RECOMMENDED PRACTICE
FOR SITE SPECIFIC ASSESSMENT
OF MOBILE JACK-UP UNITS

THIRD EDITION – MAY 2007

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NOTE:

The 1.15 value for the load factor given in Section 8 was the result of analytical studies on a limited number of jack-ups and should be considered provisional pending further research. In the interim, alternative lesser values can be used when acceptable rationale is provided. For unmanned units in severe storm conditions, additional reductions are acceptable provided suitable rationale is given and appropriate consideration is taken as to whether the unit is operating in proximity to a platform. Such alternative values would be applicable to all the Acceptance Criteria Checks given herein

This document has been evolved by a joint industry project (JIP) sponsored by a large number of companies who are listed on the next page. Technical and administrative management of the project has been provided by Noble Denton Consultancy Services Ltd. This document has not been produced by SNAME although some SNAME members have participated in its production. SNAME has, at the request of the working group for the JIP, published this document so that it may be widely disseminated in industry. However, SNAME takes no responsibility for any of the technical or other contents of this document. SNAME cannot provide any technical or other support for this document. For naval architects, engineers, or any other persons using this document, technical support is available on a fee-paying basis from American Bureau of Shipping. The contact at American Bureau of Shipping at the time of publication of this document is:

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Although this document is entitled "Recommended Practice for Site Specific Assessment of Mobile Jack-up Units," it must not be construed as a recommended practice proposed by SNAME.

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1 INTRODUCTION

1.1 The purpose of this document is to provide a Recommended Practice (PRACTICE) for use with the 'Guideline for Site Specific Assessment of Jack-Up Units' (GUIDELINE). Each assessment should address the areas of this document as appropriate for the particular jack-up and location as described in Section 1.4 of the GUIDELINE.

1.2 This document has been formulated as a result of a Joint Industry Project involving all sections of the industry. It is not intended to obviate the need for applying sound judgment as to when and where this PRACTICE should be utilized.

1.3 The formulation and publication of this PRACTICE is not in any way intended to impose calculation methods or procedures on any party. It leaves freedom to apply alternative practices within the framework of the accompanying GUIDELINE.

1.4 This PRACTICE relates only to the assessment of independent leg jack-up units in the elevated condition. The development has been based on 3 legged truss-leg units and caution is advised when applying the PRACTICE to other configurations. Transportation to and from the site and moving on and moving off location are not covered in this document.

1.5 This PRACTICE may be revised if and when more information/research results become available.

1.6 For further details of the applicability and limitations, refer to the GUIDELINE.

1.7 This PRACTICE may be used by anyone desiring to do so, and a diligent effort has been made by the authors to assure the accuracy and reliability of the information contained herein. However, the authors make no representation, warranty or guarantee in connection with the publication of this PRACTICE and hereby expressly disclaim any liability or responsibility for loss, damage or injury resulting from its use, for any violation of local regulations with which a recommendation may conflict, or for the infringement of any patent resulting from the use of this publication.

1.8 The load factors presented in Section 8 herein were determined from the reliability analysis of a limited number of jack-up/site combinations. The load factors are provisional pending the further evaluation of the results from a wider range of assessments by the SNAME OC-7 panel.

See also the Note at the foot of the cover page of this document.

2 OBJECTIVES

2.1 The principal objective of this PRACTICE is to provide acceptance criteria and associated engineering methods that may be applied in the site specific assessment of a jack-up to:

a) Establish the geometric suitability of the jack-up with respect to leg length, airgap and leg penetration.

b) Establish that the jack-up is structurally adequate for its intended application.

c) Ensure that the foundation can offer suitable support to meet this objective.

d) Ensure adequate overturning stability.

2.2 This PRACTICE is applicable to the various possible modes of jack-up operation (drilling, production, accommodation, construction, etc.) in all areas of the world. It should be noted that different extreme environmental return periods may be appropriate for manned and unmanned operations.

2.3 The user of this PRACTICE is advised that, in some areas of the world, the requirements of the local regulatory bodies may be more onerous than those recommended herein.

2.4 Scope of the Assessment

2.4.1 The primary objective of the site specific assessment is to ensure the integrity of the jack-up in the elevated condition. The assumptions incorporated into the assessment must conform with the structural condition of the unit.

2.4.2 The assessment will normally assume that the jack-up is in sound mechanical and structural condition and it is the responsibility of the owner to ensure that this is so. The existence of valid documents indicating that the jack-up is presently in class by a recognized classification society is usually sufficient to verify the mechanical and structural condition of the jack-up to the assessor.

2.4.3 Accidental loads (dropped objects, ship impact, etc.) are not specifically addressed and should be covered at the design stage. Furthermore, the site specific assessment addresses the global structural integrity, hence local damage not affecting the overall integrity is outside the scope of the PRACTICE.

2.4.4 As indicated in Section 1.4.1 of the GUIDELINE, the assessment of the jack-up may be carried out at various degrees of complexity. These are as expanded below, at increasing levels of complexity. The objective of the assessment is to show that the acceptance criteria of Section 8 of this PRACTICE are met. If this is achieved by a particular level there is no need to consider a more complex level.

1. Compare site conditions with design conditions or other existing assessments determined in accordance with this PRACTICE.

2.4.4 2. Carry out appropriate calculations according to the simple methods given in this PRACTICE. Possibly compare results with those from existing more detailed/complex calculations.

3. Carry out appropriate detailed calculations according to the more complex methods given in this PRACTICE.

In all cases the adequacy of the foundation should be assessed.

An overall flow chart for the assessment is given in Figure 2.1 overleaf.

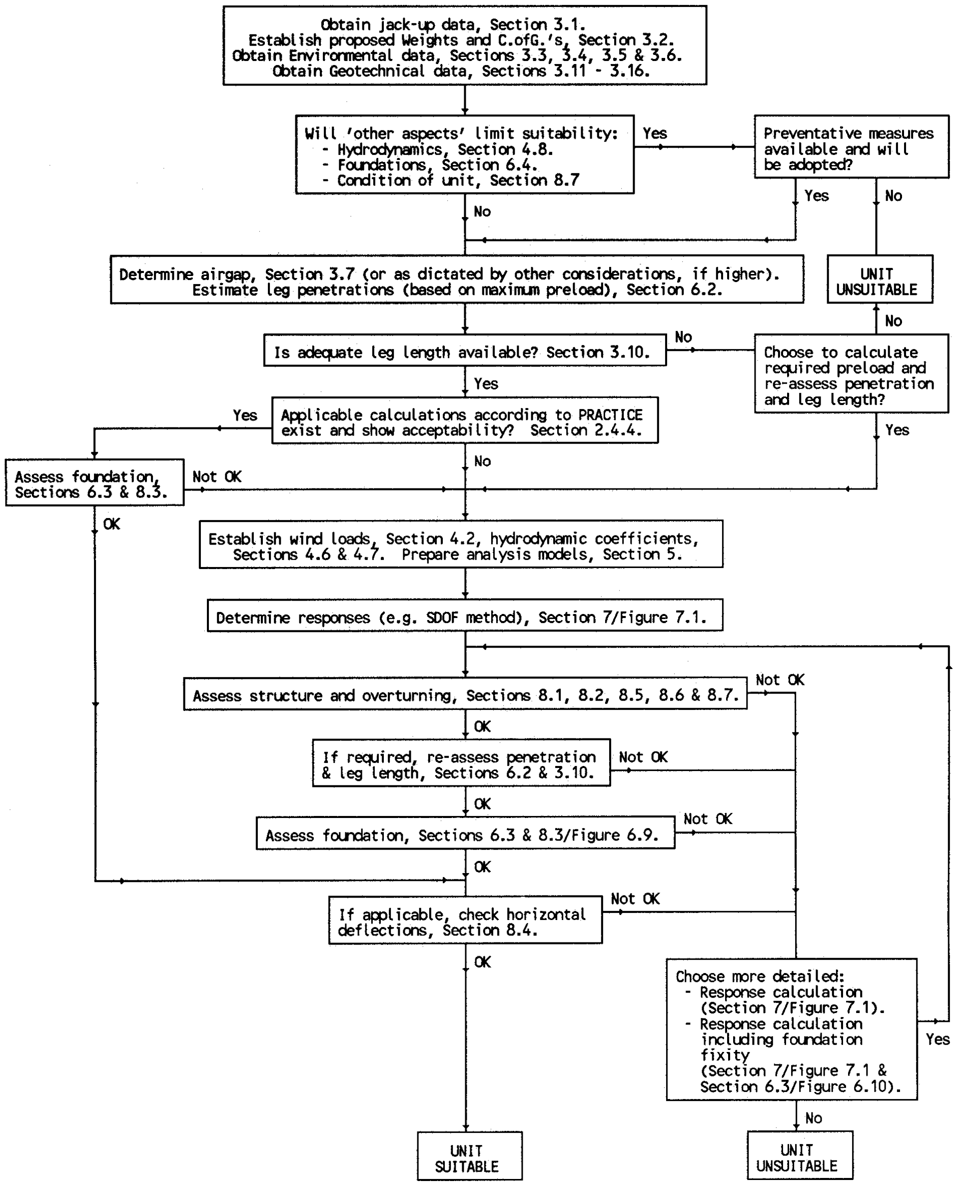


Figure 2.1 - Overall flow chart for the assessment

3 ASSESSMENT INPUT DATA

3.1 Rig data

3.1.1 The information that may be required to perform the assessment is outlined in Section 2.1 of the GUIDELINE.

3.1.2 The operating procedures and limitations of the jack-up should be clearly defined in the Operating Manual. Those sections of the Operating Manual which give relevant information and are required to perform a site assessment in accordance with this PRACTICE are to be provided.

3.2 Functional Loadings

3.2.1 The operating and survival conditions may be treated separately, provided it is practical to change the mode of the jack-up unit from operating to survival mode on receipt of an unfavorable weather forecast, and appropriate procedures exist. The limits of operational loading conditions may depend on the drilling program proposed and consideration should be given to loadings on the conductors if supported by the jack-up.

3.2.2 For both operational and survival conditions, the following shall be defined:

a) Maximum and minimum elevated weight and weight distribution (fixed and variable load), excluding legs. In the absence of other information the minimum elevated weight may normally be determined assuming 50% of the variable load permitted by the operating manual.

b) Extreme limits of center of gravity position (or reactions of the elevated weight on the legs) for the conditions in a) above.

c) Substructure and derrick position, hook load, rotary load, setback and conductor tensions for the conditions in a) above.

d) Weight, center of gravity and buoyancy of the legs.

3.2.3 With reference to Section 4.1.3 of the GUIDELINE, if a minimum elevated weight or a limitation of center of gravity position is required to meet the overturning safety factor in survival conditions, then the addition of water in lieu of variable load is permitted, provided that:

a) Maximum allowable loadings are not exceeded.

b) Procedures, equipment and instructions exist for performing the operation.

c) The maximum variable load, including added water, is used for all appropriate assessment checks (preload, stress, etc.).

3.3 Environmental Conditions - General

3.3.1 The environmental data required for an assessment and their application are discussed in Section 2.3 of the GUIDELINE.

3.3.2 Section 2.3 of the GUIDELINE recommends that 50 year return period extremes are normally used, however in particular circumstances other return periods may be appropriate.

3.3.3 Unless there is specific data to the contrary, wind, wave and current loadings shall be considered to be those caused by the individual return period extremes acting in the same direction and at the same time as the extreme water level. Seasonally adjusted values may be adopted as appropriate to the duration of the operation.

Note:
Where directional and/or seasonal data are utilized, these should generally be factored so that the data for the worst direction and/or season equals the omni-directional/all-year data for the assessment return period.

3.4 Wind

3.4.1 The wind velocity shall be the 1 minute sustained wind for the assessment return period, related to a reference level of 10.0m above mean sea level. The Commentary discusses the conversion of data for averaging periods other than 1 minute to 1 minute values.

3.4.2 The wind velocity profile is normally taken as a power law with exponent $1/10$ unless site specific data indicates otherwise (see Section 4.2.2).

3.4.3 Different jack-up configurations (weight, center of gravity, cantilever position, etc.) may be specified for operating and survival modes. In such cases, the maximum wind velocity considered for the operating mode should not exceed that permitted for the change to the survival mode.

3.5 Waves

3.5.1 The extreme wave height environment used for survival conditions shall, as a minimum, be computed according to the following sub-sections based on the three-hour storm duration with an intensity defined by the significant wave height, H_{srp}, for the assessment return period. The seasonally adjusted wave height may be used as appropriate for the operation.

The wave height information for a specific location may also be expressed in terms of H_{max}, the individual extreme wave height for the return period, rather than the significant wave height H_{srp}. The relationship between H_{srp} and H_{max} must be determined accounting for the effects of storms (longer than 3 hours) and for the additional probability of other return period storms (see Commentary Section C3.5.1). This relationship will depend on the site specific conditions, however H_{srp} may usually be determined from H_{max} using the generally accepted relationship for non-cyclonic areas:

$$H_{srp} = H_{max}/1.86$$

For cyclonic areas the recommended relationship is:

$$H_{srp} = H_{max}/1.75$$

3.5.1 <u>Note:</u>

The wave load can be computed either stochastically (through a random frequency or time domain approach) or deterministically (through an individual maximum wave approach). The scaled wave heights for the two approaches are discussed in Sections 3.5.1.1 and 3.5.1.2 respectively (see Commentary). The scaled wave heights are to be used only in conjunction with the associated kinematics modeling recommended in Section 4.4 and the hydrodynamic coefficients given in Sections 4.6 to 4.8.

3.5.1.1 For stochastic/random wave force calculations Airy wave theory is implied, see Section 4.4.2. To account for wave asymmetry, which is not included in Airy wave theory, a scaling of the significant wave height should be applied to capture the largest wave forces at the maximum crest amplitude. The effective significant wave height, H_s, may be determined as a function of the water depth, d in meters, from:

$$H_s = [1 + 0.5e^{(-d/25)}]\, H_{srp} \qquad (d \geq 25\text{m})$$

and should be used with the wave kinematics model described in Section 4.4.2.

For water depths less than 25m a regular wave analysis should be considered.

The selection of wave period for use in stochastic/random wave force analysis is discussed in Section 3.5.3 and the Note thereto.

3.5.1.2 For deterministic/regular wave force calculations it is appropriate to apply a kinematics reduction factor of 0.86 in order to obtain realistic force estimates (see Commentary). This factor may be considered to implicitly account for spreading and also the conservatism of deterministic/regular wave kinematics traditionally accomplished by adjusting the hydrodynamic properties.

The factor should be applied by means of a reduced wave height, H_{det}. H_{det} may be determined as a function of H_{max} from:

$$H_{det} = 0.86\, H_{max}$$

The use of a factor smaller than 0.86 may be justified by analysis explicitly accounting for the effects of three-directional spreading. However, such effects should be properly balanced by the inclusion of second-order interaction effects between spectral wave components.

The wave loads should be determined using an appropriate wave kinematics model in accordance with Section 4.4.1.

In the analysis a single value for the wave period T_{ass}, in seconds, associated with the maximum wave may be considered. Unless site specific information indicates otherwise T_{ass} will normally be between the following limits:

$$3.44\sqrt{(H_{srp})} < T_{ass} < 4.42\sqrt{(H_{srp})}$$

where H_{srp} is the return period extreme significant wave height in meters.

3.5.2 For airgap calculations the wave crest elevation may be obtained from the formulations of an appropriate deterministic wave theory (see Section 4.4.1) and the maximum wave height, H_{max}, from the relationship:

$$H_{max} = 1.86\ H_{srp}$$

In Tropical Revolving Storm areas the relationship:

$$H_{max} = 1.75\ H_{srp}$$

may alternatively be applied.

It is noted that the minimum return period recommended by the GUIDELINE for H_{srp} for airgap calculations is 50 years, even if a lower return period is used for other purposes.

3.5.3 Where the analysis method requires the use of spectral data, the choice of the analytical wave spectrum and associated spectral parameters should reflect the width and shape of spectra for the site and significant wave height under consideration. In cases where fetch and duration of extreme winds are sufficiently long a fully developed sea will result (this is rarely realized except, for example, in areas subject to monsoons). Such conditions may be represented by a Pierson-Moskowitz spectrum. Where fetch or duration of extreme winds is limited, or in shallow water depths, a JONSWAP spectrum may normally be applied (see Note at the end of this Section).

The wave spectrum can be represented by the power density of wave surface elevation $S_{\eta\eta}(f)$ as a function of wave frequency by:

$$S_{\eta\eta}(f) = (16 I_0(\gamma))^{-1} H_s^2 T_P (T_P f)^{-5} \exp(-1.25/(T_P f)^4) \gamma^q$$

[Note: An alternative formulation is given in the Commentary]
where;

q	=	$\exp(-(T_p f-1)^2/2\sigma^2)$ with:
σ	=	0.07 for $T_p f <= 1$
σ	=	0.09 for $T_p f > 1$ (Carter 1982, [1])

and;

H_s	=	significant wave height (meters), including depth correction, according to Section 3.5.1.1
T_p	=	peak period (seconds)
f	=	frequency (Hz)
γ	=	peak enhancement factor
$I_0(\gamma)$	=	is discussed below.

The above definition yields a single parameter Pierson-Moskowitz spectrum when $\gamma = 1$ and $T_p = 5\sqrt{(H_s)}$, with H_s in meters. In this case an appropriate T_p/T_z ratio is 1.406 (see below).

When considering a JONSWAP spectrum, the peak enhancement factor γ varies between 1 and 7 with a most probable average value of 3.3. There is no firm relationship between γ, H_s and T_p. Relationships between variables for different γ according to Carter (1982) [1] are as follows:

γ	$I_0(\gamma)$	T_p/T_z
1	.200	1.406
2	.249	1.339
3	.293	1.295
3.3	.305	1.286
4	.334	1.260
5	.372	1.241
6	.410	1.221
7	446	1.205

$$\left[\begin{array}{l}\text{Alternatively:} \\ I_0(\gamma) = \dfrac{0.2}{1 - 0.287\,\mathrm{Ln}(\gamma)}\end{array}\right]$$

Unless site specific information indicates otherwise $\gamma = 3.3$ may be used.

For a given significant wave height the wave period depends on the significant wave steepness which in extreme seas in deep water often lies within the range 1/20 to 1/16. This leads to an expression for zero-upcrossing period T_z, related to H_{srp} in meters, as follows:

$$3.2\sqrt{(H_{srp})} < T_z < 3.6\sqrt{(H_{srp})}$$

However in shallow water the wave steepness can increase to 1/12 or more, leading to a zero-upcrossing period T_z as low as $2.8\sqrt{(H_{srp})}$. This is because the wave height increases and wave length decreases for a given T_z.

Note:

If a JONSWAP spectrum is applied the response analysis should consider a range of periods associated with H_{srp} based on the most probable value of T_p plus or minus one standard deviation. However it should be ensured that the assumptions made in deriving the spectral period parameters are consistent with the values used in the analysis. Alternatively, applicable combinations of wave height and period may be obtained from a scatter diagram determined from site specific measurements; in this case specialist advice should be obtained on a suitable spectral form for the location. To avoid the need for analyses of several wave periods a practical alternative is to use a 2 parameter spectrum with $\gamma = 1.0$ in combination with the site specific most probable peak period.

3.5.4 For stochastic/random wave force calculations, the short-crestedness of waves (i.e. the angular distribution of wave energy about the dominant direction) may be accounted for when site-specific information indicates that such effects are applicable. In all cases the potential for increased response due to short-crested waves should be investigated. The effect may be included by means of a directionality function $F(\alpha)$, as follows:

$$S_{\eta\eta}(f, \alpha) = S_{\eta\eta}(f).F(\alpha)$$

where;

α = angle between direction of elementary wave trains and dominant direction of the short-crested waves.

$S_{\eta\eta}(f, \alpha)$ = directional short-crested power density spectrum.

$F(\alpha)$ = directionality function.

and, in the absence of more reliable data:

$$F(\alpha) = C.Cos^{2n}\alpha \text{ for } -\frac{\pi}{2} \leq \alpha \leq \frac{\pi}{2}$$

where;

n = power constant

C = constant chosen such that:

$$\sum_{-\pi/2}^{\pi/2} F(\alpha)\ .d\alpha = 1.0$$

The power constant n, should not normally be taken as less than:

n = 2.0 for fatigue analysis

n = 4.0 for extreme analysis

3.5.5 Where the natural period of the jack-up is such that it may respond dynamically to waves (Section 7.3), the maximum dynamic response may be caused by wave heights or seastates with periods outside the ranges given in Sections 3.5.1.2 and 3.5.3. Such conditions shall also be investigated to ensure that the maximum (dynamic plus quasi-static) response is determined.

3.5.6 For fatigue calculations (Section 7.4), the long term wave climate may be required. For the purposes of the fatigue analysis the long-term data may be presented deterministically in terms of the annual number of waves predicted to fall into each height/period/direction group. Alternatively the probability of occurrence for each seastate (characterized by wave energy spectra and the associated physical parameters) may be presented in the form of a significant wave height versus zero-upcrossing period scatter diagram or as a table of representative seastates.

3.6 Current

3.6.1 The extreme wind driven surface current velocity shall be that associated with the assessment return period wind, seasonally adjusted if appropriate. When directional information regarding other current velocity components is available the maximum surface flow of the mean spring tidal current and the assessment return period surge current, seasonally adjusted if appropriate, shall be vectorially added in the down-wind direction and combined with the wind driven surface current as indicated in Section 3.6.2.

If directional data are not available the components shall be assumed to be omni-irectional and shall be summed algebraically.

Note: A site specific study will normally be required to define the current velocity components.

3.6.2 The current profile may be expressed as a series of velocities at certain stations from seabed to water surface. Unless site specific data indicates otherwise, and in the absence of other residual currents (such as circulation, eddy currents, slope currents, internal waves, inertial currents, etc.), an appropriate method for computing current profile is (see Figure 3.1):

$$V_C = V_t + V_s + (V_w - V_s)\,[(h+z)/h], \text{ for } |z| \le h \text{ \underline{and} } V_s < V_w$$

$$V_C = V_t + V_s \qquad \text{for } |z| > h \text{ \underline{or} } V_s \le V_w$$

where;

V_C = current velocity as a function of z. Note that a reduction may be applicable according to Section 4.5.
V_t = downwind component of mean spring tidal current.
V_s = downwind component of associated surge current (excluding wind driven component).
V_w = wind generated surface current. In the absence of other data this may conservatively be taken as 2.6% of the 1 minute sustained wind velocity at 10m.
h = reference depth for wind driven current. In the absence of other data h shall be taken as 5 meters.
z = distance above still water level (SWL) under consideration (always negative).

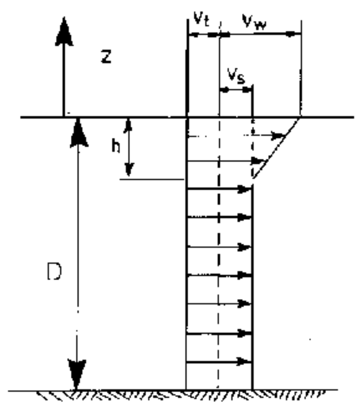


Figure 3.1 - Suggested current profile

3.6.3 In the presence of waves the current profile should be stretched/compressed such that the surface component remains constant. This may be achieved by substituting the elevation as described in Section 4.4.2. Alternative methods may be suitable, however mass continuity methods are not recommended. The current profile may be changed by wave breaking. In such cases the wind induced current could be more uniform with depth.

3.6.4 For a fatigue analysis, current may normally be neglected.

3.7 Water Levels and Airgap

3.7.1 The water depth at the location shall be determined and related to lowest astronomical tide (LAT). The relationship between LAT and Chart Datum is discussed in the Commentary.

3.7.2 The mean water level (MWL) related to the seabed shall be expressed as the mean level between highest astronomical tide (HAT) and lowest astronomical tide (LAT) i.e.:

MWL = (HAT + LAT)/2

3.7.3 The extreme still water level (SWL) shall be expressed as a height above LAT, and shall be the sum of;

Mean high water spring tide (MHWS)
\+ 50 year extreme storm surge (see Note 1).

unless reliable data indicates that an alternative summation is appropriate.

3.7.4 When lower water levels are more onerous the minimum still water level (SWL) to be considered in the loading calculations shall be the sum of:

Mean Low Water Spring Tide (MLWS)
\+ 50 year negative Storm Surge.

3.7.5 The Airgap (see Note 2) is defined in Section 3.2 of the GUIDELINE as the distance between the underside of the hull and LAT during operations. It shall be not less than the sum of:

Distance of the extreme still water level (SWL), from Section 3.7.3, above LAT
\+ 50 year extreme wave crest height associated with H_{max} as defined in Section 3.5.2 (see Note 1),
\+ 1.5m Clearance to the underside of the hull (or any other vulnerable part attached to the hull, if lower). See Commentary.

Notes:
1. Section 3.2.1 of the GUIDELINE recommends that values for a return period of no less than 50 years be applied, even if a lower return period is used for other purposes.
2. The definition of Airgap used herein differs from that used in other areas of offshore engineering where the Clearance used here is often defined as Airgap.

In areas subject to freak waves a higher airgap may be applicable.

3.8 Temperatures

The lowest average daily air and water temperatures shall be compared with the steel design temperature limits of appropriate parts of the jack-up. If these are not met, suitable adjustments should be made to the properties applied in the strength assessment.

3.9 Marine Growth

Where existing marine growth is not to be cleaned between locations or where the operation is to last long enough for significant growth to occur, the influence of growth on the leg hydrodynamic properties should be considered as stated in Section 4.2.3 of the GUIDELINE. Where applicable, location specific data should be obtained. In the absence of such data, default values for thickness and distribution are given in Section 4.7.3.

3.10 Leg Length

Recommendations regarding the reserve leg length are given in Section 3.3 of the GUIDELINE.

3.11 Geotechnical and Geophysical Information

Adequate geotechnical and geophysical information must be available to assess the location and the foundation stability. Aspects which should be investigated are shown in Table 3.1 and are discussed in more detail in the referenced Sections. The information obtained from the surveys and investigations set out in Sections 3.12 to 3.16 is required for areas where there is no data available from previous operations. In areas where information is available it may be possible to reduce the requirements set out below by use of information obtained from other surveys or activities in the area. See Section 2.4 of the GUIDELINE.

3.12 Bathymetric Survey

3.12.1 An appropriate bathymetric survey should be supplied for an area approximately 1 kilometer square centered on the location. Line spacing of the survey should typically be not greater than 100 meters x 250 meters over the survey area. Interlining is to be performed within an area 200 meters x 200 meters centered on the location. Interlining should have spacing not exceeding 25 meters x 50 meters.

3.12.2 Further interlining should be performed if any irregularities are detected.

3.13 Seabed Surface Survey

3.13.1 The seabed surface shall be surveyed using sidescan sonar or high resolution multibeam echosounder techniques and shall be of sufficient quality to identify obstructions and seabed features and should cover the immediate area (normally a 1 km square) of the intended location. The slant range selection shall give a minimum of 100% overlap between adjacent lines. A magnetometer survey may also be required if there are buried pipelines, cables and other metallic debris located on or slightly below the sea floor.

