to variable load
VH_E	Combined vertical and horizontal load vector due to environmental load
VH_{Da}	Combined vertical and horizontal load vector due to dynamic inertia load effects
VHM_D	Combined vertical, horizontal and moment load vector due to dead load
VHM_L	Combined vertical, horizontal and moment load vector due to variable load
VHM_E	Combined vertical, horizontal and moment load vector due to environmental load
VHM_{Da}	Combined vertical, horizontal and moment load vector due to dynamic inertia load effects
z	Distance below mudline

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