RISK	METHODS FOR EVALUATION & PREVENTION	REFERENCE SECTION(S)
Installation problems	- Bathymetric survey	3.12
Punch-through	- Shallow seismic survey - Soil sampling and other geotechnical testing and analysis	3.14 3.16 6.2.6
Settlement under storm loading/Bearing failure	- Shallow seismic survey - Soil sampling and other geotechnical testing and analysis - Ensure adequate jack-up preload capability	3.14 3.16 6.2.6 6.3
Sliding failure	- Shallow seismic survey - Soil sampling and other geotechnical testing and analysis - Increase vertical footing reaction - Modify the footing(s)	3.14 3.16 6.3.3
Scour	- Bathymetric survey (identify sand waves) - Surface soil samples and seabed currents - Inspect footing foundations regularly - Install scour protection (gravel bag/ artificial seaweed) when anticipated	3.12 3.15 6.4.3
Seafloor instability (mudslides)	- Side scan sonar, shallow seismic survey - Soil sampling and other geotechnical testing and analysis	3.13 3.14 3.16 6.4.4
Gas pockets/ Shallow gas	- Digital seismic with attribute analysis processing (shallow seismic)	3.14 6.4.5
Faults	- Shallow seismic survey	3.14
Metal or other object, sunken wreck, anchors, pipelines etc.	- Magnetometer and side scan sonar - Diver/ROV inspection	3.13
Local holes (depressions) in seabed, reefs, pinnacle rocks or wooden wreck	- Side scan sonar - Diver/ROV inspection	3.13
Legs stuck in mud	- Geotechnical data - Consider change in footings - Jetting	3.14 3.16
Footprints of previous jack-ups	- Evaluate location records - Consider filling/modification of holes as necessary	3.12 3.13 6.4.2

Table 3.1 - Foundation risks, methods for evaluation and prevention

3.13.2 Where seabed obstructions such as pipelines and wellheads are known to be present, sufficient information to enable safe positioning of the jack-up is required. In some cases an ROV or diver's inspection may be required in addition to a sidescan sonar survey.

3.13.3 Seabed surface surveys can become out-of-date, particularly in areas of construction/drilling activity or areas with mobile sediments. Good judgment should be used regarding the applicability of all surveys, especially with regard to validity. In open locations the maximum period for the validity of seabed surveys for debris and mobile sediment conditions should be determined taking account of local conditions. For locations close to existing installations seabed surveys for debris and sediment conditions should, subject to practical considerations, be undertaken immediately prior to the arrival of the jack-up at the location.

3.14 <u>Geophysical Investigation - Shallow Seismic Survey</u>

3.14.1 The principal objectives of the shallow seismic survey are:

- To determine near surface soil stratigraphy. This requires correlation of the seismic data with (existing) soil boring data in the vicinity.

- To reveal the presence of shallow gas concentrations.

Due to the 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.

3.14.2 A shallow seismic survey should be performed over an approximately 1 kilometer square area centered on the location. Line spacing of the survey should typically not be greater than 100 meters x 250 meters over the survey area. Equipment should normally be capable of giving detailed data to a depth equal to the greater of 30 meters or the anticipated footing penetration plus 1.5 to 2 times the footing diameter. Further guidance on seismic surveys is given in reference [2].

3.14.3 The survey report should include at least two vertical cross-sections passing through the location showing all relevant reflectors and allied geological information. The equipment used should be capable of identifying reflectors of 0.5m and thicker.

3.15 <u>Surface Soil Samples</u>

The site investigation should be sufficient to identify the character of the soil surface and allow evaluation of the possibility of scour occurring. (See Commentary to Section 6.4.3)

3.16 Geotechnical Investigations

3.16.1 Site specific geotechnical testing is recommended in areas where any of the following apply:

- the shallow seismic survey cannot be interpreted with any certainty,
- significant layering of the strata is indicated,
- the location is in a new operating area,
- the area is known to be potentially hazardous.

3.16.2 A geotechnical investigation should comprise a minimum of one borehole to a depth equal to 30 meters or the anticipated footing penetration plus 1.5 to 2 times the footing diameter, whichever is the greater. All the layers should be adequately investigated and the transition zones cored at a sufficient sampling rate.

The number of boreholes required should account for the lateral variability of the soil conditions, regional experience and the geophysical investigation. When a single borehole is made, the preferred location is at the center of the leg pattern at the intended location.

3.16.3 "Undisturbed" soil sampling and laboratory testing and/or in-situ cone penetrometer testing may be conducted. Other recognized types of in-situ soil testing may be appropriate such as vane shear and/or pressure meter tests.

3.16.4 The geotechnical report should include borehole logs, cone penetrometer records (if appropriate) and documentation of all laboratory tests, together with interpreted soil design parameters. Design parameters should be selected by a competent person. For the methods recommended in Section 6, the design parameters should include profiles of undrained shear strength and/or effective stress parameters, soil indices (plasticity, liquidity, grain size, etc.), relative density, unit weight and, where applicable, the over consolidation ratio (OCR).

Additional soil testing to provide shear moduli and cyclic/dynamic behavior may be required if more comprehensive analysis are to be applied or where the soil strength may deteriorate under cyclic loading.

3 GLOSSARY OF TERMS - ASSESSMENT INPUT DATA

C	=	Constant in expression for $F(\alpha)$.
d	=	Water depth.
f	=	Wave frequency.
$F(\alpha)$	=	Directionality function
	=	$C.Cos^{2n}\alpha$
h	=	Reference depth for wind driven current.
	=	5.0 m in the absence of other data.
HAT	=	Water depth at highest astronomical tide.
H_{det}	=	Reduced wave height which may be used for deterministic wave force calculations, allowing for the conservatisms of higher order wave theories.
	=	$1.60\ H_{srp}$
H_{max}	=	The individual extreme wave height for a given return period defined as the wave height with an annual probability of exceedence of 1/return period (e.g. the 50 year return period H_m has a 2% annual probability of exceedence). Where local data is not available: $H_{max} = 1.86\ H_{srp}$ (for non-tropic revolving storm areas), $H_{max} = 1.75\ H_{srp}$ (for tropical revolving storm areas.) When H_{max} is used for airgap calculations the minimum return period for H_{srp} is recommended as 50 years, even if a lower return period is used for other purposes.
H_s	=	Significant wave height (meters), including depth/asymmetry correction, according to Section 3.5.1.1.
H_{srp}	=	The assessment return period significant wave height for a three hour storm.
$I_0(\gamma)$	=	Parameter depending on γ used in the expression for $S_{\eta\eta}(f)$.
LAT	=	Water depth at lowest astronomical tide.
MHWS	=	Height of mean high water spring tide above LAT.
MLWS	=	Height of mean low water spring tide above LAT.
MWL	=	Mean water level related to the seabed.
n	=	Power constant in expression for $F(\alpha)$.
	=	2 or 4.
q	=	Exponent in expression for $S_{\eta\eta}(f)$.
	=	$\exp(-(T_pf-1)^2/2\sigma^2)$
$S_{\eta\eta}(f)$	=	Power density spectrum of long crested wave surface elevation as a function of frequency, f.
	=	$(16I_0(\gamma))^{-1}H_s^2T_p(T_pf)^{-5}\exp(-1.25/(T_pf)^4)\gamma^q$
$S_{\eta\eta}(f,\alpha)$	=	Power density spectrum of short-crested wave surface elevation as a function of frequency, f.
	=	$S_{\eta\eta}(f).F(\alpha)$
SWL	=	Height of extreme still water level above LAT.
	=	MHWS + 50 year storm surge.
	=	MLWS + 50 year negative storm surge (if more onerous).
T_{ass}	=	Wave period associated with H_{max} (also used with H_{det}).
T_p	=	Peak period associated with H_{srp} (also used with H_s).
T_z	=	Zero-upcrossing period associated with H_{srp} (also used with H_s).
V_C	=	Current velocity as a function of z.
V_s	=	Downwind component of surge current.
V_t	=	Downwind component of mean spring tidal current.
V_w	=	Wind generated surface current.
	=	2.6% of 1 minute sustained wind velocity at 10m, in the absence of other data.
z	=	Distance above still water level used in determination of V_C.

3 GLOSSARY OF TERMS - ASSESSMENT INPUT DATA (Continued)

α	=	Angle between direction of elementary wave trains and dominant direction of short-crested waves.
γ	=	Peak enhancement factor used in expression for $S_{\eta\eta}(f)$. For JONSWAP spectrum varies between 1 and 7 with a most probable average value of 3.3.
σ	=	Constant in expression for q
	=	0.07 for $T_p f <= 1$
	=	0.09 for $T_p f > 1$

4 CALCULATION METHODS - HYDRODYNAMIC AND WIND FORCES

4.1 Introduction

4.1.1 The models, methods and coefficients given in this Section are matched to represent a consistent method such that the whole Section should be considered together. No force coefficients should be used unless they correspond to a particular stated analysis method.

4.1.2 The environmental forces may be determined according to the recommendations of this Section based on the dimensions of the members and the environmental criteria as described in Section 3 (wind speed, wave height and period and current velocity and profile).

4.1.3 Since differences in shape, proportions and even detail can result in considerable differences in the resultant forces, rational data from model testing may be used by the assessor at his discretion subject to the conditions of Section 4.7.6.

4.2 Wind Force Calculations

4.2.1 For wind load application according to Section 5.7.2, the wind force for each component (divided into blocks of not more than 15m vertical extent), F_{Wi}, may be computed using the formula:

$$F_{Wi} = P_i A_{Wi}$$

where;

P_i = the pressure at the center of the block.

A_{Wi} = the projected area of the block considered.

and the pressure P_i shall be computed using the formula:

$$P_i = 0.5 \rho (V_{ref})^2 C_h C_s$$

where;

ρ = density of air (to be taken as 1.2224 kg/m^3 unless an alternative value can be justified for the location).

V_{ref} = the 1 minute sustained wind velocity at reference elevation (normally 10m above MWL), see Section 3.4.1.

C_h = height coefficient, as given in Section 4.2.2.

C_s = shape coefficient, as given in Section 4.2.3.

Note:
The wind area of the hull and associated structures (excluding derrick and legs) may normally be taken as the profile area viewed from the direction under consideration.

4.2.2 C_h may be derived from the wind velocity profile;

$$V_Z = V_{ref} (Z/Z_{ref})^{1/N}$$

where;

V_Z = the wind velocity at elevation Z.

V_{ref} = the 1 minute sustained wind velocity at elevation Z_{ref} (normally 10m above MWL), see Section 3.4.1.

N = 10 unless site specific data indicate that an alternative value of N is appropriate.

Hence:

$$C_h = (V_Z/V_{ref})^2 = (Z/Z_{ref})^{2/N}, \text{ but always} \geq 1.0$$

Alternatively, the approximate coefficients shown in Table 4.1 may be applied. The height is the vertical distance from the still water surface to the center of area of the block considered. Blocks which have a vertical dimension greater than 15 m shall be sub-divided, and the appropriate height coefficients applied to each part of the block.

Height m	Height coefficient C_h
0 - 15	1.00
15 - 30	1.18
30 - 45	1.30
45 - 60	1.39
60 - 75	1.47
75 - 90	1.53
90 - 105	1.58
105 - 120	1.62
120 - 135	1.66
135 - 150	1.70
150 - 165	1.74
165 - 180	1.77
180 - 195	1.80

Table 4.1 - Height coefficients

In deriving Table 4.1 the wind velocity used to obtain C_h for the block below 15.0m is the V_{ref} value. For all other blocks the C_h value is that for the mid-height of the block. When using Table 4.1 the wind velocity is derived from Section 3.4.1 for a reference height of 10m above the still water.

4.2.3 Shape coefficients shall be derived from Table 4.2;

Type of member or structure	Shape coefficient C_s
Hull side, (flat side)	1.0, based on total projected area
Deckhouses, jack-frame structure, sub-structure, draw-works house, and other above-deck blocks	1.1, based on the total projected area (i.e. the area enclosed by the extreme contours of the structure)
Leg sections projecting above jack-frame structure and below the hull	C_s = C_{De} as determined from Section 4.6, except that marine growth may be omitted. A_{Wi} determined from D_e and section length.
Isolated tubulars (crane pedestals, etc.)	0.5
Isolated structural shapes (angles, channels, box, I-sections)	1.5, based on member projected area
Derricks, crane booms, flare towers (open lattice sections only, not boxed- in sections)	The appropriate shape coefficient for the members concerned applied to 50% of the total projected profile area of the item (25% from each of the front and back faces)
Shapes or combinations of shapes which do not readily fall into the above categories will be subject to special consideration	

Table 4.2 - Shape coefficients

4.3 Hydrodynamic Forces

4.3.1 Wave and current forces on slender members having cross sectional dimensions sufficiently small compared with the wave length should be calculated using Morison's equation. Note: Morison's equation is normally applicable providing:

λ > $5D_i$ where;
λ = wavelength and
D_i = reference dimension of member (e.g. tubular diameter)

Morison's equation specifies the force per unit length as the vector sum:

$$\Delta F = \Delta F_{drag} + \Delta F_{inertia} = 0.5\ \rho\ D\ C_D\ v_n\ |v_n| + \rho\ C_M\ A\ \dot{u}_n$$

where the terms of the equation are described in the following.

4.3.2 To obtain the drag force, the appropriate drag coefficient (C_D) is to be chosen in combination with a reference diameter, including any required additions for marine growth, as described in Section 4.7.

The Morison's drag force formulation is:

$$\Delta F_{drag} = 0.5 \rho C_D D v_n |v_n|$$

where;

ΔF_{drag} = drag force (per unit length) normal to the axis of the member considered in the analysis and in the direction of v_n.
ρ = mass density of water (normally 1025 kg/m^3).
C_D = drag coefficient (= C_{Di} or C_{De} from Section 4.6-7).
v_n = relative fluid particle velocity resolved normal to the member axis.
D = the reference dimension in a plane normal to the fluid velocity v_n (= D_i or D_e from Section 4.6-7).

<u>Note:</u> The relative fluid particle velocity, v_n, may be taken as:

$$v_n = u_n + V_{Cn} - \alpha \dot{r}_n$$

where;

$u_n + V_{Cn}$ = the combined particle velocity found as the vectorial sum of the wave particle velocity and the current velocity, normal to the member axis.
$\dot{r}_n$ = the velocity of the considered member, normal to the member axis and in the direction of the combined particle velocity.
α = 0, if an absolute velocity is to be applied, i.e. neglecting the structural velocity.
= 1, if relative velocity is to be included. May only be used for stochastic/random wave force analyses if:

$$uT_n/D_i \geq 20$$

where u = particle velocity = $V_C + \pi H_s/T_z$
T_n = first natural period of surge or sway motion
and D_i = the reference diameter of a chord.

<u>Note:</u>
See also Section 7.3.7 for relevant damping coefficients depending on α.

4.3.3 To obtain the inertia force, the appropriate inertia coefficient (C_M) is to be taken in combination with the cross sectional area of the geometric profile, including any required additions for marine growth, as described in Section 4.7.

The Morison's inertia force formulation is:

$$\Delta F_{inertia} = \rho C_M A \dot{u}_n$$

where;

$\Delta F_{inertia}$ = inertia force (per unit length) normal to the member axis and in the direction of $\dot{u}_n$.
ρ = mass density of water (normally 1025 kg/m^3).
C_M = inertia coefficient.
A = cross sectional area of member (= A_i or A_e from Section 4.6)
$\dot{u}_n$ = fluid particle acceleration normal to member.

4.4 Wave Theories and Analysis Methods

4.4.1 For deterministic analyses an appropriate wave theory for the water depth, wave height and period shall be used, based on the curves shown in Figure 4.1, after HSE [3]. For practical purposes, an appropriate order of Dean's Stream Function or Stokes' 5th (within its bounds of applicability) is acceptable for regular wave survival analysis.

4.4.2 For random wave (stochastic) analyses, it is recommended that the random seastate is generated from the summation of at least 200 component Linear (Airy) waves of height and frequency determined to match the required wave spectrum. The phasing of the component waves should be selected at random.

The extrapolation of the wave kinematics to the free surface is most appropriately carried out by substituting the true elevation at which the kinematics are required with one which is at the same proportion of the still water depth as the true elevation is of the instantaneous water depth. This can be expressed as follows:

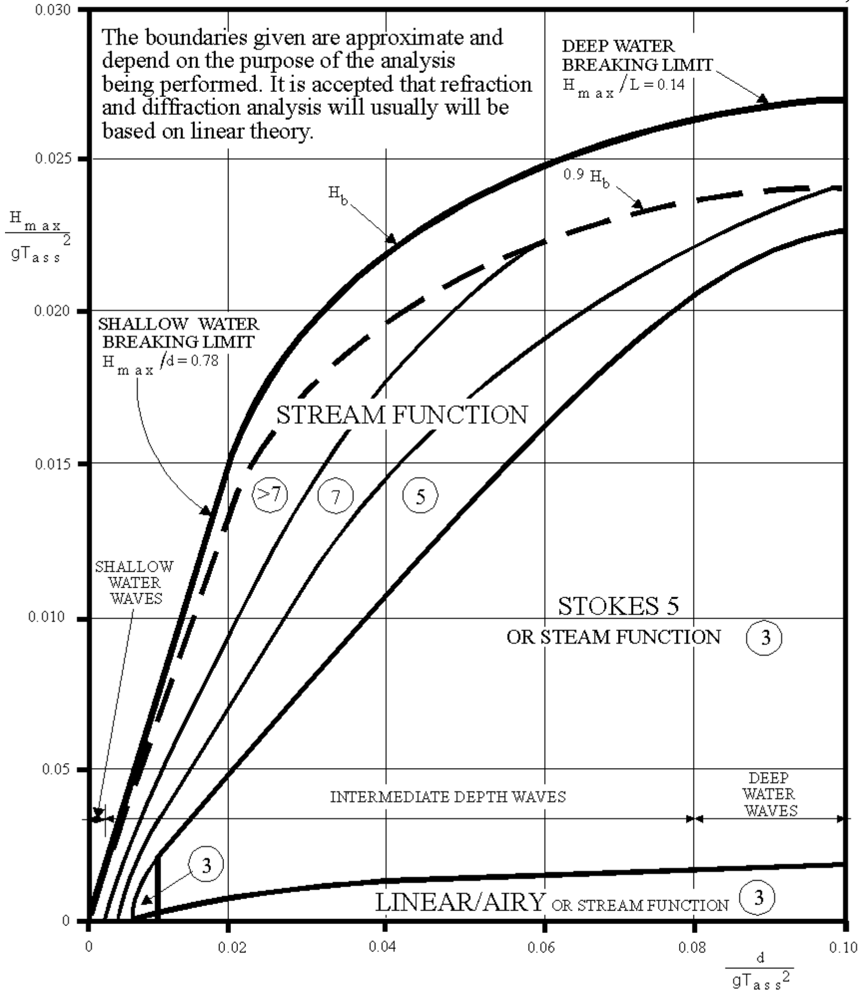
$$z' = \frac{z - \zeta}{1 + \zeta / d}$$

where;

z'	=	The modified coordinate to be used in particle velocity formulation
z	=	The elevation at which the kinematics are required (coordinate measured vertically upward from the still water surface)
ζ	=	The instantaneous water level (same axis system as z)
d	=	The still, or undisturbed water depth (positive).

This method ensures that the kinematics at the surface are always evaluated from the linear wave theory expressions as if they were at the still water level, Wheeler (1969) [4] (see Figure C4.4.2 in the Commentary).

4.4.3 If breaking waves are specified according to Figure 4.1, it is recommended that the wave period is changed to comply with the breaking limit for the specified height.



Notes

1) None of these theories is theoretically correct at the breaking limit.
2) Wave theories intended for limiting height waves should be referenced for waves higher than $0.9H_b$ when stream function theory may underestimate the kinematics.
3) Stream function theory is satisfactory for wave loading calculations over the remaining range of regular waves. However, stream function programs may not produce a solution when applied to near breaking waves or deep water waves
4) The order of stream function theory likely to be satisfactory is circled. Any solution obtained should be checked by comparison with the results of a higher order solution.
5) The error involved in using Airy theory outside its range of applicability is discussed in the background document.

Nomenclature

H_{max}/gT_{ass}^2	=	Dimensionless wave steepness
d/gT_{ass}^2	=	Dimensionless relative depth
H_{max}	=	Wave height (crest to trough)
H_b	=	Breaking wave height
d	=	Mean water depth
T_{ass}	=	Wave period
L	=	Wave length (distance between crests)
g	=	Acceleration due to gravity

Figure 4.1 - Range and validity of different wave theories for regular waves, (after HSE [3])

4.5 Current

4.5.1 The current velocity and profile as specified in Section 3.6 shall be used. Interpolation between the data points may be required and linear interpolation is recommended for simplicity.

4.5.2 The current induced drag forces are to be determined in combination with the wave forces. This is to be carried out by the vectorial addition of the wave and current induced particle velocities prior to the drag force calculations.

4.5.3 The current may be reduced due to interference from the structure on the flow field of the current, Taylor [5]. The current may be reduced as follows (see Commentary):

$$V_C = V_f\,[1 + C_{De}D_e/(4D_1)]^{-1}$$

where;

V_C = the current velocity to be used in the hydrodynamic model, V_C should be not taken as less than $0.7V_f$.
V_f = the far field (undisturbed) current.
C_{De} = equivalent drag coefficient, as defined in 4.6.5.
D_e = equivalent diameter, as defined in 4.6.5.
D_1 = face width of leg, outside dimensions.

4.6 Leg Hydrodynamic Model

4.6.1 The hydrodynamic modeling of the jack-up leg may be carried out by utilizing 'detailed' or 'equivalent' techniques. In both cases the geometric modeling procedure corresponds to the respective modeling techniques described in Section 5.6.4. The hydrodynamic properties are then found as described below:

'Detailed' model
All relevant members are modeled with their own unique descriptions for the Morison term values with the correct orientation to determine v_n and $\dot{u}_n$ and the corresponding $C_D D = C_{Di}D_i$ and $C_M A = C_{Mi}\pi D_i^2/4$, as defined in Section 4.7.

'Equivalent' model
The hydrodynamic model of a bay is comprised of one, 'equivalent' vertical tubular located at the geometric center of the actual leg. The corresponding (horizontal) v_n and $\dot{u}_n$ are applied together with equivalent $C_D D = C_{De}D_e$ and $C_M A = C_{Me}A_e$, as defined in Sections 4.6.5 and 4.6.6. The model should be varied with elevation, as necessary, to account for changes in dimensions, marine growth thickness, etc.

Note:
The drag properties of some chords will differ for flow in the direction of the wave propagation (wave crest) and for flow back towards the source of the waves (wave trough). Often the combined drag properties of all the chords on a leg will give a total which is independent of the flow direction along a particular axis. When this is not the case it is recommended that the effect is included directly in the wave-current loading model. If this is not possible it is recommended that:

1. Regular wave deterministic calculations use a value appropriate to the flow direction under consideration, noting that the flow direction is that of the combined wave and current particle motion.

2. An average drag property is considered for random wave analyses which are solely used to determine dynamic effects for inclusion in a final regular wave deterministic calculation which will be made on the basis of 1. above.

3. The drag property in the direction of wave propagation is used for random wave analyses from which the final results are obtained directly.

4.6.2 Lengths of members are normally taken as the node-to-node distance of the members in order to account for small non-structural items (e.g. anodes, jetting lines of less than 4" nominal diameter). Large non-structural items such as raw water pipes and ladders are to be included in the model. Free standing conductor pipes and raw water towers are to be considered separately from the leg hydrodynamic model.

4.6.3 The contribution of the part of the spudcan above the seabed should be investigated and only excluded from the model if it is shown to be insignificant. In water depths greater than 2.5 H_s or where penetrations exceed 1/2 the spudcan height, the effect of the spudcan is normally insignificant.

4.6.4 For leg structural members, shielding and solidification effects should not normally be applied in calculating wave forces. The current flow is however reduced due to interference from the structure on the flow field, see Section 4.5.3.

4.6.5 When the hydrodynamic properties of a lattice leg are idealized by an 'equivalent' model description the model properties may be found using the method given below:

The equivalent value of the drag coefficient, C_{De}, times the equivalent diameter, D_e, to be used in Section 4.3.2 for C_{Dei} of the bay may be chosen as:

$$C_{De}\, D_e = D_e\, \Sigma\, C_{Dei}$$

The equivalent value of the drag coefficient for each member, C_{Dei}, is determined from:

$$C_{Dei} = [\,\sin^2\beta_i + \cos^2\beta_i \sin^2\alpha_i\,]^{3/2}\, C_{Di}\, \frac{D_i l_i}{D_e s}$$

where;

C_{Di} = drag coefficient of an individual member (i) as defined in Section 4.7.

D_i = reference diameter of member 'i' (including marine growth as applicable) as defined in Section 4.7.

D_e = equivalent diameter of leg, suggested as $\sqrt{(\Sigma D_i^2 l_i / s}$

l_i = length of member 'i' node to node center.

s = length of one bay, or part of bay considered.

α_i = angle between flow direction and member axis projected onto a horizontal plane.

β_i = angle defining the member inclination from horizontal (see Figure 4.2).

<u>Note:</u>

Σ indicates summation over all members in one leg bay

The above expression for C_{Dei} may be simplified for horizontal and vertical members as follows:

Vertical members (e.g. chords): $C_{Dei} = C_{Di} (D_i/D_e)$

Horizontal members: $C_{Dei} = \sin^3\alpha \, C_{Di} (D_i l_i/D_e s)$

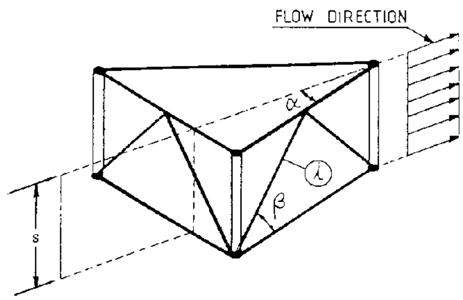


Figure 4.2: Flow angles appropriate to a lattice leg
(after DNV Class Note 31.5, February 1992, [6])

4.6.6 The equivalent value of the inertia coefficient, C_{Me}, and the equivalent area, A_e, to be used in Section 4.3.3, representing the bay may be chosen as:

C_{Me} = equivalent inertia coefficient which may normally be taken as 2.0 when using A_e

A_e = equivalent area of leg per unit height = $(\Sigma A_i l_i)/s$

A_i = equivalent area of element = $\pi D_i^2/4$

D_i = reference diameter chosen as defined in Section 4.7

For a more accurate model the C_{Me} coefficient may be determined as:

$$C_{Me} A_e = A_e \Sigma C_{Mei}$$

where;

$$C_{Mei} = [1 + (\sin^2\beta_i + \cos^2\beta_i \sin^2\alpha_i)(C_{Mi} - 1)] \frac{A_i l_i}{A_e s}$$

C_{Mi} = the inertia coefficient of an individual member, C_{Mi} is defined in Section 4.7 related to reference dimension D_i.

Note:
For dynamic modeling the added mass of fluid per unit height of leg may be determined as $\rho A_i(C_{mi} - 1)$ for a single member or $\rho A_e(C_{Me} - 1)$ for the equivalent model, provided that A_e is as defined above.

4.7 Hydrodynamic Coefficients for Leg Members

4.7.1 Hydrodynamic coefficients for leg members are given in this Section. Tubulars, brackets, split tube and triangular chords are considered. Hydrodynamic coefficients including directional dependence are given together with a fixed reference diameter D_i. No other diameter should be used unless the coefficients are scaled accordingly. Unless better information is available for the computation of wave and current forces, the values of drag and inertia coefficients applicable to Morison's equation should be obtained from this Section.

4.7.2 Recommended values for hydrodynamic coefficients for tubulars (<1.5m diameter) are given in Table 4.3 based on the data discussed in the commentary.

Surface condition		C_{Di}	C_{Mi}
Smooth	See Note	0.65	2.0
Rough		1.00	1.8

Table 4.3: Base hydrodynamic coefficients for tubulars

Note:
The smooth values will normally apply above MWL + 2m and the rough values below MWL + 2m, where MWL is as defined in Section 3.7.2. If the jack-up has operated in deeper water and the fouled legs are not cleaned the surface should be taken as rough for wave loads above MWL + 2m. See Commentary.

4.7.3 When applicable, marine growth is to be included in the hydrodynamic model by adding the appropriate marine growth thickness, to, on the boundary of each individual member below MWL + 2m where MWL is as defined in Section 3.7.2 i.e. for a tubular $D_i = D_{original} + 2t_m$. Site specific data for marine growth is preferred (see Section 3.9). If such data are not available all members below MWL + 2m shall be considered to have a marine growth thickness t_m = 12.5 mm (i.e. total of 25 mm across the diameter of a tubular member). Marine growth on the teeth of elevating racks and protruding guided surfaces of chords may normally be ignored.

The effects of marine growth may be ignored if anti-fouling, cleaning or other means are applied, however the surface roughness is still to be taken into account (see Commentary).

4.7.4 The in-line force due to gussets in any vertical plane shall be determined using a drag coefficient:

$$C_{Di} = 2.0$$

applied together with the projected area of the gusset visible in the flow direction, unless model test data shows otherwise. This drag coefficient may be applied together with a reference diameter D_i and corresponding length l_i chosen such that their product equals the plane area, $A = D_i l_i$ and $D_i = l_i$ (see Figure 4.3). In the equivalent model of Section 4.6 the gussets may then be treated as a horizontal element of length l_i , with its axis in the plane of the gusset. C_{Mi} should be taken as 1.0 and marine growth may be ignored.

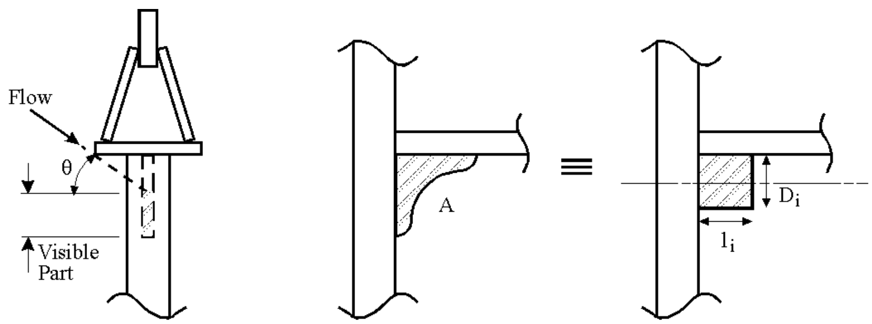


Figure 4.3: Gusset plates

4.7.5 For non-tubular geometries (e.g. leg chords) the appropriate hydrodynamic coefficients may, in lieu of more detailed information, be taken in accordance with Figures 4.4 or 4.5 and corresponding formulas, as appropriate.

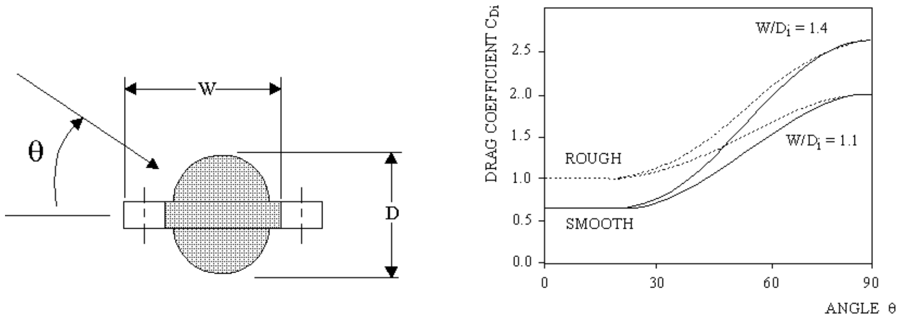


Figure 4.4: Split tube chord and typical values for C_{Di}

For a split tube chord as shown in Figure 4.4, the drag coefficient C_{Di} related to the reference dimension $D_i = D+2t_m$, the diameter of the tubular including marine growth as in Section 4.7.3, may be taken as:

$$C_{Di} = \begin{cases} C_{D0} & ; \quad 0° < \theta \le 20° \\ C_{D0} + (C_{D1} W / D_i - C_{D0}) \mathrm{Sin}^2[(\theta - 20°)9/7] & ; \quad 20° < \theta \le 90° \end{cases}$$

where;

θ = Angle in degrees, see Figure 4.4

C_{D0} = The drag coefficient for a tubular with appropriate roughness, see Section 4.7.2. (C_{D0} = 1.0 below MWL+2m and C_{D0} =0.65 above MWL+2m.)

C_{D1} = The drag coefficient for flow normal to the rack (θ = 90°), related to projected diameter, W. C_{D1} is given by:

$$C_{D1} = \begin{cases} 1.8 & ; \quad W/D_i < 1.2 \\ 1.4 + \tfrac{1}{3}(W/D_i) & ; \quad 1.2 < W/D_i < 1.8 \\ 2.0 & ; \quad 1.8 < W/D_i \end{cases}$$

The inertia coefficient C_{Mi} = 2.0, related to the equivalent volume $\pi D_i^2/4$ per unit length of member, may be applied for all heading angles and any roughness.

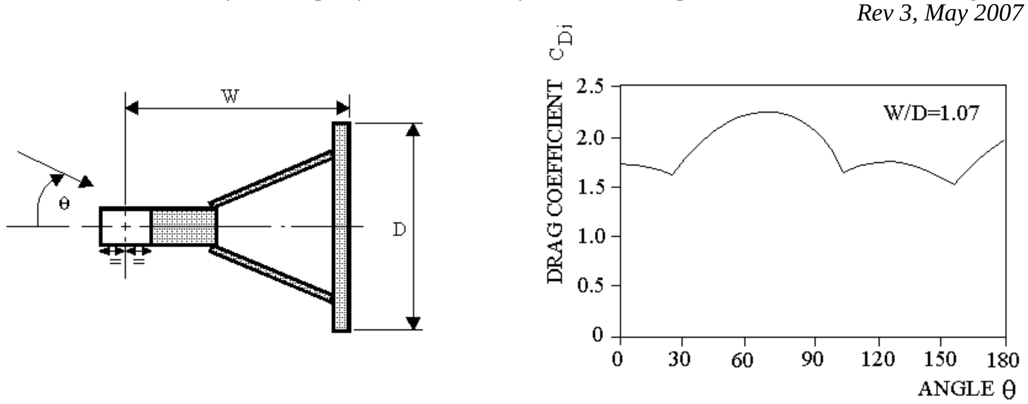


Figure 4.5: Triangular chord and typical values of C_{Di}

For a triangular chord as shown in Figure 4.5, the drag coefficient C_{Di} related to the reference dimension $D_i = D$, the backplate width, may be taken as:

$$C_{Di} = C_{Dpr}(\theta)\, D_{pr}(\theta) / D_i$$

where the drag coefficient related to the projected diameter, C_{Dpr}, is determined from:

$$C_{Dpr} = \begin{cases} 1.70 & ; \quad \theta = 0° \\ 1.95 & ; \quad \theta = 90° \\ 1.40 & ; \quad \theta = 105° \\ 1.65 & ; \quad \theta = 180° - \theta_o \\ 2.00 & ; \quad \theta = 180° \end{cases}$$

Linear interpolation is to be applied for intermediate headings. The projected diameter, $D_{pr}(\theta)$, may be determined from:

$$Dpr(\theta) = \begin{cases} D\cos(\theta) & ; \quad 0 < \theta < \theta_o \\ W\sin(\theta) + 0.5D|\cos(\theta)| & ; \quad \theta_o < \theta < 180 - \theta_o \\ D|\cos(\theta)| & ; \quad 180 - \theta_o < \theta < 180 \end{cases}$$

The angle θ_o, where half the rackplate is hidden, $\theta_o = \tan^{-1}(D/(2W))$.

The inertia coefficient $C_{Mi} = 2.0$ (as for a flat plate), related to the equivalent volume of $\pi D_i^2/4$ per unit length of member, may be applied for all headings and any roughness.

4.7.6 Shapes, combinations of shapes or closely grouped non-structural items which do not readily fall into the above categories should be assessed from relevant literature (references to be provided) and/or appropriate interpretation of (model) tests. The model tests should consider possible roughness, Keulegan-Carpenter and Reynolds number dependence.

4.8 Other Considerations

Local load effects will normally have been addressed at the design stage. Should the wind or current and/or wave height parameters at the location exceed those applicable at the design stage further consideration may be required. The Commentary provides further details and references to calculation methods.

4 GLOSSARY OF TERMS - CALCULATION METHODS HYDRODYNAMICS AND WIND FORCES

A_{Wi} = Projected area of the block considered in wind computations.
A = Cross sectional area of member.
A_e = Equivalent area of leg per unit height = $(\sum A_i 1_i)/s$.
A_i = Equivalent area of element = $\pi D_i^2/4$.
C_D = Drag coefficient.
C_{De} = Equivalent drag coefficient.
C_{Di} = Drag coefficient of an individual member, related to D_i.
C_{D0} = The drag coefficient for chord at direction $\theta = 0°$.
C_{D1} = The drag coefficient for flow normal to the rack, $\theta = 90°$.
C_{Dpr} = The drag coefficient related to the projected diameter.
C_M = Inertia coefficient.
C_{Me} = Equivalent inertia coefficient.
C_{Mi} = Inertia coefficient of a member, related to D_i.
C_h = Height coefficient for wind.
C_s = Shape coefficient for wind related to projected area.
d = The mean, undisturbed water depth (positive).
D = Member diameter or backplate width.
D_e = Equivalent diameter of leg.
D_i = Reference dimension of individual leg members.
D_1 = Face width of leg, outside dimensions.
D_{pr} = The projected diameter.
F_{Wi} = Wind force for block i.
H_s = The effective significant wave height (Section 5.5.1.3).
l_i = Length of member 'i' node to node center.
P_i = Wind pressure at the center of block i.
$\dot{r}_n$ = Velocity of the considered member, normal to the member axis and in the direction of the combined particle velocity.
s = Length of one bay, or part of bay considered.
t_m = Marine growth thickness.
T_n = First natural period of sway motion.
T_z = The zero-upcrossing period associated with H_s.
u = Wave particle velocity.
u_n = Wave (only) particle velocity normal to the member.
$\dot{u}_n$ = Wave particle acceleration normal to the member.
v_n = Total (relative) flow velocity normal to the member.
V_{Cn} = Current velocity to be used in the hydrodynamic model, normal to member.
V_f = Far field (undisturbed) current.
V_{ref} = One minute sustained wind velocity at elevation Z_{ref}.
V_Z = Wind velocity at elevation Z.
W = Dimension from backplate to pitch point of triangular chord or dimension from root of one rack to tip of other rack of split-tubular chord.
z = Coordinate measured vertically upward from the mean water surface.
z' = Modified coordinate to be used in particle velocity formulation.
Z = Elevation measured from the mean water surface.
Z_{ref} = Reference elevation for wind speed.
α = Indicator for relative velocity, 0 or 1.
α_i = Angle defining flow direction relative to member.
β_i = Angle defining the member inclination.
ΔF_{drag} = Drag force per unit length.
$\Delta F_{inertia}$ = Inertia force per unit length.

5 GLOSSARY OF TERMS - CALCULATION METHODS HYDRODYNAMICS AND WIND FORCES (Continued)

ζ	=	The instantaneous water surface elevation (same axis system as z).
ρ	=	Mass density of water or air.
θ	=	Angle in degrees of water particle velocity relative to the chord orientation.
θ_o	=	Angle at which half rackplate of Δ chord is hidden = $\tan^{-1}$ (D/(2W))
λ	=	Wave length.

5 CALCULATION METHODS - STRUCTURAL ENGINEERING

5.1 General Conditions

5.1.1 Structural calculations should be carried out in accordance with the following sections.

5.1.2 A range of environmental approach directions and storm water levels should be considered, such that the most onerous (i.e. that leading to the extreme maximum and/or minimum loading) is determined for each assessment check {strength of each major type of element (chord, brace, etc.), overturning stability, foundation capacity, horizontal deflections, holding system, etc.}.

5.1.3 In deterministic calculations the most critical wave phase position(s) should be considered for each case identified under 5.1.2. Normally the phase giving maximum base shear and/or overturning moment will be found critical for overturning, leeward leg stresses, leeward leg foundations and windward leg foundations.

5.1.4 For fatigue calculations it may be necessary to determine the load or stress ranges, and hence other phase positions may also need to be considered.

5.2 Seabed Reaction Point

For independent leg jack-up units, the reaction point for horizontal and vertical loads at each footing shall be situated on the geometric vertical axis of the leg/spudcan, at a distance above the spudcan tip equivalent to:

a) Half the maximum predicted penetration (when spudcan is partially penetrated), or

b) Half the height of the spudcan (when the spudcan is fully, more than fully penetrated).

If detailed information exists regarding the soils and spudcan the position of the reaction point may be calculated. (Brekke et al, [7])

5.3 Foundation Fixity

5.3.1 For analyses of an independent leg jack-up unit under extreme storm conditions the foundations may normally be assumed to behave as pin joints, and so are unable to sustain a bending moment. Analysis and practical experience suggest that this may be a conservative approach for bending moment in the upper parts of the leg in way of the lower guides.

5.3.2 In cases where the inclusion of rotational foundation fixity is justified and is included in the structural analysis, it is essential that the nonlinear soil-structure interaction effects are properly taken into account. The model should include the interaction of rotational, lateral and vertical soil forces.

5.3.3 Methods of establishing the degree of fixity of rotational restraint, or fixity, at the footings are discussed further in Section 6.3.4 and the Commentary to Section 6. Upper or lower bound values should be considered as appropriate for the areas of the structure under consideration.

5.3.4 For checking the spudcans, the leg-to-can connection and the lower parts of the leg, appropriate calculations considering soil-structure interaction shall be carried out to determine the upper bound can moment. These areas may be checked assuming that a percentage of the maximum storm leg moment at the lower guide (derived assuming a pinned footing) is applied to the spudcan together with the associated horizontal and vertical loads. This percentage would normally be not less than 50%. For such simplified checks the loading on the spudcan may be modeled assuming that the soil is linear-elastic and incapable of taking tension.

5.4 Leg Inclination

The effects of initial leg inclination should be considered. Leg inclination may occur due to leg-hull clearances and the hull inclination permitted by the operating manual. Thus the total horizontal offset due to leg inclination, O_T, may be determined as:

$$O_T = O_1 + O_2$$

where;

O_T = Total horizontal offset of leg base with respect to hull.

O_1 = Offset due to leg-hull clearances.

O_2 = Offset due to maximum hull inclination permitted by the operating manual.

If detailed information is not available, O_T should be taken as 0.5% of the leg length below the lower guide.

The effects of leg inclination need be accounted for only in structural strength checks. This will normally be accomplished by increasing the effective moment in the leg at the lower guide by an amount equal to the offset O_T times the factored vertical reaction at the leg base due to dead, live, environmental, inertial and P-Δ loads.

5.5 P-Δ Effects

5.5.1 The P-Δ Effect occurs because the jack-up is a relatively flexible structure and is subject to lateral displacement of the hull (sidesway) under the action of environmental loads. As a result of the hull translation the line of action of the vertical spudcan reaction no longer passes through the centroid of the leg at the level of the hull. Consequently the leg moments at the level of the hull are increased over those arising from a linear quasi- static analysis by an amount equal to the individual leg load P times the hull translation, D.

This additional moment will cause additional deflection over that predicted by standard linear-elastic theory. The increased deflection is a function of the ratio of the applied axial load to the Euler load.

Furthermore the shift in the hull center of gravity due to the hull translation will increase the overturning moment (or decrease the righting moment). Consequently the axial loads in the leeward leg(s) will increase and the axial loads in the windward leg(s) will reduce.

The consequences of the above are:

a) Increased hull deflections (which will increase the linear-elastic P-Δ moments).

b) A redistribution of base shears (in global axes) such that the increase in lower guide moment is reduced in the leeward leg(s) and increased in the windward leg(s).

5.5.2 An analysis using a standard linear elastic (small displacement) finite element program will not allow for these effects. The following Sections describe techniques which may be used to account for the P-Δ/Euler effects. The large displacement methods are the most accurate, but require more rigorous analysis. The geometric stiffness methods are simpler and generally of sufficient accuracy.

5.5.3 *Large displacement methods;*

These methods are part of a number of finite element (F.E.) programs. In such methods the non-linear (large-displacement) solution is obtained by applying the load in increments and iteratively generating the stiffness matrix for the next load increment from the deflected shape (nodal deflections) of the previous increment. Some F.E. programs offer an intermediate solution in which the deflected geometry from an initial linear- elastic solution is used as the input to the final 'corrected' solution.

5.5.4 *Geometric stiffness methods;*

5.5.4.1 These methods are also available within a number of F.E. programs. A linear correction is made to the element stiffness matrix based on the axial load present in the element. Iteration is also required for this solution procedure.

5.5.4.2 A simplified geometric stiffness approach allows incorporation of P-Δ effects in a standard linear-elastic F.E. program without recourse to iteration (refer to Commentary for derivation). In this approach a correction term is introduced into the global stiffness matrix prior to analysis. When the analysis is complete the hull deflections, leg axial loads and leg bending moments will include the P-Δ effects. The derivation of the method is described in appendix C5.A of the Commentary.

The correction term is: $-P_g/L$

where;

P_g = Total effective gravity load on legs at hull. This includes the hull weight and weight of the legs above the hull.

L = The distance from the spudcan reaction point to the hull vertical center of gravity.

This single (negative) value is incorporated into the global stiffness matrix by attaching a pair of orthogonal horizontal translational earthed spring elements to a node representing the hull center of gravity and entering the negative value for each of the spring constants. Some F.E. packages allow direct matrix manipulation.

The negative stiffness term at the hull will produce an additional lateral force at the hull proportional to the structural deflection. The resulting (additional) base overturning moment will be equal to the gravity load times the hull displacement.

The additional lateral load (due to the negative stiffness term) will cause an over-prediction of the base shear (in global axes). Typically this is not critical. However, the base shear at each leg can be reduced by an amount equal to the difference between the total base shear and the shear due to the applied loads (both in global axes) divided by the number of legs.

5.5.4.3 An alternative geometric stiffness approach is given below. Here the P-Δ effects are determined by amplifying the linear-elastic displacement (excluding P-Δ) as follows:

$$\Delta = \delta_s / (1 - \frac{P}{P_E})$$

where;

Δ = the approximate displacement including P-Δ.
δ_s = the linear-elastic first order hull displacement.
P = the average axial load in the leg at the hull (i.e. the total leg load at the hull divided by the number of legs).
P_E = Euler buckling load of an individual leg.(See Section 7.3.5 for general formulation).

Corrections can then be made to a global linear-elastic solution by manually adding P-Δ moments to the results. The P-Δ moments are computed using the amplified deflection, Δ, and P's adjusted to account for this. (This approach is not strictly valid because it ignores the fact that the deflection of all the legs at the hull must be approximately equal. The imposition of this constraint will lead to a redistribution of the global base shear between the legs.) Ignoring the redistribution will generally be conservative for leeward leg(s) and their foundation loads and non- conservative for windward leg(s) and their foundation loads.

5.6 Structural Modeling

5.6.1 *Introduction*

It is important that the structural model accurately reflects the complex mechanism of the jack-up. For most jack-up configurations the load distribution at the leg-hull interface is not amenable to manual calculation, therefore, it is necessary to develop a Finite Element (F.E.) computer model. A number of different modeling techniques can be used to depict the jack-up structure. The recommended techniques are summarized below and their applicability and limitations are discussed in more detail in Section 5.6.3.

a) Fully detailed model of legs and hull/leg connections with detailed or representative stiffness model of hull and spudcan.

b) Simplified lower legs and spudcans, detailed upper legs and hull/leg connections with detailed or representative stiffness model of hull.

c) Equivalent stiffness model of legs and spudcans, equivalent hull/leg connection springs and representative beam-element hull grillage.

d) Detailed leg (or leg section) and hull/leg connection model.

Section 5.6.3 and Table 5.1 outline the limitations of the various modeling techniques and should be referenced to ensure that the selected models address all aspects required for a specific assessment.

5.6.2 *General Considerations*

In the elevated condition the most heavily loaded portion of the leg is normally between the upper and lower guides and in way of the lower guide. The stress levels in this area depend on the design concept of the jack-up. A specific jack-up design concept can be described by the combination of the following components (see Commentary Figure C5.5):

a) With or without fixation system,

b) Fixed or floating jacking system,

c) Opposed or unopposed pinions.

In units having fixation systems the transfer of moment between the leg and the hull is largely by means of a couple due to vertical loads carried from the chord into the fixation or jacking system.

Where a fixed or floating jacking system is fitted (and there is no fixation system) the transfer of moment between the leg and the hull is partly by means of a couple due to horizontal loads carried from the chords into the upper and lower guides. In this case and when the chord/guide contact occurs between bracing nodes significant local chord bending moments are normal.

If the jacking system has unopposed pinions local chord moments will arise due to:

- the horizontal pinion load component (due to the pressure angle of the rack/pinion).

- the vertical pinion load component acting at an offset from the chord neutral axis.

The modeling of the various design aspects is critical and recommended modeling techniques are outlined in the following sections. The Commentary provides detailed information regarding the combination of the above three components for current jack-up units.

5.6.3 *Applicability and Limitations*

It is most desirable to fully model the jack-up when assessing its structural strength. Very often assumptions and simplifications such as equivalent hull, equivalent leg, etc. will be made in the process of building the model. In view of this, various levels of modeling described in a) through d) below may be used. It should be noted that some of these methods may have limitations with respect to the accuracy of assessing the structural adequacy of a jack-up and when simplified models, such as those described in (c) and (d) are used it may be appropriate to calibrate against a more detailed model.

a) *Fully detailed 3-leg model*

The model consists of 'detailed legs', hull, hull/leg connections and spudcans modeled in accordance with 5.6.4(a),5.6.5, 5.6.6 and 5.6.7, respectively. The results from this model can be used to examine the preload requirements, overturning resistance, leg strength and the adequacy of the jacking system or fixation system.

b) *Combination leg 3-leg model*

The model consists of a combination of 'detailed leg' for the upper portion of legs and 'equivalent leg' for the lower portion of the legs modeled in accordance with 5.6.4. The hull, hull/leg connections and spudcans are modeled in accordance with 5.6.5, 5.6.6 and 5.6.7 respectively. The results from this model can be used to examine the preload requirements, overturning resistance, leg strength and the adequacy of the jacking system or fixation system.

c) *Equivalent 3-stick-leg model*

The model consists of 'equivalent legs' modeled in accordance with 5.6.4(b), hull structure modeled using beam elements in accordance with 5.6.5, leg to hull connections modeled in accordance with 5.6.6 and spudcans modeled as a stiff or rigid extension to the equivalent leg. The results from this model can be used to examine the preload requirements and overturning resistance. This model may also used to obtain the reactions at the spudcan or internal forces and moments in the leg at the vicinity of lower guide for application to the 'detailed leg' and hull/leg model (d) which should be used to assess the strength of the leg in the area between lower and upper guides.

d) *Single detailed leg model*

The model consists of a 'detailed leg' or a portion of a 'detailed leg' modeled in accordance with 5.6.4(a), the hull/leg connection modeled in accordance with 5.6.6 and, when required, the spudcan modeled in accordance with Section 5.6.7. This model is to be used in conjunction with the reactions at the spudcan or the forces and moments in the vicinity of lower guide obtained from Model (c). The results from this model can be used to examine the leg strength and the adequacy of the jacking system or the fixation system.

	Applicability (see Note 1)						
Model Type	I Global Loads	II Overturning Checks	III Foundation Checks	IV Global Leg Loads	V Leg Member Loads	VI Pinion/ Fixation System Loads	VII Hull Element Loads
a	Y	Y	Y	Y	Y	Y	2
b	Y	Y	Y	Y	Y	Y	2
c	Y	Y	Y	Y	-	-	-
d	-	-	-	-	Y	Y	-

Legend:
Y = Applicable
\- = Not applicable

Notes:
1. Large displacement and dynamic effects to be included where appropriate.
2. VII, hull stresses will only be available from more complex hull models.

Table 5.1 - Applicability of the suggested models

5.6.4 *Modeling the Leg*

The leg can be modeled as a 'detailed leg', an 'equivalent leg' or a combination of the two. The 'detailed leg' model consists of all structural members such as chords, horizontal, diagonal and internal braces of the leg structure and the spudcan (if required). The 'equivalent leg' model consists of a series of colinear beam elements (stick model) simulating the complete leg structure. It is recommended that the leg model(s) be generated in accordance with the following:

a) *'Detailed Leg' Model*

The coordinates of the joints for this model are to be defined by the intersection of the chord and brace centerlines. For joints where there is more than one brace, it is unlikely that there will be one (1) common point of intersection between the braces and chord. In this instance, it is usually sufficient to choose an intermediate point between the chord/brace centerline intersections. Gusset plates normally need not be included in the structural leg model, however their effects may be taken into account in the calculation of member and joint strength checks.

b) *'Equivalent Leg' Model*

The leg structure can be simulated by a series of colinear beams with the equivalent cross sectional properties calculated using the formulas indicated in Figure 5.1 or derived from the application of suitable 'unit' load cases (see Commentary C5.5) to the 'Detailed Leg' model described in 5.6.4 (a). Where such a model is used, detailed stresses, pinion loads, etc. will be derived either directly or indirectly from a 'detailed model'.

c) *'Combination Leg' model*

To facilitate obtaining detailed stress, pinion loads, etc. directly, a 'detailed leg' model can be generated covering the region between the guides, and extending at least 4 bays below and, where available, at least 4 bays above this region. The remainder is then modeled as an 'equivalent leg'. Care is required to ensure an appropriate interface and consistency of boundary conditions at the connections. The 'detailed leg'/'equivalent leg' connection should be modeled so that the plane of connection remains a plane after the leg is bent.

Note:

The leg stiffness used in the overall response analysis may account for a contribution from a portion of the rack tooth material. Unless detailed calculations indicate otherwise, the assumed effective area of the rack teeth should not exceed 10% of their maximum cross sectional area. When checking the capacity of the chords the chord properties should be determined discounting the rack teeth.

5.6.5 *Modeling the Hull*

The hull structure should be modeled so that the loads can be correctly transferred to the legs and the hull flexibility is represented accurately. Recommended methods are given below:

a) *Detailed Hull Model*

The model can be generated using plate elements in which appropriate directional modeling of the effect of the stiffeners on the plates should be included. The elements should be capable of carrying in-plane and, where applicable, out-of plane loads.

b) *Equivalent Hull Model*

Alternatively, the hull can be modeled by using a grillage of beams. Deck, bottom, side shell and bulkheads can be used to construct the grillage. The properties of the beam can be calculated based on the depth of the bulkheads, side-shell and the 'effective width' of the deck and bottom plating. Attention should be paid to the in-plane and torsional properties due to the continuity of the deck and bottom structures.

5.6.6 *Modeling the Hull/Leg Connection*

The hull/leg connection modeling is of extreme importance to the analysis since it controls the distribution of leg bending moments and shears carried between the upper and lower guide structures and the jacking or fixation system. It is therefore necessary that these systems are properly modeled in terms of stiffness, orientation and clearance. For the 'Equivalent 3-stick-leg model' a simplified derivation of the equivalent leg-hull connection stiffness may be applicable.

For jack-ups with a fixation system, the leg bending moment will be shared by the upper and lower guides, the jacking and the fixation systems. Normally the leg bending moment and axial force due to environmental loading are resisted largely by the fixation system because of its high rigidity. Depending on the specified method of operation, the stiffnesses, the initial clearances and the magnitude of the applied loading a portion of the environmental leg loading may be resisted by the jacking system and the guide structures. Typical shear force and bending moment diagrams for this configuration are shown in Figure 5.2.

For jack-ups without a fixation system, the leg bending moment will be shared by the jacking system and guide structure. For a fixed jacking system, the distribution of leg moment carried between the jacking system and guide structure mainly depends on the stiffness of the jacking pinions. Typical shear force and bending moment diagrams for this design are shown in Figures 5.3 and 5.4.

For a floating jacking system, the distribution of leg bending moment carried between the jacking system and guide structure depends on the combined stiffness of the shock pads and pinions. Typical shear force and bending moment diagrams for this design are shown in Figure 5.5.

The hull/leg connection should be modeled considering the effects of guide and support system clearances, wear, construction tolerances and backlash (within the gear-train and between the drive pinion and the rack).

The following techniques are recommended for modeling hull/leg connections (specific data for the various parts of the structure may be available from the designers data package):

<u>Detailed modeling</u>

a) <u>Upper and Lower Guides</u> - The guide structures should be modeled to restrain the chord member horizontally only in directions in which guide contact occurs. The upper and lower guides may be considered to be relatively stiff with respect to the adjacent structure, such as jackcase, etc. The nominal lower guide position relative to the leg may be derived using the sum of leg penetration, water depth and airgap. It is however recommended that at least two positions are covered when assessing leg strength: one at a node and the other at the midspan. This is to allow for uncertainties in the prediction of leg penetration and possible differences in penetration between the legs.

The finite lengths of the guides may be included in the modeling by means of a number of discrete restraint springs/connections to the hull. Care is required to ensure that such restraints carry loads only in directions/senses in which they can act. Alternatively the results from analyses ignoring the guide length may be corrected, if necessary, by modification of the local bending moment diagram to allow for the proper distribution of guide reaction, see Figure 5.6.

b) Jacking Pinions - The jacking pinions should be modeled based on the pinion stiffness specified by the manufacturer and should be modeled so that the pinions can resist vertical and the corresponding horizontal forces. A linear spring or cantilever beam can be used to simulate the jacking pinion. The force required to deflect the free end of the cantilever beam a unit distance should be equal to the jacking pinion stiffness specified by the manufacturer. The offset of the pinion/rack contact point from the chord neutral axis should be incorporated in the model.

c) Fixation System - The fixation system should be modeled to resist both vertical and horizontal forces based on the stiffness of the vertical and horizontal supports and on the relative location of their associated foundations. It is important that the model can simulate the local moment capacity of the fixation system arising from its finite size and the number and location of the supports.

d) Shock Pad - Floating jacking systems generally have two sets of shock pads at each jackcase, one located at the top and the other at the bottom of the jackhouse. Alternatively shock pads may be provided for each pinion. The jacking system is free to move up or down until it contacts the upper or lower shock pad. In the elevated condition, the jacking system is in contact with the upper shock pad and in the transit condition it is in contact with the lower shock pad. The stiffness of the shock pad should be based on the manufacturer's data and the shock pad should be modeled to resist vertical force only. It should also be noted that the shock pad stiffness characteristics may be nonlinear.

e) Jackcase and associated bracing - The jackcase and associated bracing should be modeled based on the actual stiffness since it has direct impact on the horizontal forces that the upper guide can resist.

Note:
Where the hull is not modeled it is normally suitable to earth the base of the jackcase and associated bracing, the foundations of the fixation system and the lower guide structures at their connections to the hull.

Simple modeling

f) For applications such as those described in Section 5.6.3 c) (Equivalent 3-stick-leg model) a simplified representation of the hull to leg connection is required. In this instance the rotational stiffness may be represented by rotational springs and, where applicable, horizontal and vertical stiffnesses by linear springs. Where these are derived from a more detailed modeling, as described above, it is important that suitable loading levels (typical of the cases to be analyzed) are selected so that the effects of clearances, etc. do not dominate the result. Hand calculations may also be applicable. See Section C5.5 in the Commentary.

5.6.7 *Modeling the Spudcan*

When modeling the spudcan, rigid beam elements are considered sufficient to achieve an accurate load transfer of the seabed reaction into the leg chords and bracing in the area between upper and lower guides. It should be noted that, due to the sudden change in stiffness, rigid beams can cause artificially high stresses at the leg to spudcan connections. Hence the modeling and selection of element type should be carefully considered when an accurate calculation of chord stresses is required in this area.

For a strength analysis of the spudcan and its connections to the leg it may be appropriate to develop a separate detailed model with appropriate boundary conditions.

5.7 Load Application

The assessment follows a partial factor format. The partial load factors are applied to loads as defined in other sections (i.e. they are load factors, NOT load-effect factors). The jack-up response is non-linear, and hence the application of the combined factored loads will not in general develop the same result as the factored combination of individual load effects.

For typical jack-up assessments, the time-varying nature of the wave loading will amplify the static responses and must be considered. The extreme response can be assessed either by a quasi-static analysis procedure (Section 7.2) including an inertial loadset (Section 7.3.6) or by a more detailed dynamic analysis procedure (Section 7.3.7). In the former case (quasi-static analysis including an inertial loadset), the load factors should be directly applied to the appropriate combinations of quasi-static environmental loading and inertial loadsets. In the latter case (detailed dynamic analysis), alternative methods can be used when acceptable rationale is provided.

The loads and load effects to be included in the analysis, with their designators used in Section 8 in ( ), comprise:

a) Self weight and non-varying loads (D), variable and drilling loads (L).

b) Wind loads (E).

c) Hydrodynamic wave-current loads (E).

d) Inertial loads due to dynamic response (D_n).

e) Second order effects (associated with D,L,E & D_n).

These are discussed in turn below.

5.7.1 *Self weight, variable and drilling loads*

Depending on the initial positions of the legs with respect to guide clearances, and the operation of the jacking and fixation systems (if fitted), the distributed hull loading and stiffness will lead to hull sagging which may impose bending moments on the legs which remain present for the remainder of the period on location. Such moments should be considered in the site assessment analyses, and will be larger in shallow waters where the leg extension below the hull is small and consequently the leg bending stiffness is higher.

To correctly capture these effects the hull loads should be applied to the model in such a manner as to represent their correct vertical and horizontal distribution. If dynamic analyses are to be performed all weights should be represented by means of masses together with vertical gravitational acceleration. It is generally appropriate to apply these masses by means of factored element self-weight with additional correction masses applied as necessary to obtain the correct total mass and center of gravity. Alternatively, it may be sufficient to apply point masses at the node points of the model.

It is noted that an F.E. model with distributed hull stiffness and loading will incorporate hull sag effects if the hull and variable gravity loading is 'turned on' with the unit defined in its initially undeflected shape at the operating airgap. It should be verified that the amount of hull sag moment arising is applicable, given the operating procedures pertaining to the unit. It may be necessary to apply corrections to the final results for any discrepancies in the hull sag induced loadings. Further guidance is given in Section 5.3.3 of the Commentary.

5.7.2 *Wind loads*

The wind loading on the legs above and below the hull may be applied as distributed or nodal loads. Where nodal loads are used a sufficient number of loads should be applied to reflect the distributed nature of the loading and it should be ensured that the correct total shear and overturning moment is applied on each leg. Similarly the wind loading on the hull and associated structure may be applied as distributed or nodal loads. The application should ensure the correct total shear and overturning moment is applied to the hull.

5.7.3 *Hydrodynamic wave-current loads*

The wave-current loading on the leg and spudcan structures above the mudline may be applied as distributed or nodal loads. Where nodal loads are used the application should ensure the correct total shear and overturning moment on each leg, and reflect the distributed nature of the loading.

5.7.4 *Inertial loads due to dynamic response*

When the dynamic approach (see Section 7) leads to the explicit determination of an inertial loadset, this should be applied to the hull model. In simpler dynamic approaches the inertial load may be represented by a single lateral point loading acting at the hull center of gravity, or by a number of point loads applied to other parts of the hull having the same line of action. In more complex approaches a more complete distributed load vector may be applied to the hull and legs.

5.7.5 *Second order effects*

Methods for including P-Δ effects are described in Section 5.5.

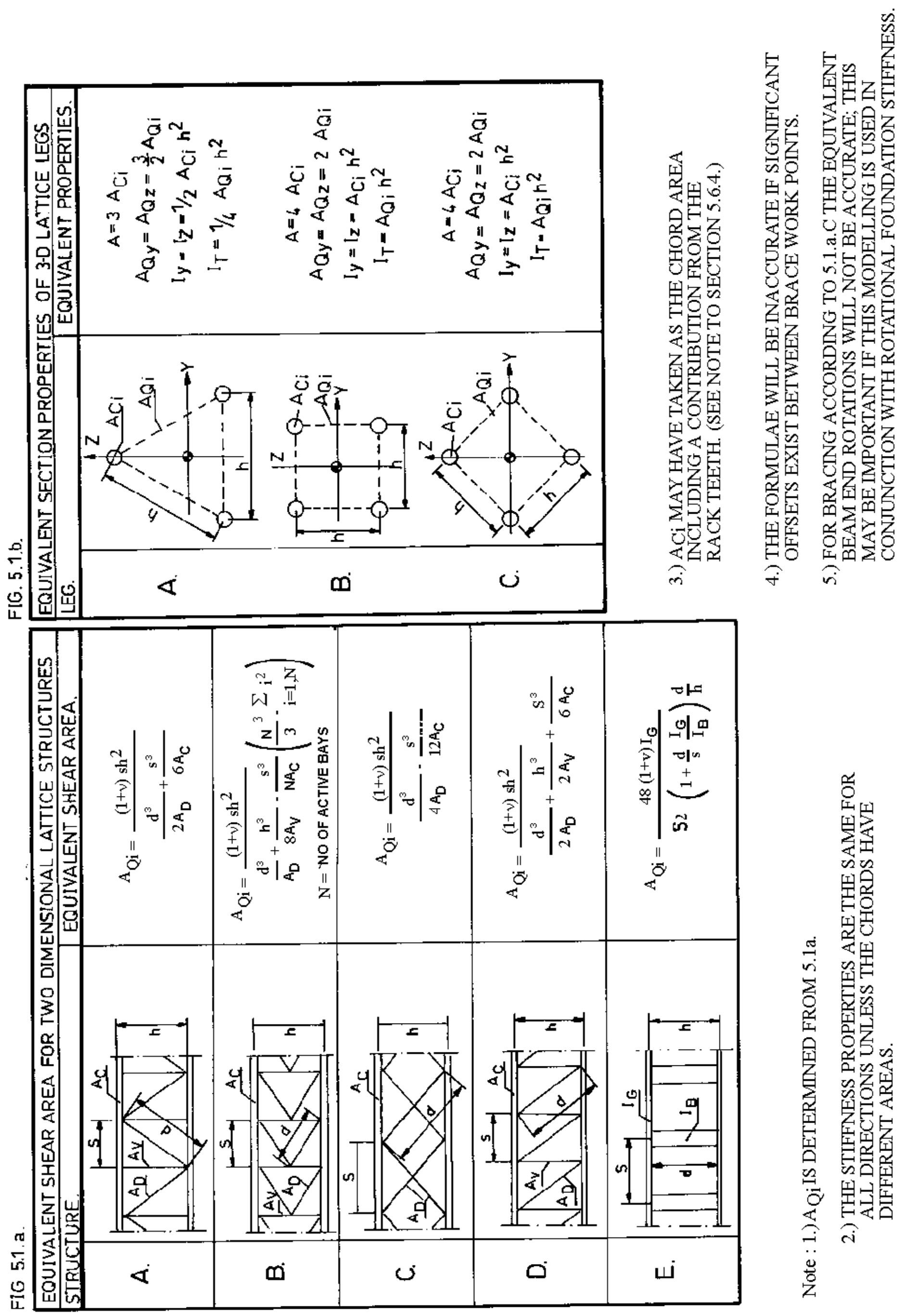


Figure 5.1: Formulas for the determination of equivalent member properties;(After DNV Class Note 31.5 1992 [6] (corrected)

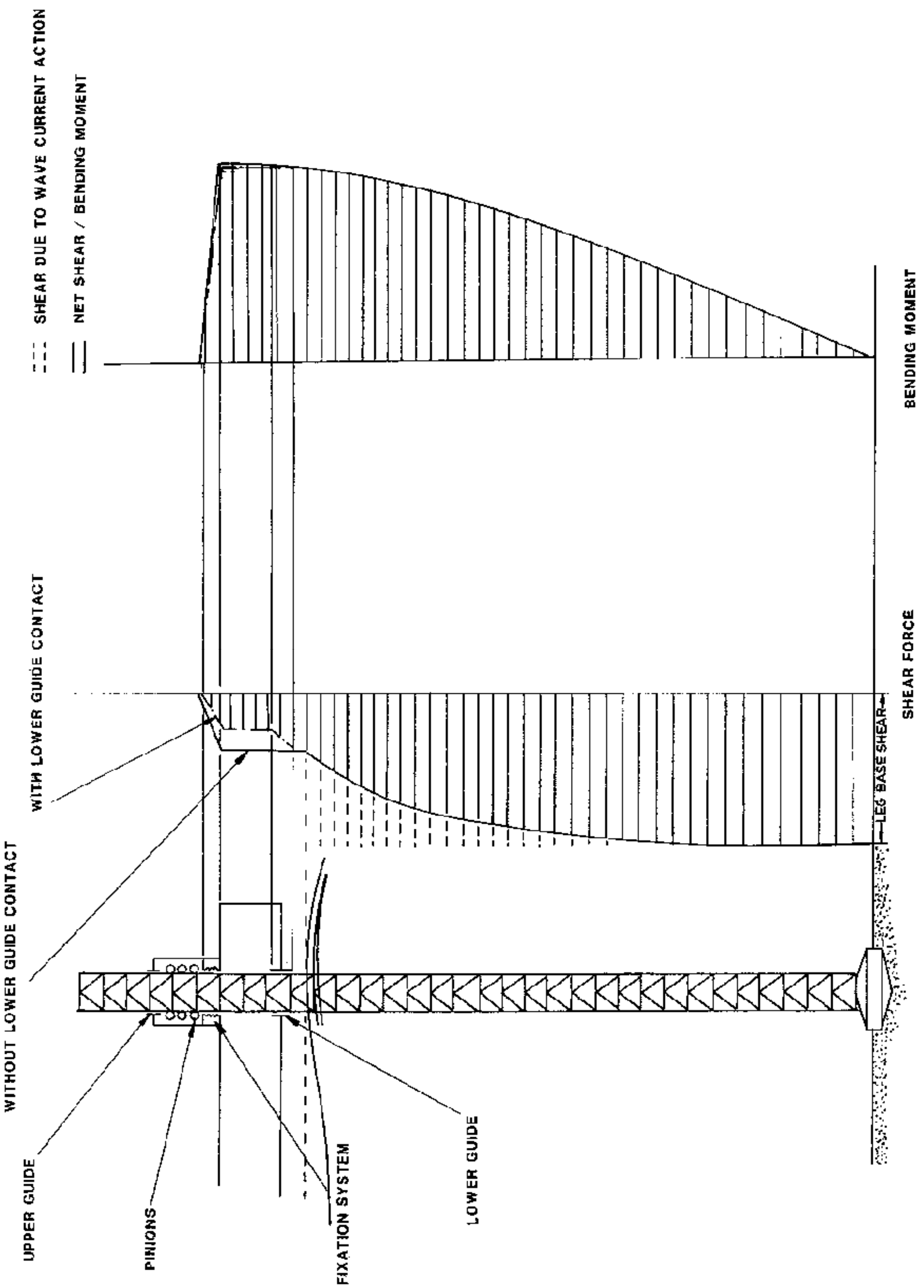


Figure 5.2: Leg shear force and bending moment - jack-ups with a fixation system

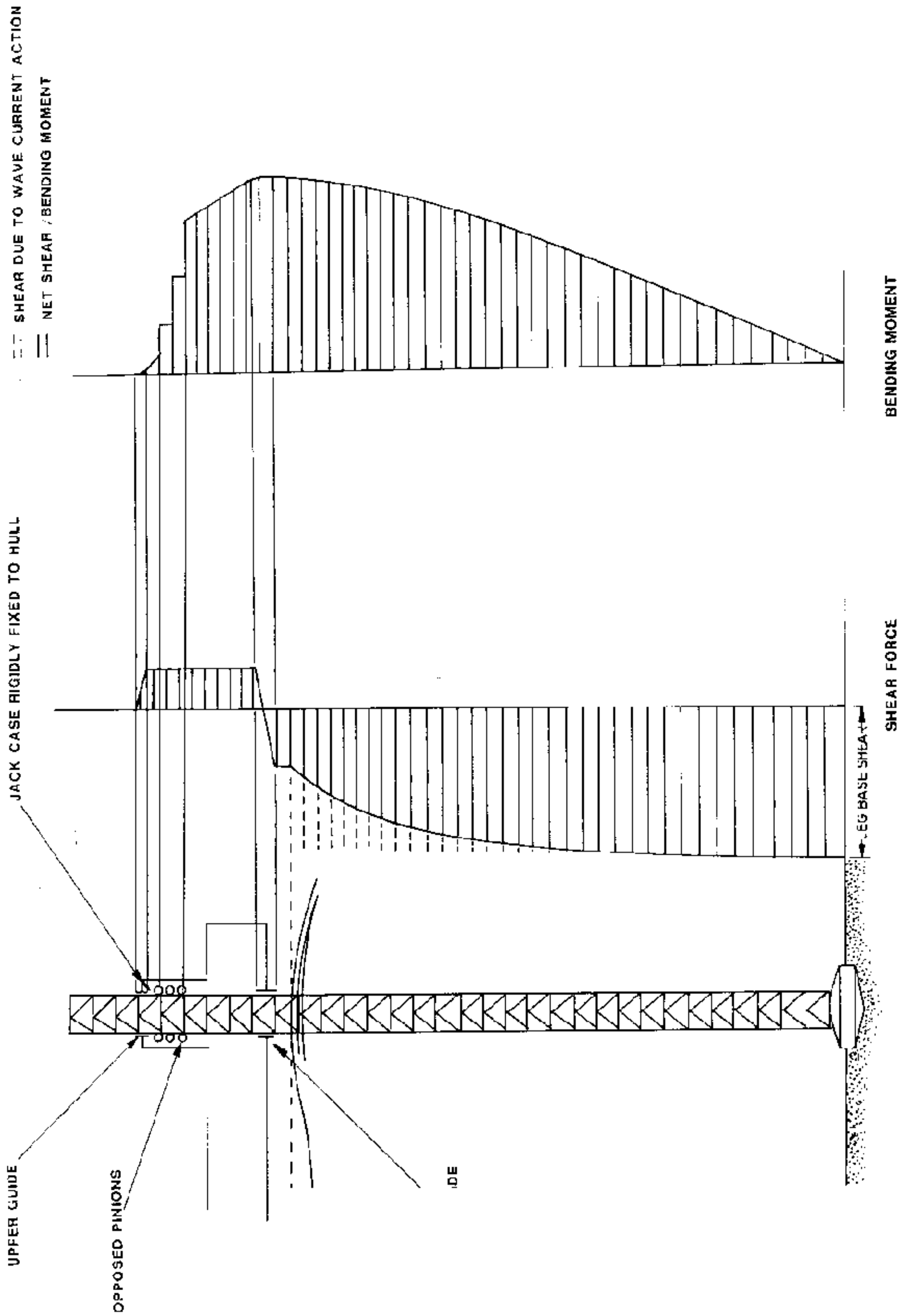


Figure 5.3: Leg shear force and bending moment - jack-ups without a fixation system and having a fixed jacking system with opposed pinions

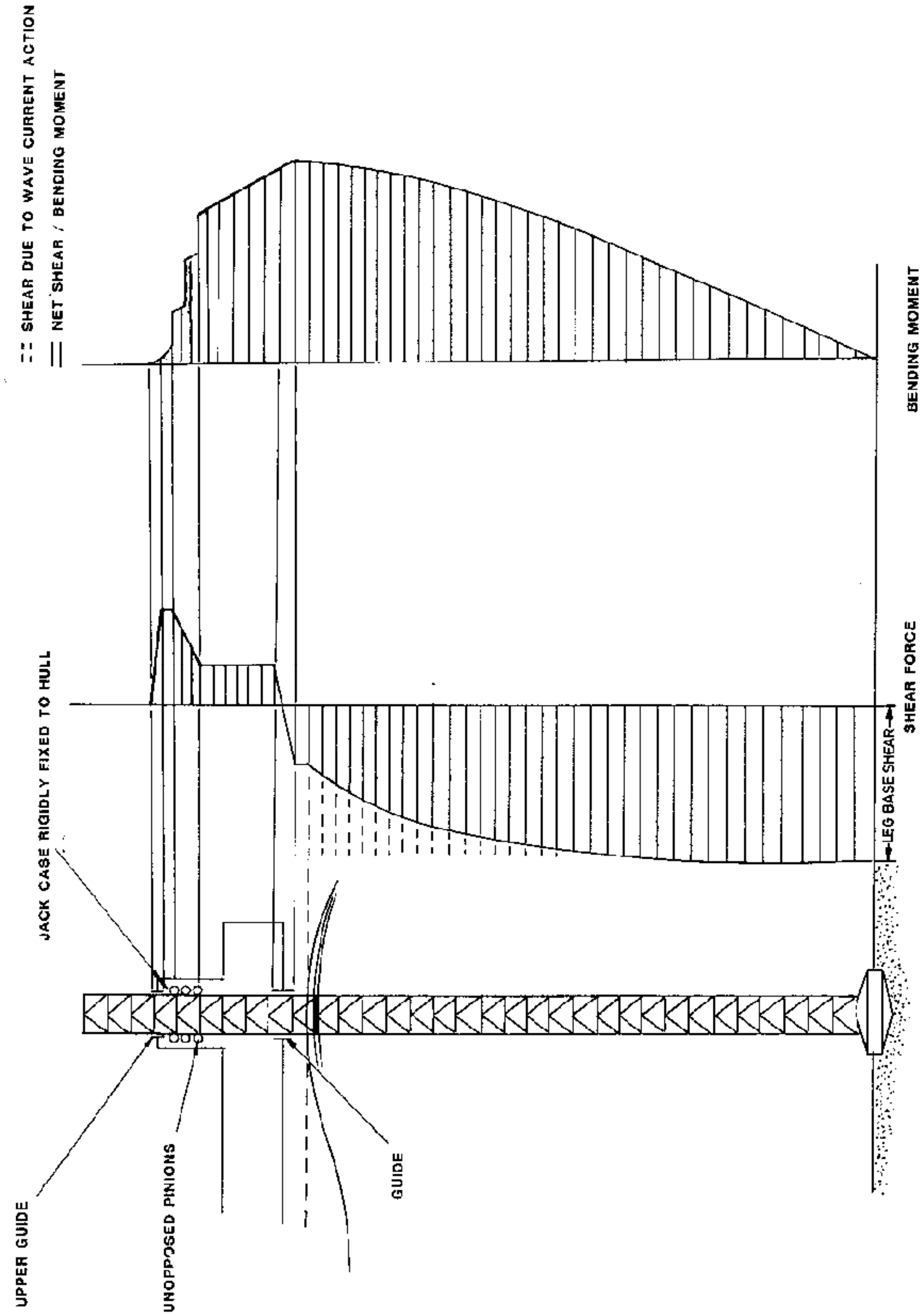


Figure 5.4: Leg shear force and bending moment - jack-ups without a fixation system and having a fixed jacking system with unopposed pinions

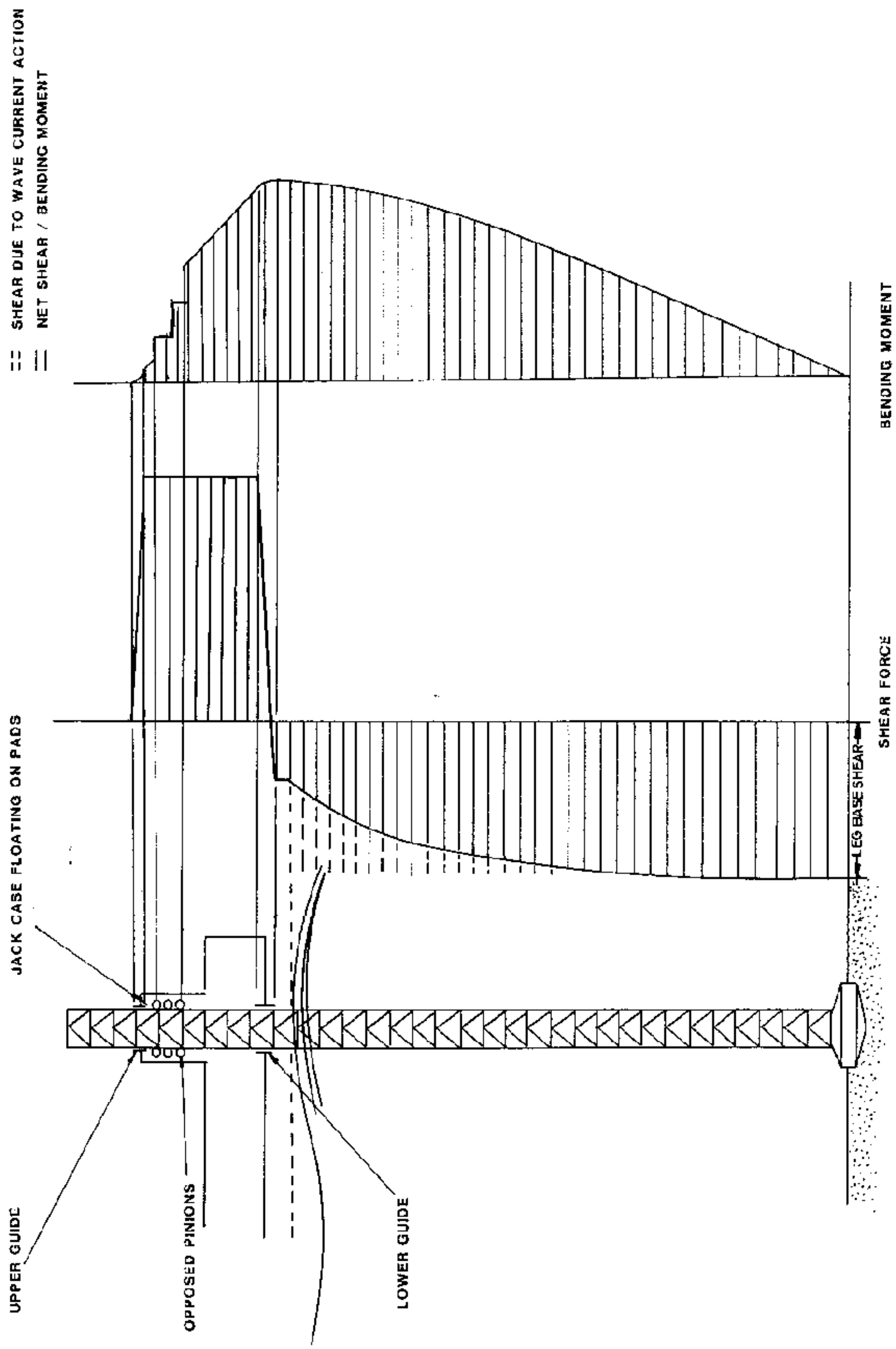
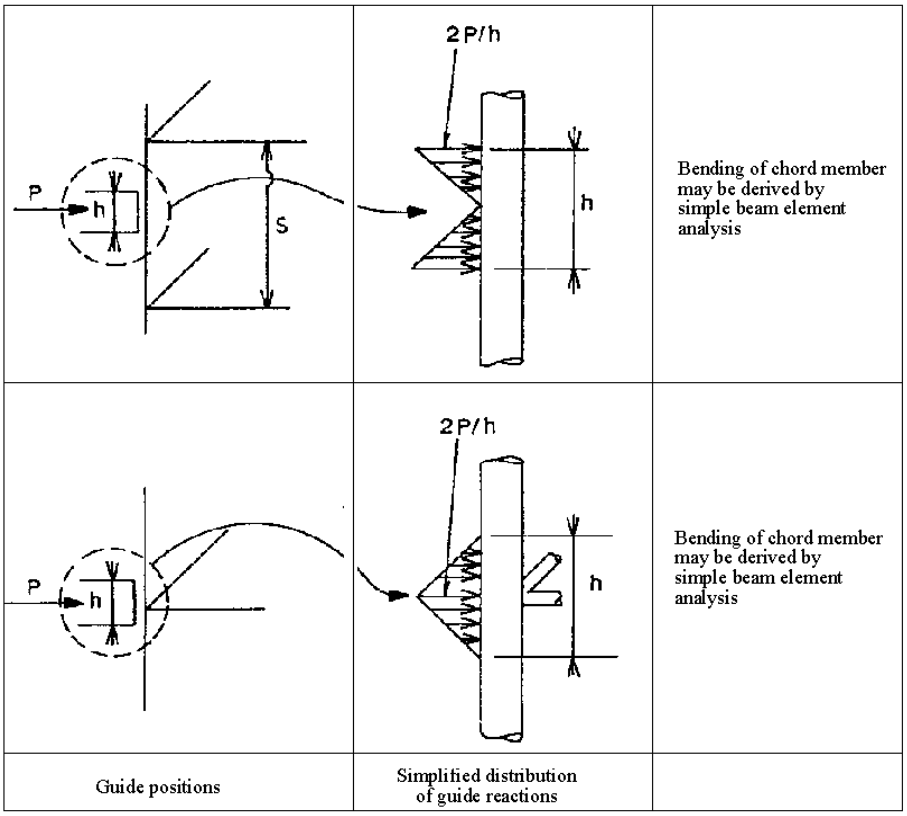


Figure 5.5: Leg shear force and bending moment - jack-ups without a fixation system and having a floating jacking system



<u>Figure 5.6: Correction of point supported guide model for finite guide length (After DNV Class Note 31.5, 1992 [6])</u>

5 GLOSSARY OF TERMS - STRUCTURAL ENGINEERING

A	=	Equivalent axial area of a leg for stiffness calculations.
A_{Ci}	=	Area of chord including a contribution from the rack teeth (see note to Section 5.6.4.)
A_D	=	Axial area of an inclined brace.
A_{Qi}	=	Equivalent shear area of a leg face.
A_{Qy}	=	Equivalent shear area of a leg in y direction.
A_{Qz}	=	Equivalent shear area of a leg in z direction.
A_V	=	Axial area of a brace perpendicular to the chords.
d	=	Length of inclined brace or face to face distance between chords for lattice structures without inclined braces.
D	=	Self weight and non-varying loads.
D_n	=	Inertial loads due to Dynamic response.
E	=	Environmental loads.
h	=	Distance between chord centroids.
h	=	Length of guide.
I_B	=	Second moment of area of 'brace'.
I_G	=	Second moment of area of longitudinal girder.
I_T	=	Equivalent torsional constant of leg about longitudinal axis.
I_Y	=	Equivalent second moment of area of leg about y-y axis for stiffness calculations.
I_z	=	Equivalent second moment of area of leg about z-z axis for stiffness calculations.
L	=	Variable loads.
L	=	Distance from the spudcan reaction point to the hull vertical center of gravity.
N	=	Number of bays, used in determination of equivalent shear area A_Q.
O_T	=	Total horizontal offset of leg base with respect to hull
	=	$O_1 + O_2$
O_1	=	Offset of leg base with respect to hull due to leg-hull clearances.
O_2	=	Offset of leg base with respect to hull due to maximum hull inclination permitted by the operating manual.
P	=	Average axial load in the legs at the hull (total leg load divided by number of legs).
P	=	Guide reaction.
P_E	=	Euler buckling load of an individual leg.
P_g	=	Total effective gravity load on legs at hull, including the hull weight and weight of legs above hull.
s	=	Leg bay height (distance between brace nodes).
δ_s	=	Linear elastic first order displacement of hull.
Δ	=	Approximate hull displacement including P-Δ effects
	=	$\delta_s /(1 - P/P_E)$
v	=	Poissons ratio for the material
	=	0.3 for steel.

6 CALCULATION METHODS - GEOTECHNICAL ENGINEERING

6.1 Introduction

6.1.1 Section 6 addresses three groups of geotechnical areas of concern which are discussed in the following subsections:

6.2 Prediction of footing penetration during preloading.

6.3 Jack-up foundation stability after preloading.

6.4 Other aspects of jack-up foundation performance during or after preloading.

6.1.2 Where geotechnical analyses are performed they should be based on geotechnical data obtained from a site investigation incorporating soil sampling and/or in-situ testing (see Section 3.16).

6.1.3 Uncertainties regarding the geotechnical data should be properly reflected in the interpretation and reporting of analyses for which the data are used.

6.1.4 The majority of spudcans are effectively circular in plan but other spudcan geometries are not uncommon. Typical spudcan designs are illustrated in Figure 6.1. The bearing capacity formulas given in this section are consistent with 'circular' spudcan footings without skin-friction on the leg. Due consideration should be given to the tapered geometry of most spudcans for bearing capacity assessment.

Note: Terms which are not defined in the text may be found in the Glossary to this Section.

6.2 Prediction of Footing Penetration During Preloading

6.2.1 *Analysis Method*

The conventional procedure for the assessment of spudcan load/penetration behavior is given in the following steps:

1. Model the spudcan.

2. Compute the vertical bearing capacity of the footing at various depths below seabed using closed form bearing capacity solutions and plot as a curve.

3. Enter the vertical bearing capacity versus footing penetration curve with the specified maximum preload and read off the predicted footing penetration.

For conventional foundation analyses the spudcan can often be modeled as a flat circular foundation. The equivalent diameter is determined from the area of the actual spudcan cross section in contact with the seabed surface, or where the spudcan is fully embedded, from the largest cross sectional area. Foundation analyses are then performed for this circular foundation at the depth (D) of the maximum cross sectional area in contact with the soil. (See Figure 6.2). Alternative shapes, e.g. tubular legs, should be treated as appropriate.

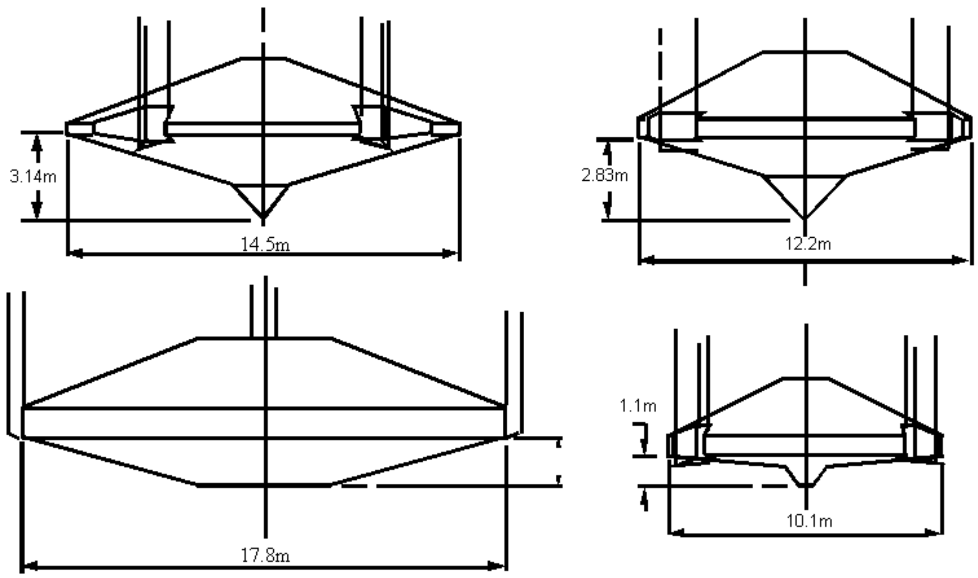


Figure 6.1: Typical spudcan geometries

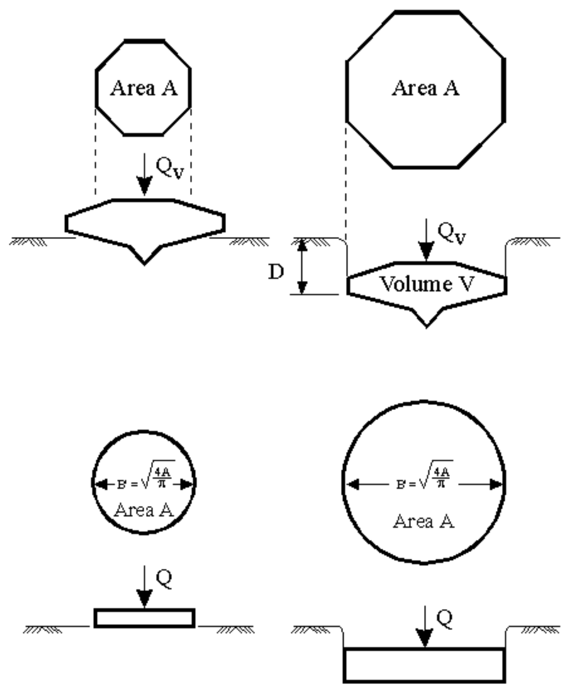


Figure 6.2: Spudcan foundation model

The depth of spudcan penetration is usually defined as the distance from the spudcan tip to the mudline. It is therefore necessary to correct for this when referring to the analytical foundation model.

The possibility of soil back-flow over the footing should be considered when computing bearing capacity. In very soft clays complete back-flow may occur whereas in firm to stiff clays and granular materials, where limited footing penetration may be expected, the significance of back-flow diminishes.

Back-flow in clay may be assumed not to occur if:

$$D \leq \frac{Nc_{us}}{\gamma'}$$

where, in this case, c_{us} is taken as the average undrained cohesive shear strength over the depth of the excavation, N is a stability factor and γ' is the submerged unit weight of the soil.

Conservative stability factors in uniform clays, as a function of excavation depth and diameter, are summarized in Figure 6.3. Alternative stability factors are given in the Commentary. For spudcan penetration analyses it is recommended that conservative criteria are used and the excavation depth be considered as the depth to the maximum spudcan bearing area.

Both the bearing capacity analyses and the above back-flow analysis are based on simple solutions developed for other geotechnical purposes or foundation conditions. These differences should be recognized and are discussed further in the Commentary.

The equations given in the following sections may be considered with or without soil back-flow over the footing. The additional load from back-flow on the footing increases the maximum penetration. In general two cases can be distinguished:

- Immediate back-flow

- Hole side walls collapse after the installation phase.

For deeply penetrated footings the effect of side wall collapse after preloading will be to significantly reduce the ultimate vertical bearing capacity of the foundation. Where relevant this phenomenon should be considered.

For spudcan penetration analyses the ultimate vertical bearing capacity, F_V, may be determined at a series of spudcan penetration depths according to the criteria given in Sections 6.2.2 to 6.2.6.

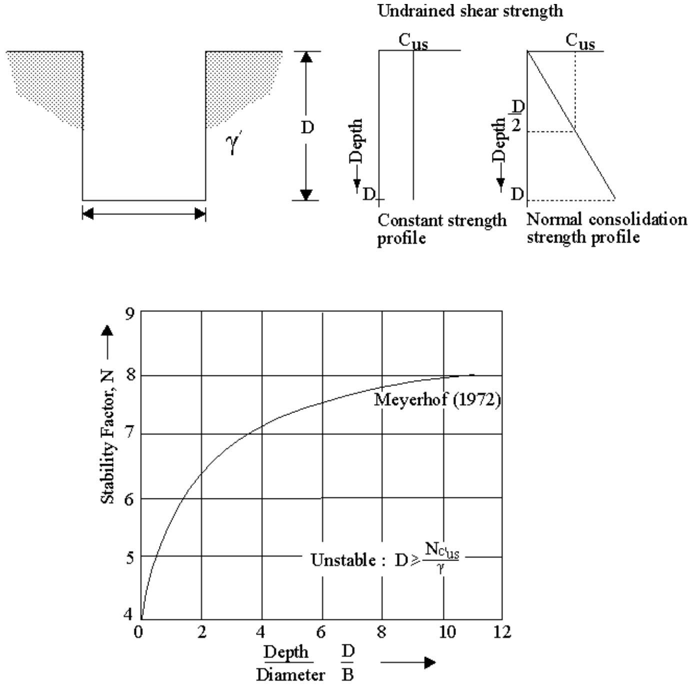


Figure 6.3: Stability factors for cylindrical excavations in clay

6.2.2 *Penetration in Clays*

The ultimate vertical bearing capacity of a foundation in clay (undrained failure in clay, $\phi = 0$) at a specific depth can be expressed by:

$$F_V = (c_u.N_c.s_c.d_c + p_o')A.$$

The maximum preload is equal to the ultimate vertical bearing capacity, F_V, taking into account the effect of backflow, $F_o'A$, and the effective weight of the soil replaced by the spudcan, $\gamma'V$ (see Commentary) i.e.:

$$V_{Lo} = F_V - F'_oA + \gamma'V$$

See Figures 6.2 and 6.4 and note that the terms $- F'_oA + \gamma'V$ should always be considered together.

It is recommended that the value of undrained cohesive shear strength, c_u, is taken as the average value over a distance B/2 from beneath the level where the maximum spudcan diameter is in contact with the soil. (Refer to the Commentary).

The bearing capacity formula given above has been empirically derived for surface foundations and does not account for foundation roughness, shape (conical for most spudcans) or the effects of increased shear strength with depth. These factors are taken into account in a method provided in the Commentary.

Note: It is recognized that the bearing capacity of a soil may reduce when subjected to cyclic loading. (Refer to the Commentary.)

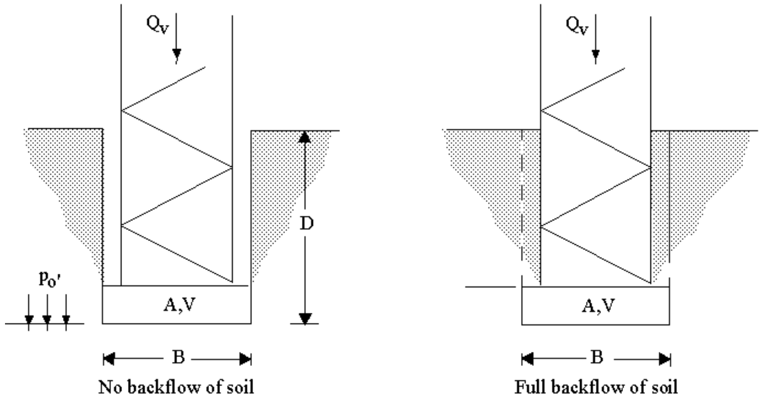


Figure 6.4: Spudcan bearing capacity analysis

6.2.3 *Penetration in Silica Sands*

The ultimate vertical bearing capacity of a circular footing resting in silica sand or other granular material can be computed by the following equation;

$$F_V = (0.5\ \gamma' B\ N_\gamma\ s_\gamma\ d_\gamma + p_o'\ N_q\ s_q\ d_q)A$$

The maximum preload is equal to the ultimate vertical bearing capacity, F_V, taking into account the effect of backflow, $F_o'A$, and the effective weight of the soil replaced by the spudcan, $\gamma'V$ (see Commentary) i.e.:

$$V_{Lo} = F_V - F'_oA + \gamma'V$$

See Figures 6.4 and note that the terms $-F'_oA + \gamma'V$ should always be considered together.

Typically observed load-penetration data for large diameter spudcans suggest that reduced friction angles may be applicable for this analysis method. To account for this it is appropriate to reduce the laboratory derived ϕ by 5°. Further recommendations on the selection of ϕ values are given in the Commentary together with a discussion regarding the use of alternative bearing capacity factors.

6.2.4 *Penetration in Carbonate Sands*

Penetrations in carbonate sands are highly unpredictable and may be minimal in strongly cemented materials, or large, in uncemented materials. Extreme care should be exercised when operating in these materials. Further discussion regarding these soil conditions is provided in the Commentary.

6.2.5 *Penetration in Silts*

It is recommended that upper and lower bound analyses for drained and undrained conditions are performed to determine the range of penetrations. The upper bound solution is modeled as a loose sand and the lower bound solution as a soft clay. Cyclic loading may significantly affect the bearing capacity of silts. See discussion in Commentary.

6.2.6 <u>*Penetration in Layered Soils*</u>

Three basically different foundation failure mechanisms are considered in spudcan predictions in layered soils:

1. General shear.
2. Squeezing.
3. Punch-through.

The first failure mechanism occurs if soil strengths of subsequent layers do not vary significantly. Thus an average soil strength (either c_u or ϕ) can be determined below the footing. The footing penetration versus foundation capacity relationship is then generated using criteria from Sections 6.2.2 through 6.2.5.

Criteria for the other two failure mechanisms (squeezing and punch-through) are given below. The last condition is of particular significance since it concerns a potentially dangerous situation where a strong layer overlies a weak layer and hence a small additional spudcan penetration may be associated with a significant reduction in bearing capacity.

6.2.6.1 *Squeezing of clay*

On a soft clay subject to squeezing overlaying a significantly stronger layer (Figure 6.5), the ultimate vertical bearing capacity of a footing given by Meyerhof [8] is:

For no back-flow conditions:

$$F_V = A\{(a + \frac{bB}{T} + \frac{1.2D}{B})\, c_u + p_o'\} \geq A\{N_c\, s_c\, d_c\, c_u + p_o'\}$$

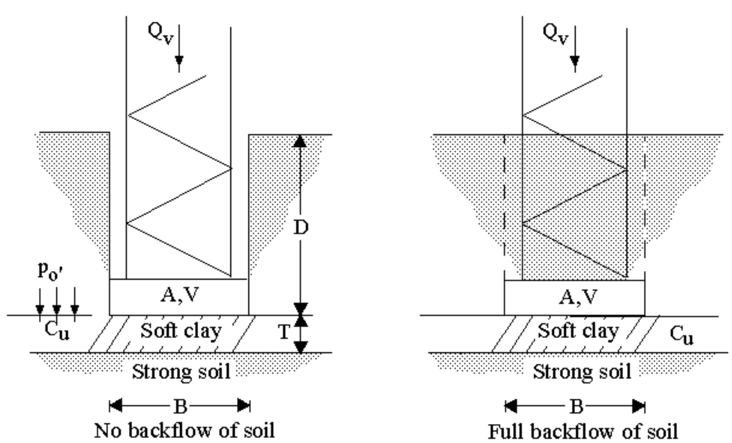
and for full back-flow conditions:

$$F_V = A\{(a + \frac{bB}{T} + \frac{1.2D}{B})\, c_u\} + V\gamma' \geq A\{N_c\, s_c\, d_c\, c_u\} + V\gamma'$$

where the following squeezing factors are recommended:

$a = 5.00$
$b = 0.33$

and c_u refers to the undrained shear strength of the soft clay layer.



<u>Figure 6.5: Spudcan bearing capacity analysis - squeezing clay layer</u>

It is noted that the lower bound foundation capacity is given by general failure in the clay layer (right hand side of equation), and that squeezing occurs when $B \geq 3.45T(1+1.1D/B)$. The upper bound capacity (for $T<<B$) is determined by the ultimate bearing capacity of the underlying strong soil layer.

Comment on the limits included in the above relationships is provided in the Commentary.

6.2.6.2 *Punch-through: Two clay layers*

The ultimate vertical bearing capacity of a spudcan on the <u>surface</u> of a strong clay layer overlying a weak clay layer can be computed according to Brown [9]:

$$F_V = A\,(3\frac{H}{B}\,c_{u,t} + N_c\,s_c\,c_{u,b}) \leq A\,N_c\,s_c\,c_{u,t}$$

See Figure 6.6.

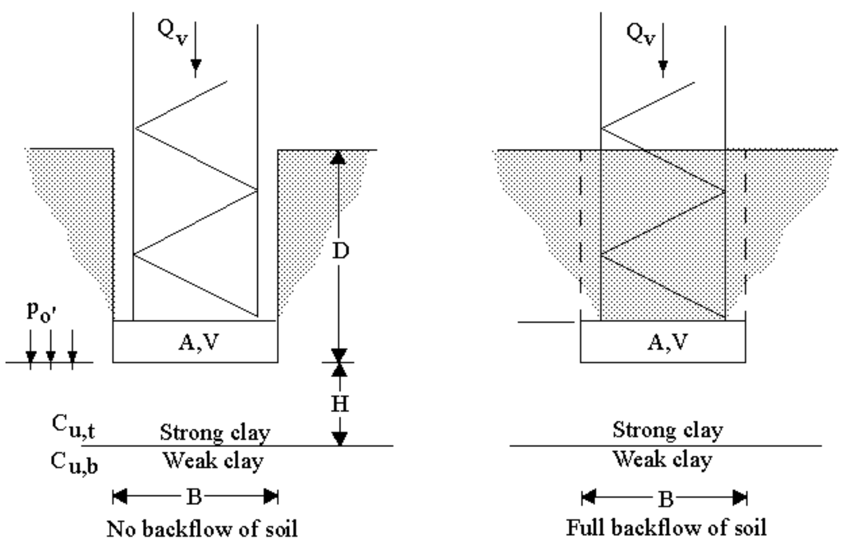
For the evaluation of punch-through potential for deep footings, and to achieve compatibility with the equations used for homogeneous clays, the following equations are recommended:

For no back-flow conditions:

$$F_V = A\,\{3\frac{H}{B}\,c_{u,t} + N_c\,s_c\,(1 + 0.2\,\frac{D+H}{B})\,c_{u,b} + p_o'\} \leq A(N_c\,s_c\,d_c\,c_{u,t} + p_o')$$

and for full back-flow conditions:

$$F_V = A\,[3\frac{H}{B}\,c_{u,t} + N_c\,s_c\,(1 + 0.2\,\frac{D+H}{B})\,c_{u,b}] + \gamma'V \leq A\,N_c\,s_c\,d_c\,c_{u,t} + \gamma'V$$



<u>Figure 6.6: Spudcan bearing capacity analysis - firm clay over weak clay</u>

6.2.6.2 It is noted that the condition (firm clay over soft clay) can also be "man-made" as in some clays artificial crusts can form during delays in the installation procedure. Caution is therefore required in situations where soil sampling/testing is performed from a jack-up prior to preloading.

6.2.6.3 *Punch-through: Sand overlying clay*

The ultimate vertical capacity of a spudcan on a sand layer overlying a weak clay layer can be computed using:

For no back-flow:

$$F_V = F_{V,b} - A\,H\gamma' + 2\,\frac{H}{B}\,(H\gamma' + 2\,p'_o)\,K_s\tan\phi\,A$$

and for full or partial back-flow:

$$F_V = F_{V,b} - A\,H\gamma' - A\,I\,\gamma' + 2\,\frac{H}{B}\,(H\gamma' + 2\,p'_o)\,K_s\tan\phi\,A$$

where;

$F_{V,b}$ is determined according to Section 6.2.2 assuming the footing bears on the surface of the lower clay layer, with no back-flow.

See Figure 6.7.

The coefficient of punching shear, K_s, depends on the strength of both the sand layer and the clay layer. For practical purposes a lower bound for the term $K_s \tan\phi$, applicable to the onset of punch-through, can be approximated by:

$K_s \tan\phi \approx 3c_u/B\gamma'$

An alternative analysis method is described in the Commentary.

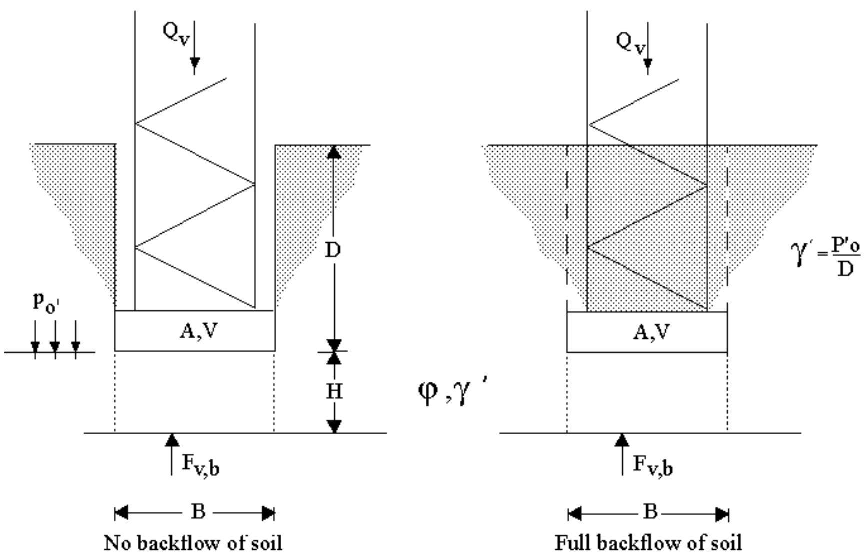
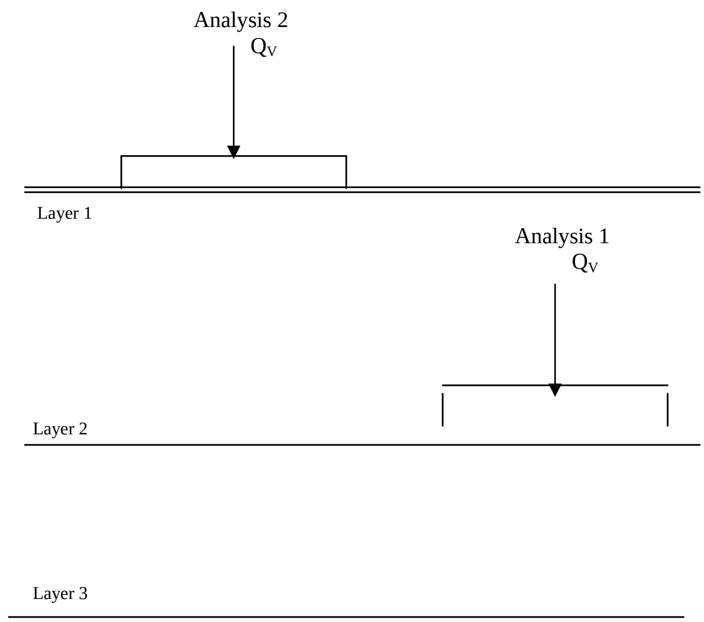


Figure 6.7: Spudcan bearing capacity analysis - sand over clay

6.2.6.4 *Three Layered Systems*

The foundation bearing capacity for a spudcan resting on three soil layers can be computed using the squeezing and punch-through criteria for two layer systems. Firstly the bearing capacity of a footing with diameter B resting on top of the lower two layers is computed. These two layers can then be treated as one (lower) layer in a subsequent two layer system analysis involving the (third) upper layer. For further explanation see Figure 6.8.



Use 2 layer bearing capacity procedures for both analyses

Analysis 2	Analysis 1
Layer 1 over (Layer 2 and 3)	Layer 2 over layer 3

Figure 6.8: Spudcan bearing capacity analysis - three layer case

6.3 Foundation Stability Assessment

6.3.1 *Approach*

The overall foundation stability may be assessed using a phased method with three steps increasing in order of complexity (See Figure 6.9):

- Step 1 Preload and Sliding Check (Section 6.3.2). The foundation capacity check is based on the preloading capability. Sliding of the windward leg is also checked. Loads from pinned footing analysis.

- Step 2 Bearing Capacity Check.

  Step 2a Bearing capacity check (Section 6.3.3), based on resultant loading, assuming a pinned footing. (see Section 5.3.1). Also check sliding.

  Step 2b Bearing capacity check (Section 6.3.4), including rotational, vertical and translational foundation stiffness.

- Step 3 Displacement Check (Section 6.3.5). The displacement check requires the calculation of the displacements associated with an overload situation arising from Step 2b.

Any higher level check need only be performed if the lower level check fails to meet the foundation acceptance criteria given in Section 8.3.

The following sections give details regarding the three phased acceptance procedure. However, there are certain aspects which are not covered in these sections which may require further consideration. Some of the more common ones are listed below:

- Soils where the "long term" (drained) bearing capacity is less than the "short term" (undrained) capacity. This may be the case for overconsolidated cohesive soils (silts and clays) with significant amounts of sand seams.

- Where soil back-flows over the spudcan after the preload installation phase, (silts, clays).

- If a reduction of soil strength due to cyclic loading occurs. This can be of particular significance for silty soils and/or carbonate materials.

- If an increase in spudcan penetration occurs, due to cyclic loading, where a potential punch-through exists.

- In soils with horizontal seams of weak soils located beneath the spudcan it is recommended that the lateral bearing capacity/sliding stability of the foundation is verified.

If any of the above circumstances exist further analysis is required.

In the case of partial spudcan embedment, (e.g. sandy soils), additional footing embedment may result in a considerable increase in bearing capacity.

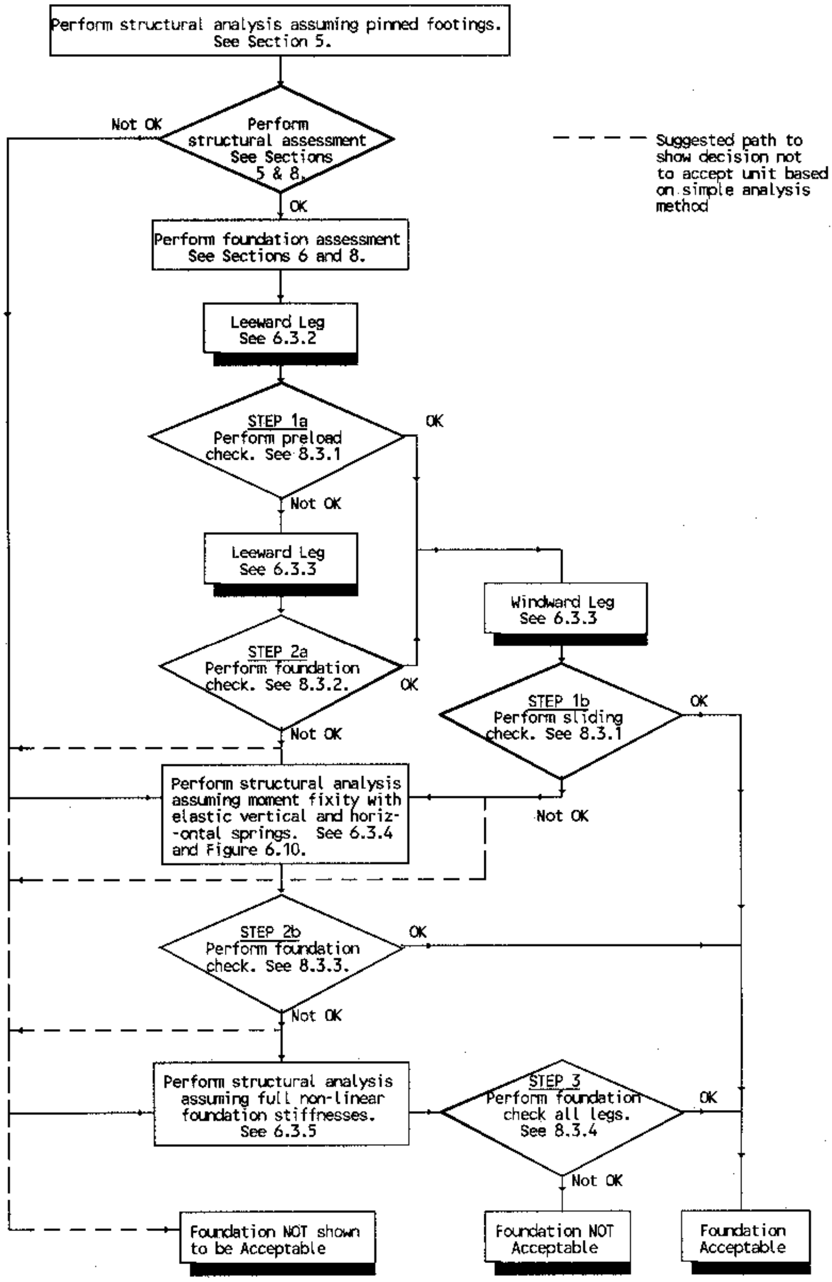


Figure 6.9: Foundation stability assessment

6.3.2 *Ultimate bearing capacity for vertical loading - Preload Check (Step 1)*

Except as discussed in 6.3.1, when the horizontal load is small, the ultimate vertical bearing capacity under extreme conditions is assumed to be the same as the maximum footing load during preloading, (V_{Lo}). The minimum requirements for V_{Lo} are given in Section 8.3.1.3 or 8.3.2 as applicable.

6.3.3 *Bearing Capacity/Sliding Check - Pinned footing (Step 2a)*

A reduction in vertical bearing capacity, F_V, of a footing occurs when it is simultaneously subjected to horizontal loading, Q_H, and moment loading, Q_M. The latter is ignored in Step 2a analyses as the footings are considered to be pinned.

The vertical/horizontal capacity envelope, F_{VH}, for sands and clays may be generated according to the following criteria, however, further discussion with regard to the analytical applicability is provided in the Commentary.

6.3.3.1 *Ultimate Vertical/horizontal bearing capacity envelopes for spudcan footings in sand:*

The general ultimate vertical/horizontal bearing capacity envelope for jack-up footings in sand is as follows:

$$F_{VH} = A\ (0.5\ \gamma'\ B\ N_\gamma\ s_\gamma\ i_\gamma\ d_\gamma + p_o'N_q\ s_q\ i_q\ d_q)$$

During the preloading phase it may be assumed that no horizontal load acts on the foundation and that the ultimate vertical bearing capacity of the soil is in equilibrium with the applied footing installation load, V_{Lo}. The applied footing installation load should include the effect of back-flow and spudcan buoyancy i.e. $V_{Lo} = F_V - F_o'A + \gamma'V$. In this instance the inclination factors assume values of unity and the remaining terms may be defined.

Substituting for i_q and i_γ the appropriate relationship may be written for generation of the foundation capacity for combined vertical and horizontal loading as:

$$F_{VH} = A\ \{0.5\ \gamma'\ B\ N_\gamma\ s_\gamma\ d_\gamma\ [1 - (F_H/\ F_{VH})^*]^{m+1} + p_o'N_q\ s_q\ d_q\ [1 - (F_H/F_{VH})^*]^m\}$$

This may be solved by the use of assumed values for (F_H/F_{VH}) designated $(F_H/F_{VH})^*$. For example use $(F_H/F_{VH})^* = 0.00, 0.04, 0.08, 0.12$, etc. For these values corresponding F_{VH} values may be determined.

The correct F_H values may then be determined as F_{VH} and $(F_H/F_{VH})^*$ are known, e.g. for $(F_H/F_{VH})^* = 0.12$, $F_H^* = 0.12\ F_{VH}^*$.

The corrected horizontal capacity, F_H, may then be given as:

$$F_H = F_H^* + 0.5\gamma'\ (k_p - k_a)\ (h_1 + h_2)\ A_s$$

The sliding capacity envelope of a footing in sand is given by:

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

where δ is the steel/soil friction angle which for a flat plate, $\delta = \phi - 5°$, and for a rough surfaced conically shaped spudcan $\delta = \phi$.

6.3.3.2 *Ultimate vertical/horizontal bearing capacity envelopes for spudcan; footings in clay*

The general equation for the horizontal and vertical bearing capacity envelopes for footings in clay is as follows:

$$F_{VH} = A\,[(N_c\, c_u\, s_c\, d_c\, i_c) + p_o'\, N_q\, s_q\, i_q\, d_q]$$

Substituting for the inclination factors for a circular footing the equation may be written as:

$$F_{VH} = A\,\{N_c\, c_u\, s_c\, d_c\,[1 - (1.5F_H^*/N_cAc_u)] + p_o'\, N_q\, s_q\,(1 - F_H^*/F_{VH})^{1.5}\, d_q\}$$

The ultimate bearing capacity envelope under inclined loading may be determined by substituting values of F_{VH} and solving for F_H^*.

F_H may then be given as:

$$F_H = F_H^* + (c_{uo} + c_{ul})A_s$$

Footing sliding capacity in clay:

When $0 \leq Q_V \leq 0.5\ F_V$ the sliding capacity in clay may be conservatively assumed constant, determined by:

$$F_H = Ac_{uo} + (c_{uo} + c_{ul})A_s$$

6.3.3.3 *Ultimate vertical/horizontal bearing capacity envelopes for spudcan for spudcan footings on layered soils.*

The above formulas (Sections 6.3.3.1 through 6.3.3.2) can also generally be used to make a conservative estimate of the ultimate F_{VH}-F_H relationship for layered soils by considering failure through the weakest zones in such a soil profile.

The bearing capacity of layered soils may be determined using the principles of limiting equilibrium analysis or the finite element method.

6.3.3.4 *Settlements resulting from exceedence of the capacity envelope*

Vertical settlement and/or sliding of a footing can occur if the storm load combination is in excess of the (F_{VH}-F_H) resistance envelope computed for the spudcan at the penetration achieved during installation. Such settlements can result in a gain of (F_{VH}-F_H) bearing capacity, e.g. in silica sands. However, the integrity of the foundation may decrease in the situation where a potential punch-through exists, e.g. where dense sand overlies soft clay. More thorough analyses are required for complex and/or potentially dangerous foundation conditions of the type listed in Section 6.3.1.

6.3.4 *Footing with moment fixity and vertical and horizontal stiffness (Step 2b)*

Foundation fixity is the rotational restraint offered by the soil supporting the foundation. The degree of fixity is dependent on the soil type, the maximum vertical footing load during installation, the foundation stress history, the structural stiffness of the unit, the geometry of the footings and the combination of vertical and horizontal loading under consideration.

Inclusion of foundation fixity in an assessment incorporates a check on bearing capacity in terms of vertical and horizontal (sliding) capacities. The amount of rotational fixity is not directly involved in a checking equation, but it serves to modify the forces (beneficially) in both the foundation and structure. The bearing and sliding checks are performed implicitly through the use of the yield function and explicitly through the bearing capacity and sliding checks described in Section 6.3.3.

Uncertainties in soil properties should be considered when including fixity in assessments. Where data reliability is uncertain, an upper/lower bound sensitivity analysis should be performed.

For performing structural analysis, horizontal and vertical spring stiffnesses should be included in addition to the rotational stiffness (see Section 5.3). The springs should be applied to the spudcan support point as defined in Section 5.2. The calculation of fixity should be based on factored environmental loading including dead, live, environmental, inertial and P-Δ loads.

6.3.4.1 *Calculation procedures accounting for moment fixity – See also 6.3.4.6*

The interaction of vertical, horizontal and rotational forces has been modeled based on a plasticity relationship (References C6 [48] through [52]). The plasticity relationship can account for moment softening at high load levels, unloading behavior and work-hardening effects. This type of foundation modeling is preferable if foundation fixity is to be included directly in a time-domain analysis.

For a pseudo-static analysis, a simplified application of this full plasticity analysis is described in this section. This simple approach can be used to create moment loads on the spudcan by inclusion of a simple linear rotational spring to generate moments at the spudcan. The moment thus induced on the spudcan is limited to a capacity based on the yield interaction relationship among vertical (Q_V), horizontal (Q_H) and moment (Q_M) loads acting at the spudcan.

This simple procedure is described in the following steps:

1. Include vertical, horizontal and (initial) rotational stiffnesses (linear springs) to the analytical model and apply the gravity and factored metocean and inertial loading.

2. Calculate the yield interaction function value using the resulting forces at each spudcan. For extreme wave analysis, the result will likely indicate the force combination falls outside the yield surface. In this case, reduce the rotational stiffness (arbitrarily) and repeat the analysis.

3. Continue with step 2 until the force combination at each spudcan lies essentially on the yield surface. If the moment is reduced to zero, and the force combination is still outside the yield surface, then a bearing failure (either vertical or horizontal) is indicated.

4. If a force combination initially falls within the yield surface, the rotational stiffness must be further checked to satisfy the reduced stiffness conditions in Section 6.3.4.3.

The following sections are applicable to traditional spudcan designs. Information on spudcans fitted with skirts can be found in references C6 [48] through [51].

6.3.4.2 *Ultimate Vertical / horizontal / rotational capacity interaction; function for spudcan footings in sand and clay*

For shallow embedment for both sand and clay, the yield interaction is defined by the following expression:

$$\sqrt{\left[\frac{F_{HM}}{H_{Lo}}\right]^2 + \left[\frac{F_M}{M_{Lo}}\right]^2} - 4\left[\frac{F_{VHM}}{V_{Lo}}\right]\left[1 - \frac{F_{VHM}}{V_{Lo}}\right] = 0$$

where V_{Lo} is taken to be equal to the vertical spudcan load achieved during preloading and H_{Lo} and M_{Lo} are defined as follows:

For sand:

$$\left.\begin{aligned} H_{Lo} &= (C_1/C_2)(V_{Lo}/4) \\ &= 0.12V_{Lo} \\ M_{Lo} &= C_1 V_{Lo} B/4 \\ &= 0.075V_{Lo}B \end{aligned}\right\} \textit{with}\ \mathrm{C}_1 = 0.3, \mathrm{C}_2 = 0.625$$

For clay: $H_{Lo} = c_{uo}A + (c_{uo} + c_{ul})\,A_s$
$M_{Lo} = 0.1V_{Lo}B$

Note that in the above expression for the yield surface, if a load combination (Q_V,Q_H,Q_M) satisfies the equality then (Q_V,Q_H,Q_M) = (F_{VHM}, F_{HM}, F_M). The load combination (Q_V,Q_H,Q_M) lies outside the yield surface if the left-hand side is greater than zero. Conversely, the load combination lies inside the yield surface if the left-hand side is less than zero.

The expression for the yield surface can be re-written to give the maximum spudcan moment as a function of the horizontal and vertical loads. Thus, for a given vertical and horizontal load combination which, with zero moment, lies inside the yield surface given above, the maximum moment at a spudcan cannot exceed the value defined below.

$$F_M = M_{Lo}\left\{16\left[\frac{Q_V}{V_{Lo}}\right]^2\left[1 - \frac{Q_V}{V_{Lo}}\right]^2 - \left[\frac{Q_H}{H_{Lo}}\right]^2\right\}^{0.5}$$

The equation above only applies when:

$$0.0 < Q_V < V_{Lo}$$

$$Q_H < 4H_{Lo}\left[\frac{Q_V}{V_{Lo}}\right]\left[1 - \frac{Q_V}{V_{Lo}}\right]$$

Embedded footings in clay achieve greater moment and sliding capacities as compared to shallow penetrations in clay. For fully or partially penetrated spudcans, the yield surface at $F_{VHM}/V_{Lo}<0.5$ can be expressed as:

$$\left[\frac{F_{HM}}{f_1 H_{Lo}}\right]^2 + \left[\frac{F_M}{f_2 M_{Lo}}\right]^2 - 1.0 = 0$$

where;

$$f_1 = \alpha + 2(1 - \alpha)\left[\frac{Q_{VHM}}{V_{Lo}}\right]$$

f_2 = f_1 where suction (i.e. uplift resistance) is available,

$= \quad 4\left[\frac{Q_{VHM}}{V_{Lo}}\right]\left[1-\frac{Q_{VHM}}{V_{Lo}}\right]$ where suction cannot be relied upon

$\alpha \quad = \quad$ 1.0 for soft clays

$= \quad$ 0.5 for stiff clays

α accounts for the degree of adhesion. Engineers may want to consider α values within the range 0.5-1.0 depending on site specific soil data, spudcan/soil interface roughness, etc. An α value less than 0.5 may be considered for situations such as a hard clay at the surface. In this case, the standard form of the yield surface should be considered.

Thus, for a given vertical and horizontal load combination which, with zero moment, lies inside the yield surface given above, the maximum moment at a spudcan for a clay foundation with $Q_{VHM}/V_{Lo}<0.5$ cannot exceed the value defined below:

$$F_M = f_2 M_{L0}\left\{1-\left[\frac{Q_{HM}}{f_1 H_{Lo}}\right]^2\right\}^{0.5}$$

The equation above only applies when:

$$0.0 < Q_V < V_{Lo}$$

$$Q_H < f_1 H_{Lo}$$

There is no existing data for deeply embedded footings in sand. The application of the yield surface calibrated to shallow penetrations will likely be conservative for the deep penetration case.

6.3.4.3 *Estimation of rotational, vertical, and horizontal stiffness*

An initial estimate for rotational stiffness, K_3, which is applicable for a flat spudcan without embedment (Winterkorn [10]) under relatively low levels of load is given below:

$K_3 = \dfrac{GB^3}{3(1-\nu)}$, flat spudcan with no embedment

Values for K_3 for other cases are given in the Commentary. The selection of the shear modulus, G, is discussed in the Commentary. An upper or lower bound value should be selected as appropriate for the analysis being undertaken.

For clays susceptible to cyclic degradation (OCR ≥ 4) the soil rotational stiffness, calculated from the degraded static soil properties, may be multiplied by a factor of 1.25, Anderson [18].

If the load combination of (Q_V,Q_H,Q_M) lies outside the yield surface, the linear rotational stiffness at the spudcan must be reduced until the load combination lies on the yield surface. The reduction in stiffness is arbitrary and requires iterative analyses.

It should be noted that if the initial load combination (Q_V,Q_H,Q_M) lies outside the yield surface, the final value of the rotational stiffness is determined only by the requirement that the generated moment at the spudcan falls on the yield surface.

If the load combination of (Q_V,Q_H,Q_M) lies inside the yield surface, the initial estimate of rotational stiffness should be reduced by a factor, f_r. The reduction factor is equal to unity when the moment and horizontal forces are zero. It is given by the following expression (Svanø, [56]):

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

where r_f is the failure ratio defined by:

$$r_f = \frac{\left\{\left[\dfrac{Q_H}{H_{Lo}}\right]^2 + \left[\dfrac{Q_M}{M_{Lo}}\right]^2\right\}^{0.5}}{4\left[\dfrac{Q_V}{V_{Lo}}\right]\left[1 - \dfrac{Q_V}{V_{Lo}}\right]}$$

Note that $r_f > 1.0$ implies that the load combination (Q_V,Q_H,Q_M) lies outside the yield surface. Under such conditions, the reduced stiffness factor is not applicable.

For fully embedded foundations in clays at vertical load ratio $F_{VHM}/V_{Lo} < 0.5$, the failure ratio may be expressed as:

$$r_f = \left\{\left[\frac{Q_{HM}}{f_1 H_{Lo}}\right]^2 + \left[\frac{Q_M}{f_2 M_{Lo}}\right]^2\right\}^{0.5}$$

where f_1 and f_2 are as defined in Section 6.3.4.2 above, but replacing F_{VHM} with Q_V.

Vertical and horizontal stiffnesses can be estimated from the elastic solutions for a rigid circular plate on an elastic half-space (assuming no embedment):

Vertical stiffness, $K_1 = \dfrac{2GB}{(1-\nu)}$

Horizontal stiffness, $K_2 = \dfrac{16GB(1-\nu)}{(7-8\nu)}$

Advice on the selection of appropriate values for G may be found in the Commentary.

6.3.4.4 *Extension of the yield surface for additional penetration*

On seabeds of silica sands, conical spudcans which are not fully seated may show a plastic moment restraint due to further penetration. The effect may be taken into account for legs with $Q_V/V_{Lo} > 0$.

The moment capacity M_p associated with further penetration is estimated as the minimum of M_{PS} and M_{PV}, calculated as follows (Svanø [56]):

$M_{PS} = 0.075\ B\ V_{Lo}(D/B)^3$

$M_{PV} = 0.15\ B\ F_{VHM}$

in which B is the plan diameter of the effective contact area after preload, and D is the plan diameter of the contact area when the spudcan is fully seated.

The combined capacity should be checked against the modified yield function:

$$\sqrt{\left[\frac{F_{HM}}{H_{Lo}}\right]^2 + \left[\frac{F_M}{M_P}\right]^2} - 4\left[\frac{F_{VHM}}{V_{Lo}}\right]\left[1 - \frac{F_{VHM}}{V_{Lo}}\right] = 0$$

For additional penetration of spudcans in clay, references C6 [49] and [52] provide work-hardening modifications to the yield surface equations. Updated stiffnesses are determined through plasticity principles.

6.3.4.5 *Deep Footing Penetration*

For deep footing penetrations, typically experienced in soft clay conditions, the calculation of foundation fixity may be augmented with the inclusion of the lateral soil resistance on the leg members due to soil back-flow over the spudcan. This lateral soil resistance is effectively added to the rotational elastic stiffness of the spudcan (as determined in Section 6.3.4.3), (Brekke [7]).

The lateral soil resistance may be modeled based on concepts proposed by Matlock [17] for lateral soil resistance of piles. The jack-up leg may be modeled as an equivalent pile for purposes of determining "p-y", or load-deflection curves.

The diameters of the individual members (i.e., leg chords and braces) give appropriate characteristic dimensions for determining the p-y curves. The p-y curve for each member is summed to form a p-y curve for the entire leg. Only one face of each leg should be assumed to be in contact with the soil and contribute to lateral resistance.

Given a set of p-y curves for the leg, the lateral force-deflection along the entire embedded leg section is thus determined. Typically, equivalent springs at each bay elevation are used to simplify the calculations.

6.3.4.6 *Calculation procedures accounting for moment fixity – further details*

Structural analyses should account for rotational, horizontal and vertical stiffnesses at all spudcans. The jack-up is then acceptable if the following conditions are met:

1. Structural conditions satisfy acceptance criteria outlined in Section 8.1.
2. Factored foundation loads Q_V, Q_H satisfy, as applicable, the bearing capacity criteria in Sections 8.3.2 or 8.3.1.5.
3. Factored foundation loads Q_V, Q_H, Q_M satisfy the appropriate unfactored yield surface criterion from Section 6.3.4.2 or 6.3.4.4. Factored foundation loads exceeding this requirement are permitted provided that the soil-structure interaction model adopted accurately captures the expansion of the foundation yield surface after first yield, and that the large-displacement effects of associated structural displacements are taken into account.
4. The analysis ensures load & displacement compatibility between the foundation and the structure.
5. The location is not prone to, or is protected from, scour so that the assumed fixity is assured.

Fixity may be included in the response simulation in three ways (Refer to Figure 6.11 below):

1. By conservatively considering effects of changes to seabed boundary reactions only and ignoring any reduction in the dynamic response with pinned footings. In this approach quasi-static analyses are used in the iterations of the procedure given in Section 6.3.4.1 to derive the foundation rotational and horizontal secant stiffnesses with loadings obtained from the pinned foundation case including dynamics. This approach is not applicable if the inclusion of fixity brings the natural period closer to the wave period.
2. By considering linearised fixity in SDOF or more detailed dynamic calculations and then carrying out a final quasi-static analysis with non-linear fixity using the procedure of Section 6.3.4.1. If this approach is adopted, care should be taken to ensure that the natural period with fixity does not fall at a cancellation point in the wave force transfer function (Sections 7.3.5.2, 7.3.5.4, C7.4 & Fig C7.1). Typically the initial linearised rotational stiffness for the dynamic analysis may be taken as 80% of value determined from the formulation in the first paragraph of Section 6.3.4.3. When this stiffness is adjusted to avoid wave force cancellation, the adjusted value may lie anywhere between 0% and 100% of the value from Section 6.3.4.3.

   This simplified approach does not capture the temporary reductions in stiffness which occur during plasticity events, but also does not capture the increased damping that is associated with these events; these two effects are considered to be largely self-cancelling. Given that care is taken to avoid wave force cancellation effects, it is considered that the dynamic response will be determined at a level which is either realistic or conservative.

   For further discussion of approaches which may be used to avoid cancellation and reinforcement effects refer to the Commentary Section C7.4.
3. By considering the effects of the foundation fixity on both the dynamic response and the seabed reactions. This approach is more complete and may require a complex iterative calculation procedure. The following outline procedure may be adopted:
   a) Use a time-domain dynamic analysis to determine structural response and foundation loadings at each time step.
   b) Compute the foundation behaviour using a non-linear elasto-plastic model, such that at each time step the plastic and elastic portions of the behaviour are captured. If desired, this model may include hysteresis. This will likely require an iterative procedure.
   c) When plasticity occurs, the responses will be influenced by the load history. Consideration should be given to ensuring that the methodology used to determine the extreme values provides stable results. In cases where the analysis is intended to provide final results (rather than DAF’s for application in subsequent analysis step) it may be appropriate to perform analyses for differing wave histories, and then determine the extremes from a procedure such as that given in C7.B.2.3.

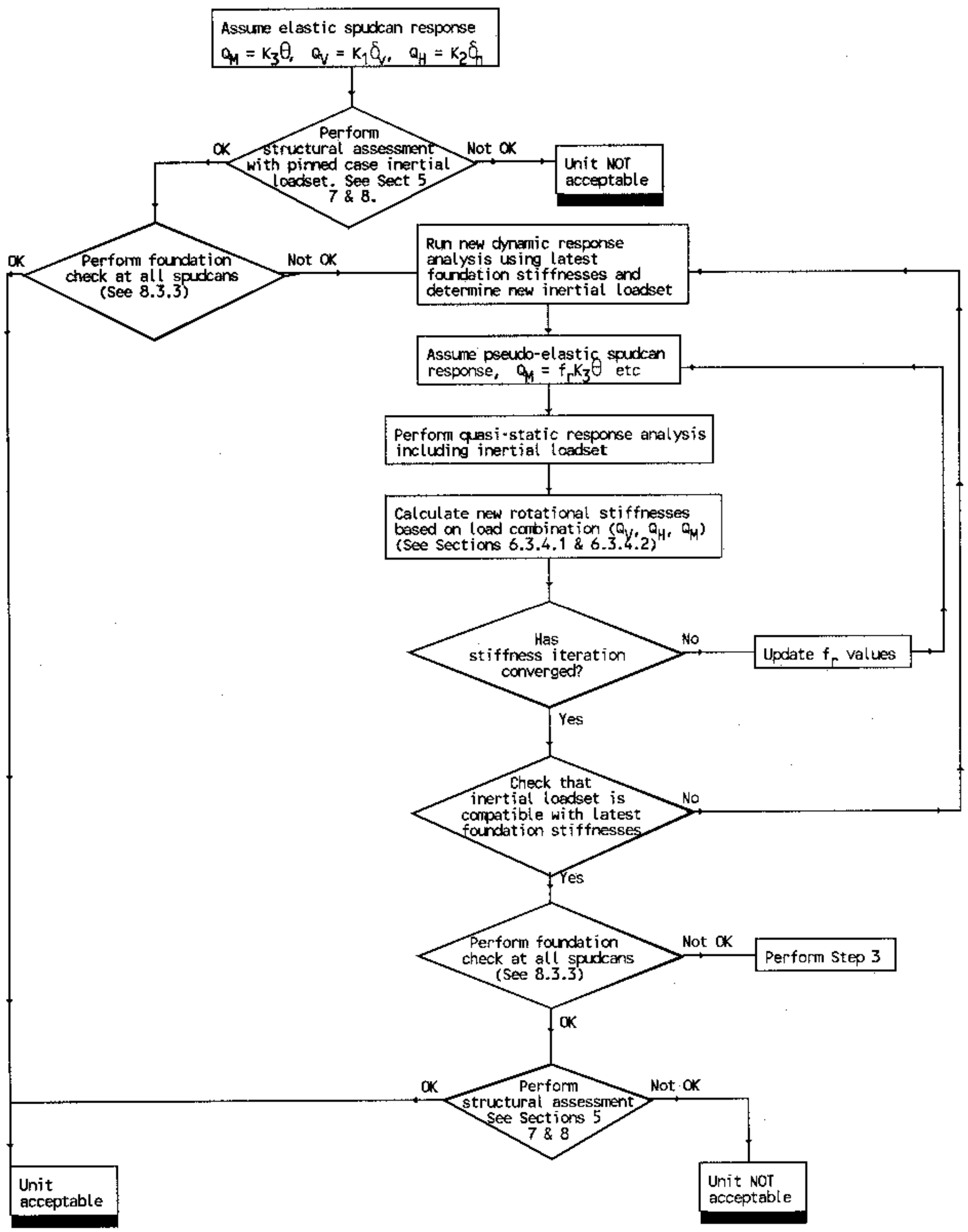


Figure 6.10: Calculation procedure to account for foundation fixity

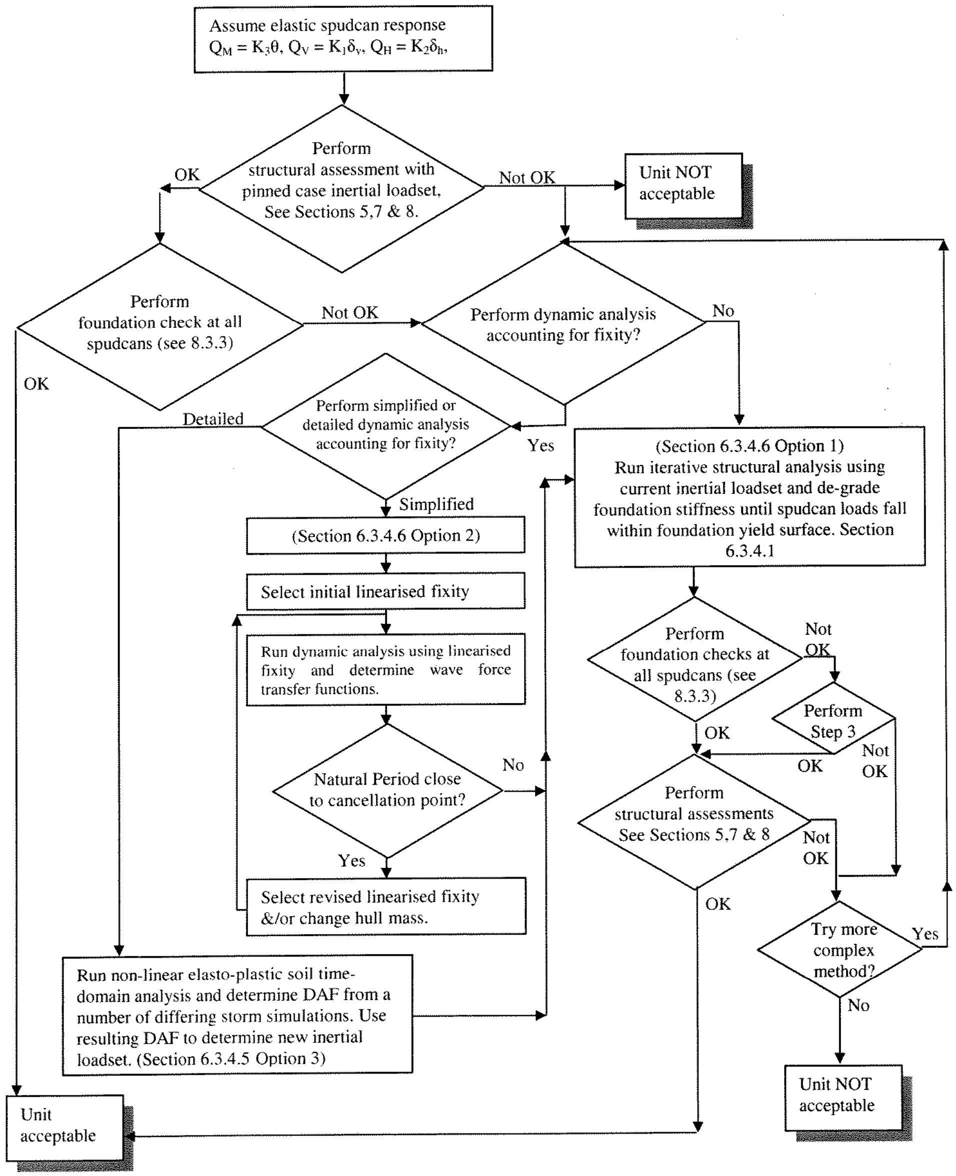


Figure 6.11: Calculation procedure to account for foundation fixity

6.3.5 *Displacement Check (Step 3)*
Structural model with nonlinear soil response included

When a Step 2 assessment results in an overload situation, Step 3 may be used to calculate the associated displacements and rotations from a full nonlinear load-displacement foundation model. The procedure should account for the load redistribution resulting from the overload and displacement of the spudcan(s). The displacements derived from the analysis should be checked against the allowable displacements of the spudcans and should satisfy the following requirements:

- The spudcan vertical and horizontal displacements should not lead to unacceptable overturning or strength checks.
- The resulting rotation of the unit should neither exceed the limitations defined by the operating manual nor lead to the possibility of contact with any adjacent structure.

6.4 Other Aspects of Jack-Up Unit Foundation Performance

6.4.1 *Leaning Instability*

Leaning instability of jack-ups can occur during preloading operations in soft clays where the rate of increase in bearing capacity with depth is small. In deep water a potentially unsafe condition (comparable to a punch-through situation) may occur. However, the potential for such incidents may be discounted if appropriate installation procedures are adopted. These may, for example, include preloading the footings individually.

Further discussion on leaning instability is included in the Commentary.

6.4.2 *Footprint Considerations*

The seabed depressions which remain when a jack-up is removed from a location are referred to as 'footprints'. The form of these features depends on several factors such as the spudcan shape, the soil conditions, the footing penetration achieved and the method of extraction. The shape, and the time period over which the form will exist, will also be affected by the local sedimentary regime.

The positioning of spudcans very close to, or partially overlapping, footprints is not recommended. The difference in resistance between the original soil and the disturbed soil in the footprint area and/or the slope at the footprint perimeter, may cause the spudcans to slide towards the footprint. The resulting leg displacements could cause severe damage to the structure and, at worst, could lead to catastrophic failure. The situation could be complicated by the proximity of a fixed structure or wellhead.

The following two operational sequences may be considered:

a) Installation of an identical jack-up design to that previously used at a particular location:

   If a jack-up with identical footing geometry to the unit previously used is to be installed, the re-positioning should not cause problems provided that the jack-up is located in exactly the same position as for the previously installed unit. Thus the footings would lie in the existing footprints. It is therefore necessary to ensure that reliable records are obtained of the exact location of existing footprints in relation to the well/jacket.

   If the new spudcan positions are not located directly over the footprints sliding of the legs may occur with the potential consequences described above.

b) Installation of a jack-up of different design to that previously used at a particular location:

   It is unlikely for two jack-up designs to have similar footing geometries. It is therefore probable that it will not be possible to locate the spudcans exactly within the existing footprints. 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. Again reliable records of existing footprint locations (and depths) are required.

Where it is not possible to locate the jack-up to avoid spudcan-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 of material stiffness.

Further discussion is included in the Commentary.

6.4.3 *Scour*

Scour may occur when a footing or other object is installed on the seabed, and its presence causes increased local current velocities. The phenomenon is usually observed around spudcans which are embedded to a shallow level in granular materials at locations with high current velocities.

Scour may partially remove the soil from below the footing, resulting in a reduction of the ultimate bearing capacity of the foundation and any seabed fixity. This is normally a gradual process and the effects of the reduced bearing capacity may not be apparent until during storm loading when (rapid) downward movement of the leg may occur. The effects of scour are potentially more severe when it occurs at a location where a potential for punch-through exists.

There is no definitive procedure for the evaluation of scour potential and emphasis must usually be placed on previous operational experience. Further guidance is given in the Commentary.

If scour is recognized to be a potential problem, then preventative measures should be implemented. These should be adopted on a trial basis and include:

a) Gravel dumping prior to installation provided the selected gravel gradation will not cause damage to the jack-up footing.

b) Installation of artificial seaweed.

c) Use of stone/gravel dumping, gravel bags or grout mattresses after installation.

6.4.4 *Seafloor Instability*

Seafloor instability may 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 movement, in the form of mudslides or slope failures. Such phenomena are often associated with deltaic deposits, and it is recommended that the advice of local experts is obtained when such situations are encountered.

Liquefaction, or cyclic mobility, occurs when the cyclic stresses 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.

Such failures may be manifested as continued foundation settlements or large scale failure of the soil mass as described above. In areas where liquefaction is known to be possible its potential must be assessed.

For further guidance refer to the Commentary.

6.4.5 *Shallow Gas*

Gas in soils may originate from biogenic degradation or thermogenic diagenesis.

Gas charged sediments may result in hazards during site investigation soil borings, reduced bearing capacity, unpredictable foundation behavior (due to seabed depressions or gas accumulations under the spudcans) and complications with shallow drilling operations, including blowouts.

The presence of gas charged sediments may be identified by geophysical digital high resolution shallow seismic surveys using attribute analysis techniques.

Any gas concentration should be avoided if it is located above the primary casing shoe level (generally 20 inch or 18.75 inch diameter casing) or the conductor pipe shoe level which are determined during the drilling program design. 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.

Of lesser risk is the potential for gas migration from depth to the surface outside the casing. Although this occurrence is uncommon the potential should not be discounted.

6.4.6 *Spudcan - Pile Interaction*

For jack-ups located in close proximity to pile-founded platforms, soil displacements caused by the spudcan penetration will induce lateral loading into the nearby piles. The amount of soil displacement will depend on the spudcan proximity (spudcan edge-to-pile distance), the spudcan diameter and penetration. If the foundation materials comprise either a deep layer of homogeneous firm to stiff clay or sand and if the proximity of the spudcan to the pile is greater than one spudcan diameter, then no significant pile loading is expected. When the proximity is closer than one spudcan diameter, then analysis by the platform owner is recommended to determine the consequences of the induced pile loading.

Guidance regarding the analytical procedures available for assessing these spudcan induced pile loads is given in the Commentary.

6 GLOSSARY OF TERMS - CALCULATION METHODS, GEOTECHNICAL ENGINEERING

A = Spudcan effective bearing area based on cross-section taken at uppermost part of bearing area in contact with soil (see Figure 6.2).
A_s = Spudcan laterally projected embedded area.
a = Bearing capacity squeezing factor.
a_u = Adhesion.
B = Effective spudcan diameter at uppermost part of bearing area in contact with the soil (for rectangular footing B = width).
b = Bearing capacity squeezing factor.
c_u = Undrained cohesive shear strength at D + B/4 below mudline.
c_{u1} = Undrained cohesive shear strength at spudcan tip.
c_{uo} = Undrained cohesive shear strength at maximum bearing area (D below mudline).
c_{us} = Undrained cohesive shear strength at D/2 below mudline.
$c_{u,b}$ = Undrained cohesive shear strength - lower clay below spudcan.
$c_{u,t}$ = Undrained cohesive shear strength - upper clay below spudcan.
$\left.\begin{matrix} C_1 = \\ C_2 = \end{matrix}\right\}$ Constants used in computation of H_{Lo} and V_{Lo} for sand.
d_c = Bearing capacity depth factor.
= $1 + 0.4\,(D/B)$ for $D/B \leq 1$.
= $1 + 0.4 \arctan(D/B)$ for $D/B > 1$.
d_q = Bearing capacity depth factor.
= $1 + 2\tan\phi(1- \sin\phi)^2\, D/B$ for $D/B \leq 1$
= $1 + 2\tan\phi(1- \sin\phi)^2 \arctan(D/B)$ for $D/B > 1$
d_γ = Bearing capacity depth factor = 1.
D = Distance from mudline to spudcan maximum bearing area.
f_1 = Factor used in yield surface equation for embedded footings on clay.
f_2 = Factor used in yield surface equation for embedded footings on clay.
f_r = Reduction factor on stiffness.
F_o' = Effective overburden pressure due to back-flow at depth of uppermost part of bearing area.
F_H = Horizontal foundation capacity.
F_{HM} = Horizontal foundation capacity in combination with moment.
F_V = Vertical foundation capacity.
$F_{V,b}$ = Ultimate vertical bearing capacity assuming the footing bears on the surface of the lower (bottom) clay layer with no back-flow.
F_{VH} = Vertical foundation capacity in combination with horizontal load.
F_{VHM} = Vertical foundation capacity in combination with horizontal and moment load.
F_M = Moment capacity of foundation.
G_v = Shear Modulus for vertical loading.
G_h = Shear Modulus for horizontal loading.
G_r = Shear Modulus for rotational loading.
h = Distance from rotation point to reaction point.
h_1 = Embedment depth to the uppermost part of the spudcan, (if not fully embedded = 0).
h_2 = Spudcan tip embedment depth.
H = Distance from spudcan maximum bearing area to weak strata below.
H_{Lo} = $(C_1/C_2)(V_{Lo}/4)$, $C_1 = 0.3$, $C_2 = 0.625$ (sand)
= $Ac_{uo} + (c_{uo} + c_{u1})A_s$ (clay)
i_c = Inclination factor (for $\phi = 0$).
= $1 - mF_H/Ac_uN_c$

6 GLOSSARY OF TERMS - CALCULATION METHODS, GEOTECHNICAL ENGINEERING (Continued)

i_q	=	Inclination factor.
	=	$(1 - F_H/F_{VH})^m$
i_γ	=	Inclination factor.
	=	$(1 - F_H/F_{VH})^{m+1}$
I		Height of soil column above spudcan.
k_a	=	Active earth pressure coefficient (for $c_u = 0$) = $\tan^2(45-\phi/2)$
k_p	=	Passive earth pressure coefficient = $1/k_a$
K_1,K_2,K_3	=	Stiffness factors.
K_s	=	Coefficient of punching shear.
L	=	Foundation length, for circular foundation L=B.
m=		For strip footing - inclination in direction of shorter side. $= (2 + B/L)/(1 + B/L)$ For strip footing - inclination in direction of longer side. $= (2 + L/B)/(1 + L/B)$ For circular footing = 1.5
M_{Lo}	=	$C_1V_{Lo}B/4$, $C_1 = 0.3$ (sand)
	=	$0.1V_{Lo}B$ (clay)
M_P	=	moment capacity associated with further spudcan penetration under environmental loading (equal to minimum of M_{PS} and M_{PV}).
M_{PS}	=	moment capacity when further spudcan penetration leads to fully seated spud conditions.
M_{PV}	=	moment capacity under further spudcan penetration, when the actual vertical force is too low to reach fully seated conditions.
n	=	Iteration factor, ≥ 2.
N	=	Stability factor.
N_c	=	Bearing capacity factor (taken as 5.14).
N_q	=	Bearing capacity factor = $e^{\pi\tan\phi}\tan^2(45 + \phi/2)$
$N\gamma$	=	Bearing capacity factor = $2(N_q + 1)\tan\phi$
p_o'	=	Effective overburden pressure at depth, D, of maximum bearing area.
Q_H	=	Applied factored horizontal load.
Q_M	=	Applied factored moment load.
Q_V	=	Applied factored vertical load.
r_f	=	Failure ratio.
s_c	=	Bearing capacity shape factor = $(1 + (N_q/N_c)(B/L))$
s_q	=	Bearing capacity shape factor = $1 + (B/L)\tan\phi$
s_γ	=	Bearing capacity shape factor = $1 - 0.4(B/L)$ (= 0.6 for circular footing under pure vertical load).
T		Thickness of weak clay layer underneath spudcan.
V	=	Volume of soil displaced by spudcan.
V_{Lo}	=	Maximum vertical foundation load during preloading.
α	=	Adhesion factor = 1.0 for soft clays, = 0.5 for stiff clays.
δ	=	Steel/soil friction angle - degrees, ($\phi-5\leq\delta\leq\phi$).
δ_v	=	Vertical displacement of foundation.
δ_h	=	Horizontal displacement of foundation.
γ'	=	Submerged unit weight of soil.
θ	=	Foundation rotation - radians.
ϕ	=	Angle of internal friction for sand - degrees.
ν	=	Poisson's ratio.

7 CALCULATION METHODS - DETERMINATION OF RESPONSES

7.1 General

7.1.1 The response of a jack-up unit is determined by combining the applied loading with a structural model to determine the internal forces in the members and the reactions at the foundations. These are compared with the resistances available to take up these loads to determine the safety of the unit. The loads consist of fixed loads (self weight and non-varying loads) and variable loads (see Section 3.2) together with hydrodynamic and wind loadings (see Section 4). The structural modeling is described in Section 5. The foundation resistance is described in Section 6. Section 8 provides the structural resistance and a methodology to check the adequacy of the various resistances to the acceptance criteria.

7.1.2 Two aspects of the response are to be distinguished and assessed separately. These are:

a) The extreme response. The maximum calculated response to the design environment occurring at a particular instant in time, which is compared with the acceptance criteria. See Sections 7.2 and 7.3.

b) Fatigue. The cumulative effect of stress/strain cycling, which is used to estimate the fatigue lives of steel components (see Section 7.4).

7.1.3 For typical jack-up assessments, the time-varying nature of the wave loading will amplify the quasi-static responses and must be considered. The extreme response can be assessed either by a quasi-static analysis procedure (Section 7.2) including an inertial loadset (Section 7.3.6) or by a more detailed dynamic analysis procedure (Section 7.3.7).

7.1.4 The dynamic amplification of the quasi-static response may not be significant for a given set of location parameters. The magnitude of the dynamic response is primarily influenced by the amount of wave energy at or near the natural period of the jack-up. The distribution of wave energy is at a maximum at the peak wave period and reduces for other periods. Thus the single most important parameter in the determination of the dynamic amplification of responses is the separation of the natural period of the jack-up from the peak period of the wave spectrum. Generally a large separation will produce a small dynamic amplification. As the separation decreases, the dynamic amplification will increase. These conclusions may be modified by effects such as wave-load cancellation and wave-current induced harmonics.

7.1.5 For many applications, the dynamic amplification may be determined using a simple, but empirical, method. This simple method is detailed in Section 7.3.6.1. Caution is advised when relying solely on results using this simple method. Specific guidance on the limitations of the method is given in Section 7.3.6.1.

Because of its simplicity, the method detailed in Section 7.3.6.1 is recommended for an initial evaluation of the dynamic amplification. If the dynamic amplification is determined to be relatively small (see Section 7.3.6.1), or, if acceptance criteria are met, then random dynamic analysis is not required.

7.1.6 For many applications the dynamic effects may be included through the addition of an inertial loadset (see Section 7.3.6.1) to the environmental loads in a quasi-static analysis procedure. In this approach the inertial loadset may be determined using a simplified model of the jack-up. An appropriate detailed model of the jack-up may then be used to determine the detailed responses when the inertial loadset is applied together with the quasi-static environmental loads.

7.1.7 Appropriate combinations of gravity loads, wave/current loads and wind loads shall be applied as required by the acceptance criteria in Section 8. Load application is described in Section 5.7. Section 5.1 requires that the analysis is carried out for a range of environmental headings with respect to the unit such that the most onerous loading(s) for each major type of element in the structural system is(are) determined. The checks cover:

Limit State Check	Section	Response Parameters(s)[1]	Load Component				
			D	L [note 2]		E	D_n
				min	max		
Strength of elements	8.1	Element load vectors[3]	Y	Y[4]	Y	Y	Y
Overturning stability	8.2	Overturning moment Stabilizing moment	5 Y	5 Y		Y	Y
Foundation capacity:	8.3						
- preload	8.3.1	Vertical leg reaction	Y		Y	Y	Y
- sliding	8.3.1	Vertical & Horizontal leg reactions	Y	Y		Y	Y
- bearing	8.3.2/3	Vertical, Horizontal (& moment) leg reactions	Y	Y[6]	Y[6]	Y	Y
- displacement	8.3.4	Leg footing displacements and reactions	Y	Y[6]	Y[6]	Y	Y
Horizontal deflection	8.4	Hull displacement.	Y	Y[6]	Y[6]	Y	Y
Holding system loads	8.5	Holding system loads vectors	Y	Y[6]	Y[6]	Y	Y

where D, L, E and D_n are defined in the glossary at the end of section 7.

Notes:
1. In all instances the responses are evaluated including the effects of deformation under dead loads (hull sag) and large displacement (P-Δ) effects.
2. Placed at most onerous center of gravity position.
3. The effects of leg offset to be added after global response analysis (see Section 5.4).
4. Consider minimum live (variable) load if this is more onerous.
5. Must be included in response calculation so P-Δ effects are included.
6. Worst case combination required.

7.2 Quasi-Static Extreme Response with Inertial Loadset

7.2.1 The most common method of analysis adopted for the determination of extreme responses is the deterministic, quasi-static wave analysis. Such an analysis shall be carried out in accordance with all relevant requirements of Sections 3 to 6. The maximum wave loading is determined by 'stepping' the maximum wave through the structure. The maximum wave is defined in Section 3.5.1.2 and the methodology for calculating the wave loading is described in Section 4.3. Various methods for determining the inertial loadset are given in Section 7.3.6. Load cases and combinations are discussed in Section 7.1.7.

The spudcan-foundation interface should normally be modeled as a pin joint. The inclusion of a degree of fixity is to be justified on a case by case basis. If foundation fixity is included it should generally be represented by a combination of horizontal, vertical and rotational springs (which may be coupled) at the spudcan, rather than by a rotational spring alone. (See also Sections 5.3, 6.3.4 and 7.3.5.2).

7.3 Dynamic Extreme Response

7.3.1 *Factors Governing Dynamics*

Dynamic amplification of the structural response must be taken into account (see Figure 7.1).

Determination of dynamic response requires the incorporation of two separate items in the analysis:

a) The dynamic characteristics of the structural system formed by the jack-up on its foundation,

b) The characteristics of the environmental excitation.

7.3.2 *The Structural System*

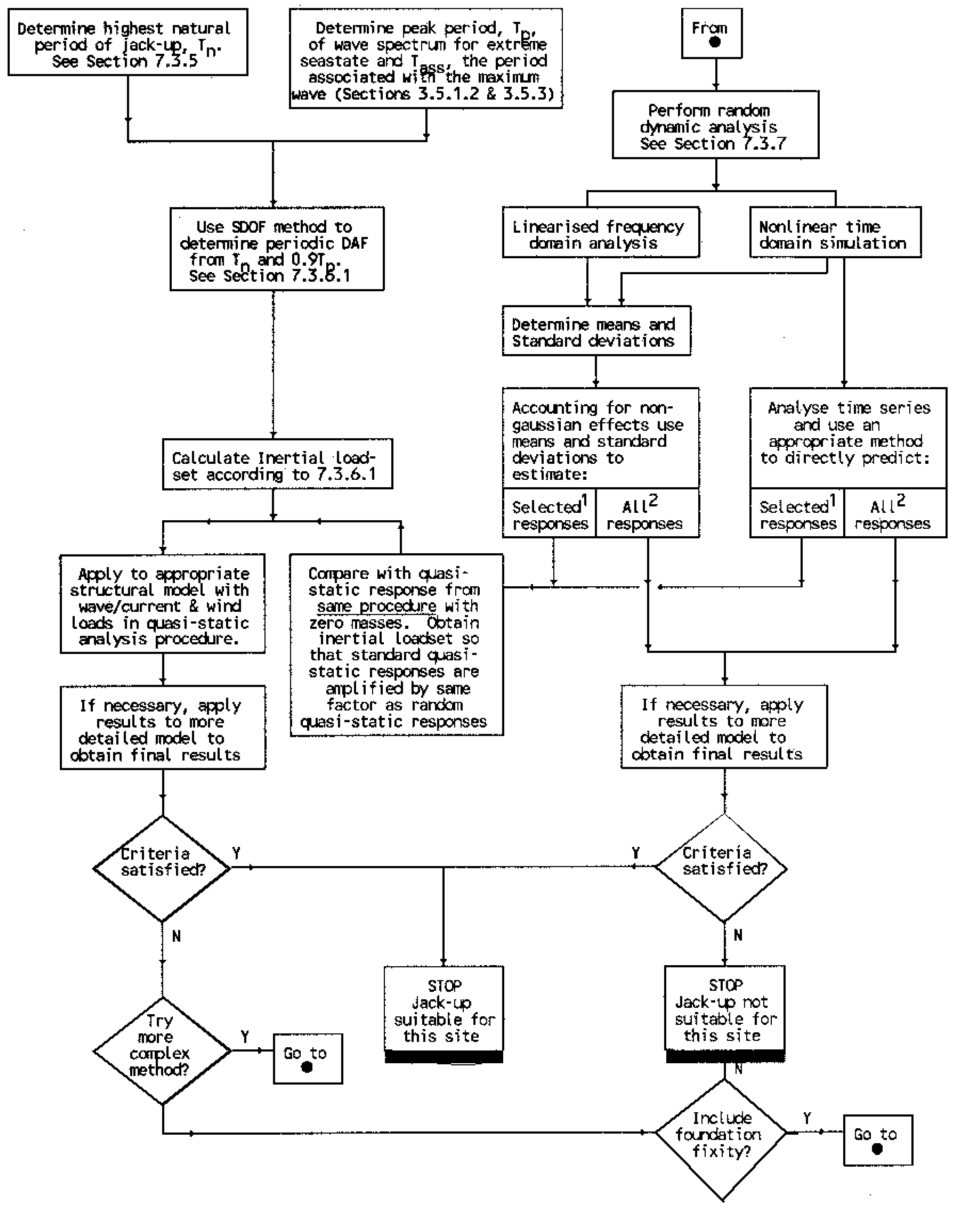
7.3.2.1 The characteristics of the structural system are governed by the following aspects:

a) The mass and mass distribution of the jack-up.
This includes structural mass, mass of equipment and variable load on board, added mass due to the surrounding water and marine growth (if applicable), etc. The magnitudes and effective centers of mass of the various mass contributions are to be accurately determined.

b) The overall (global) structural stiffness.
This includes stiffness contributions from bending, shear deformation and axial straining of the legs, the leg to hull connections, the hull and the spudcan-foundation interface (if applicable).

c) The damping.
Damping contributions arise from the structural components and their connections, the water surrounding the legs and the soil underneath/around the spudcans. For further discussion of damping refer to Section 7.3.7.

7.3.2.2 The jack-up on its foundation represents a multi degree-of-freedom system. If the dynamic behavior is to be investigated in some detail it should also be modeled as such. The model may contain a number of nonlinear elements, notably the leg to hull connections and the spudcan-foundation interfaces. The influence of gravity (P-Δ/Euler) on the effective sway stiffness should be considered (see Section 5.5).



1 The selected response will normally be base shear. This is usually determined from a simple structural model and an additional quasi-static analysis is required to include partial load factors & to determine detailed results.

2 If this path is adopted, the application of partial load factors may be difficult.

Figure 7.1: Recommended approach to determine extreme dynamic responses

Due to the fact that the mass of the hull dominates the mass distribution, the global dynamic behavior of the jack-up may in some cases be determined from an idealized single degree-of-freedom system (see Section 7.3.6.1).

Structural modeling at various levels of complexity is discussed in Section 5.6.

7.3.3 *The Excitation*

7.3.3.1 The characteristics of the environmental excitation are controlled by the fluctuating nature of the environmental factors - wind, current and waves. Currents change slowly compared with the natural periods at which jack-ups may oscillate and may hence be considered to be a steady phenomenon. Variations in wind velocity cover a wide range of periods, but the main wind energy is associated with periods which are considerably longer than the natural periods of jack-up oscillations. Therefore, in connection with jack-ups, the wind may generally be represented as a steady flow of air. The periods of waves typically lie between some 2-3 sec and some 20 sec. Since typical jack-up natural periods fall within this range, the primary source of excitation is from waves.

Sea waves are generally not regular but random in nature unless swell is predominant. This has important implications which should be considered for both the dynamic excitation and the resulting dynamic response.

As waves and currents interact these two environmental factors should be considered in combination when generating time varying hydrodynamic drag forces according to Section 4.3.

7.3.3.2 For the simplified dynamic analysis method of Section 7.3.6.1 based on a regular-wave deterministic quasi-static analysis the wave period is chosen to be $0.9T_p$ where T_p is the peak period of the wave spectrum for the extreme sea state.

For random analyses (see Sections 7.3.6.2 and 7.3.7) the most probable peak period (T_p) of the wave spectrum for the extreme seastate will normally be selected when a 2 parameter Pierson-Moskowitz spectrum is used (H_s and T_p from site specific data and $\gamma = 1$ in Section 3.5.3). If a JONSWAP spectrum is used it is recommended that the peak period is considered to vary between plus and minus one standard deviation from the most probable peak period (T_p).

Where the jack-up is sensitive to the wave period it is recommended that the range described in Section 3.5.1.2 or 3.5.3 is investigated as appropriate.

In a deterministic calculation waves with a period close to the natural period of the jack-up will give the largest dynamic amplification. It is therefore recommended that the wave associated with the highest natural period of the jack-up is also investigated.

7.3.4 *The Dynamic Analysis*

A flow chart indicating the recommended dynamic analysis approach is shown in Figure 7.1.

An initial estimate of the dynamic amplification can be obtained using the empirical methods described in Section 7.3.6.1.

Techniques for performing random dynamic analyses can be categorized as frequency domain methods or time domain (simulation) methods or hybrids thereof; see Section 7.3.7.

7.3.5 *The Natural Period(s)*

7.3.5.1 The natural period of the jack-up on its foundation in the fundamental (or first) mode of vibration is an important indicator of the degree of dynamic response to be expected.

The first and second vibrational modes are nearly always the surge and sway modes. The natural periods of these vibrational modes are usually close together; which of the two is the higher depends on which direction is less stiff. Where the period varies with environmental heading, care should be taken that the period used is applicable to the environmental direction being considered in the analysis. The third vibrational mode is normally a torsional mode, the three-dimensional effects of which may be important, in particular for environmental attack directions where the legs and hence wave loads are not symmetric about the direction of wave propagation.

7.3.5.2 If available, a finite element structural model containing the mass and stiffness properties of the jack-up may be used to obtain the various natural periods and mode shapes. This model should include the stiffness of the legs, hull and hull/leg connections according to Sections 5.6.4 to 5.6.6. If a finite element model containing only stiffness properties is available, then the global sway stiffness for the required headings may be determined by applying lateral unit loads to the hull.

Normally the foundation will be considered pinned. This assumption may however be unconservative for situations in which:

1. The structure natural period is within a cancellation region of the base shear transfer function (see Commentary Section C7.4).

2. Significant foundation nonlinearities are expected at higher loading levels typical of dominant wave frequencies but not at lower loading levels typical of inertial frequencies.

If either of these situations occur, and detailed foundation modeling is not available, it is recommended that the DAF's be calculated with fixity included and are then applied to a pinned model for response calculations.

Where the foundation stiffness is included, lateral and vertical translational springs should be included together with the rotational springs. In any case the limitations on foundation loading according to Section 6.3.4 must be verified.

7.3.5.3 If such a capability is not available, the fundamental mode period may be estimated from the system described by:

- an equivalent mass representing the mass of the jack-up and its distribution as referred to in Section 7.3.2; the equivalent mass is equal to the mass of the hull plus a contribution from the mass of the legs, including added mass, and is located at the center of gravity of the hull.

- an equivalent spring representing the combined effect of the various stiffnesses mentioned in Section 7.3.2.

The period is determined from the following equation applied to one leg:

$$T_n = 2\pi \sqrt{(M_e / K_e)}$$

where;

T_n = highest (or first mode) natural period.

M_e = effective mass associated with one leg.

$$= \frac{M_{hull}}{N} + M_{la} + \frac{M_{lb}}{2}$$

M_{hull} = full mass of hull including maximum variable load.

N = number of legs.

M_{la} = mass of leg above lower guide (in the absence of a clamping mechanism) or above the center of the clamping mechanism.

M_{lb} = mass of leg below the point described for M_{la}, including added mass for the submerged part of the leg ignoring spudcan. The added mass may be determined as $A_e\rho(C_{Me} - 1)$ per unit length of one leg (for definitions of A_e and C_{Me} see Section 4.6.6); ρ = mass density of water.

K_e = effective stiffness associated with one leg (for derivation, refer to Commentary).

$$= \frac{3EI}{L^3} * \frac{\left[1 - \frac{P}{P_E}\right]}{\left[1 - \frac{\frac{3L}{4} - \frac{12F_g I}{AF_v Y^2}\left\{\frac{EI}{K_{rs}} + \frac{L}{2}\right\} - \frac{3(EI)^2}{F_r L K_{rs} K_{rh}}}{\left\{\frac{EI}{K_{rs}} + L + \frac{EI}{F_r K_{rh}}\right\}} + \frac{7.8I}{A_s F_h L^2}\right]}$$

When the soil rotational stiffness K_{rs} at the spudcan-foundation interface is zero this may be re-written:

$$= \frac{3EI}{L^3} * \frac{\left[1 - \frac{P}{P_E}\right]}{\left[1 + \frac{12F_g I}{AF_v Y^2} + \frac{3EI}{F_r L K_{rh}} + \frac{7.8I}{A_s F_h L^2}\right]}$$

K_{rs} = rotational spring stiffness at spudcan-foundation interface.

K_{rh} = rotational stiffness representing leg to hull connection stiffness (see below).

F_r = factor to account for hull bending stiffness.

$$= \frac{1}{\left\{1+\frac{YK_{rh}}{2EI_H}\right\}}$$

I_H = representative second moment of area of the hull girder joining two legs about a horizontal axis normal to the line of environmental action.

E = Young's modulus for steel.

A = axial area of one leg (equals sum of effective chord areas, including a contribution from rack teeth - see Note to Section 5.6.4).

A_s = effective shear area of one leg (see Figure 5.1).

I = second moment of area of the leg (see Figure 5.1), including a contribution from rack teeth (see Note to Section 5.6.4).

Y = distance between center of one leg and line joining centers of the other two legs (3 leg unit).

= distance between windward and leeward leg rows for direction under consideration (4 leg unit)

F_g = geometric factor.

= 1.125 (3 leg unit), 1.0 (4 leg unit)

F_v = factor to account for vertical soil stiffness, K_{vs}, and vertical leg-hull connection stiffness, K_{vh} (see below).

$$= \frac{1}{\left\{1+\frac{EA}{LK_{vs}}+\frac{EA}{LK_{vh}}\right\}}$$

F_h = factor to account for horizontal soil stiffness, K_{hs}, and horizontal leg-hull connection stiffness, K_{hh} (see below).

$$= \frac{1}{\left\{1+\frac{EA_s}{2.6LK_{hs}}+\frac{EA_s}{2.6LK_{hh}}\right\}}$$

L = length of leg from the seabed reaction point (see Section 5.2.1) to the point separating M_{1a} and M_{1b} (see above).

P = the mean force due to vertical fixed and variable loads acting on one leg.

$$= \frac{M_{hull}\,g}{N}$$

g = acceleration due to gravity.

P_E = Euler buckling load of one leg.
= $\alpha^2 EI$
α = the minimum positive non-zero value of αL satisfying:

$$\tan(\alpha L) = \left\{ \frac{(K_{rs} + K_{rh})\alpha EI}{(\alpha EI)^2 - (K_{rs}K_{rh})} \right\}$$

Thus:

when $K_{rs} = 0$ and $K_{rh} = \infty$, $\alpha L = \pi/2$ and hence:

$$P_E = \frac{\pi^2 EI}{4L^2}$$

when $K_{rs} = \infty$ and $K_{rh} = \infty$, $\alpha L = \pi$ and hence

$$P_E = \frac{\pi^2 EI}{L^2}$$

The hull to leg connection springs, K_{rh}, K_{vh} and K_{hh} represent the interaction of the leg with the guides and supporting system and account for local member flexibility and frame action. They should be computed with respect to the point separating M_{1a} and M_{1b}, as described above. The following approximations may be applied:

K_{hh} = ∞
K_{vh} = effective stiffness due to the series combination of all vertical pinion or fixation system stiffnesses, allowing for combined action with shock-pads, where fitted.

Unit with fixation system:

K_{rh} = combined rotational stiffness of fixation systems on one leg.
= $F_n h^2 k_f$

where;

F_n = 0.5, three chord leg; = 1.0, four chord leg
h = distance between chord centers.
k_f = combined vertical stiffness of all fixation system components on one chord.

Unit without fixation system:

K_{rh} = rotational stiffness allowing for pinion stiffness, leg shear deformation and guide flexibility.

$$= F_n h^2 k_j + \frac{k_u d^2}{1 + (2.6 k_u d / EA_s)}$$

where;

h = distance between chord centers (opposed pinion chords) or pinion pitch points (single rack chords).
k_j = combined vertical stiffness of all jacking system components on one chord.
d = distance between upper and lower guides.
k_u = total lateral stiffness of upper guides with respect to lower guides.
A_s = effective shear area of leg.

7.3.5.3 The above equations for estimating the fundamental natural period are approximate and ignore the following effects:

- more realistic representation of possible fixity at the spudcan-foundation interface in the form of (coupled) horizontal, vertical and rotational spring stiffnesses.

- three dimensional influences of the system as compared with the two-dimensional single leg model.

7.3.5.4 Due to uncertainty in the parameters affecting the natural period the calculated natural period(s) will also be uncertain. The natural period(s) used in the dynamic analysis should be selected such that a realistic but conservative value of the dynamic response is obtained for the particular application envisaged. Care should be taken to ensure that the maximum dynamic amplification is not selected as coincident with a cancellation period causing minimum environmental loading. The potential for increased response due to shortcrested waves should be considered (see Section 7.3.7.5). For further details refer to the Commentary Section C7.4 and Figure C7.1.

7.3.6 <u>*Inertial Loadset Approaches*</u>

In inertial loadset approaches the dynamic response is represented in a global quasi-static response model by either a distributed inertial loadset or an equivalent point load applied at the hull center of gravity. The inertial loadset may be derived from the simple approach described in Section 7.3.6.1 or from the more complex methods discussed in Sections 7.3.6.2 and 7.3.6.3.

7.3.6.1 *The classical SDOF analogy*

This representation assumes that the jack-up on its foundation may be modeled as an equivalent mass-spring-damper mechanism; see Section 7.3.2. The (highest) natural period of the vibrational modes may be determined as described in Section 7.3.5. The torsional mode and corresponding three-dimensional effects cannot be included in this representation.

The single degree-of-freedom (SDOF) method is fundamentally empirical because (1) the wave-current loading does not occur at the mass center and (2) the loading is non-periodic (random) and non-linear.

It should also be noted that all global and detailed response parameters are not equally amplified. The method described below will generally lead to a reasonable approximation of the jack-up's real behavior and has been calibrated against more rigorous methods. The following cautions are noted when using the SDOF method:

1. If the ratio of the jack-up natural period to the wave excitation period, Ω, is less than 0.5 and the current is 'relatively small' the SDOF method should give reasonably accurate results when compared to a more rigorous analysis.
2. If Ω is greater than 0.5, the relative position of the jack-up natural period within the base shear transfer function should be checked. If the natural period falls near a wave force peak, then the SDOF method may be unconservative because it ignores forcing at other than the full wave excitation period. Note that the calculation of natural periods should include a range of periods to account for a reasonable estimate of foundation fixity (see Section 7.3.5.2).
3. The SDOF method may be unconservative for cases with relatively high currents. If the results of the assessment are close to the acceptance criteria further detailed analysis is recommended.

The ratio of (the amplitudes of the) dynamic to the quasi-static response as a function of frequency (ω) or period (T) of steady state, periodic and sinusoidal excitation is calculated as the classical dynamic amplification factor (DAF):

$$\mathrm{DAF} = \frac{1}{\sqrt{\left[(1-\Omega^2)^2 + (2\zeta\Omega)^2\right]}}$$

where;

$$\Omega = \frac{\text{Wave Excitation frequency}}{\text{Jack – up natural frequency}} = \frac{\omega}{\omega_n}$$

$$= \frac{\text{Jack - up natural period}}{\text{Wave excitation period}} = \frac{T_n}{T}$$

ζ = Damping ratio or fraction of critical damping
= (% Critical Damping)/100, ≤ 0.07.
T = $0.9T_p$.
T_p = most probable peak wave period.
T_n = the jack-up natural period as derived in 7.3.5.

The damping parameter ζ in this model represents the total of all damping contributions (structural, hydrodynamic and soil damping). For the evaluation of extreme response using the SDOF method a value not exceeding 0.07 is recommended.

The calculated DAF from the SDOF method is used to estimate an inertial loadset which represents the contribution of dynamics over and above the quasi-static response in accordance with Figure 7.1. This inertial loadset should be determined as follows and applied at the hull (center of gravity) in the down-wind direction:

$$F_{in} = (\mathrm{DAF} - 1)\, BS_{Amplitude}$$

where;

F_{in} = Magnitude of the inertial loadset for use in conjunction with the SDOF method.
$BS_{Amplitude}$ = Amplitude of quasi-static Base Shear over one wave cycle.
= $(BS_{(Q-S)Max} - BS_{(Q-S)Min})/2$
$BS_{(Q-S)Max}$ = Maximum quasi-static wave/current Base Shear.
$BS_{(Q-S)Min}$ = Minimum quasi-static wave/current Base Shear.

Note: The above equation is part of a calibrated procedure and should not be altered. A more general inertial loadset procedure, using the results from random analysis, is described in Section 7.3.6.3.

7.3.6.2 *Other SDOF approaches*

An alternative use of the SDOF method is to apply the entire DAF function for all frequencies (periods), rather than a single point DAF at one frequency. This method reflects the random wave plus current excitation more correctly. Execution of this procedure is as per the relevant parts of Section 7.3.7.

7.3.6.3 Inertial loadset based on random analysis

The inertial loadset may be derived from random frequency or time domain analysis according to the recommendations of Section 7.3.7. The inertial loadset should be such that it increases the responses of the deterministic quasi-static analysis by the same ratios as those determined between the random quasi-static (zero mass) analysis and the random dynamic analysis (see Figure C7.B.1) In such cases the structural model (used for dynamic analysis) may be simplified and does not need to contain all the structural details, but will nevertheless be a multi degree-of-freedom model. The approach to the modeling and determination of the inertial loadset is described further in the Commentary, Section C7.B.2.

The inertial loadset can be determined to model the effect of dynamic amplification in a more realistic manner as required. The simplest alternative uses a single point force to match inertial overturning moment effects as shown in the Commentary, Section C7.B.2. However the use of a distributed inertial loadset is considered more representative and will therefore provide a more accurate description of the component dynamic amplification effects as well as global response amplification. The distribution of the loadset is based on the fundamental sway modes and mass distribution. Note that the use of a distributed inertial loadset is recommended for units where a significant proportion of the total mass (including fluid added mass) acts at a location other than the hull center of gravity. The mathematical procedure for calculation of the distributed loadset is given in Figure 7.2. A brief description of the calculation process is as follows:

Step 1
Perform random response analysis using a wave attack direction along the selected main axis (x or y) and establish the global response dynamic amplification factors for base shear and overturning moment, whereby the dynamic amplification factors are defined as $DAF_3 = MPME_{dyn}/MPME_{static}$.

Step 2
Establish a set of two simultaneous equations using combinations of 2-D mode shapes, nodal masses and unknown modal scalar, which match the inertial base shear and moment along the selected main axis. Solve this equation set to determine the two modal multipliers.

Step 3
Establish the (2-D) inertial loadset F_{in} by a combination of the selected structural mode shapes (φ_1, φ_2), scalar multipliers (α, β) and nodal masses (M), i.e. $F_{in} = \alpha\, \varphi_1 M + \beta\, \varphi_2 M$.

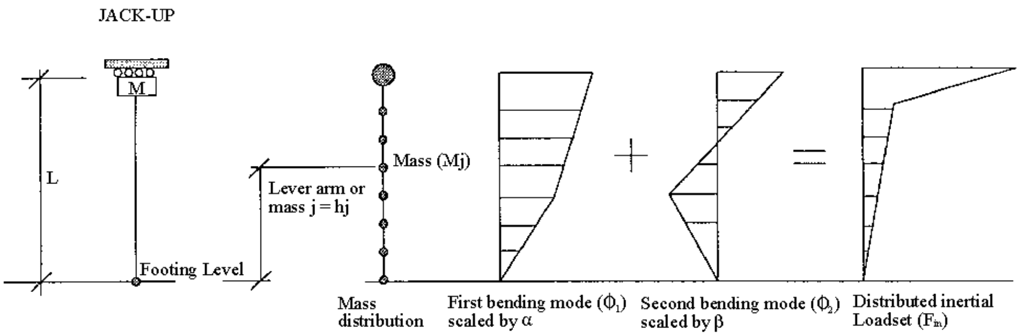


Figure 7.2 - Procedure for calculation of distributed inertial loadset (2-D response)

7.3.7 *Detailed Dynamic Analysis Methods*

Fully detailed random dynamic analysis will be necessary indicated in Figure 7.1. Random dynamic analysis may be performed in the time or in the frequency domain.

7.3.7.1 The waves may be modeled as a linear random superposition model which is fully described by the wave spectrum (see Section 3.5.3). The statistics of the underlying random process are gaussian and fully known theoretically. An empirical modification around the free surface may be needed to account for free surface effects. This, together with the fact that drag forces are a nonlinear (squared) transformation of wave kinematics, makes the hydrodynamic force excitation always nonlinear. As a result, the random excitation is non-gaussian. The statistics of such a process are generally not known theoretically, but the extremes are generally larger than the extremes of a corresponding gaussian random process. For a detailed investigation of the dynamic behavior of a jack-up the non-gaussian effects must be included. A number of procedures for doing this are presented in the Commentary.

7.3.7.2 The spudcan-foundation interface should normally be modeled as a pin joint in the absence of justifiable site-specific foundation fixity information, but see Section 7.3.5.2.

If foundation fixity is included, it should be represented by a combination of horizontal, vertical and rotational springs. Coupling of the springs is preferable. In any case the limitations on foundation loading according to Section 6.3.4 must be verified.

7.3.7.3 When the random displacements of the submerged parts are small and the velocities are significant with respect to the water particle velocities the damping is not well represented by the relative velocity formulation in Morison's equation, which will tend to overestimate the damping and underpredict the response. A criterion for determining the applicability of the relative velocity formulation is given in Section 4.3.2.

7.3.7.4 Table 7.1 summarizes appropriate percentages of global critical damping for the various damping sources which should be summed to provide the total global damping as a percentage of critical damping.

Damping source	Global damping not to exceed (% of critical damping)
Structure, holding system, etc.	2%
Foundation	2% or 0%(1)
Hydrodynamic	3% or 0%(2)

Notes:

1. Where a non-linear foundation model is adopted the hysteresis foundation damping will be accounted for directly and should not therefore be included in the global damping.

2. In cases where the relative velocity formulation may be used ($\alpha = 1$ in Section 4.3.2) the hydrodynamic damping will be accounted for directly and should not therefore be included in the global damping.

Table 7.1 - Recommended damping from various sources

7.3.7.5 The effects of directionality and wave spreading may be considered in any dynamic analysis. It is recommended that a comparison be made between the Base Shear Transfer Function (BSTF) for the chosen 2-D (long crested/unspread) analysis direction and the 3-D (short crested/spread) BSTF to determine whether the selected direction is unconservative. Optimally the direction of the 2-D seastate should be chosen to obtain a match with the 3-D BSTF for the entire wave spectrum. If this is not possible the match between the spread and unspread BSTFs should be good at the natural period.

A 3-D BSTF, H_{3D}, can be generated from a set of 2-D BSTFs, H_{2D}, by the following expression:

$$H_{3D}(\omega) = \int_0^{2\pi} \left[H_{2D}(\omega,\theta)\right]^2 \cos^{2n}(\theta)d\theta$$

where:

ω = Wave excitation frequency
θ = Angle between 2-D BSTF and dominant direction of 3-D BSTF
n = Power constant of spreading function
$\geq$ 2.0 for fatigue analysis
$\geq$ 4.0 for extreme analysis

A simple approximation to the incorporation of wave spreading into inertial load calculations is to perform a 2-D analysis with the wave approach angle which is between the two approach angles which give the maximum and minimum forces at the cancellation and reinforcement points (see Figure C7.1 in the Commentary).

7.3.7.6 Tables 7.2 and 7.3 respectively identify the most important factors associated with each type of analysis method and with each approach to determining the extreme responses. Further details of the methods are provided in the Commentary.

7.3.8 *Acceptance Criteria*

The results of a dynamic extreme response analysis shall be assessed against the acceptance criteria described in Section 8. The required load factors should be introduced when combining the component loads into total load combinations.

Method	Recommendations
Frequency Domain	Consider linearization assumptions with respect to: - wave-current loading (quadratic dependence on particle velocity and finite wave ht). - structural non-linearity. Generate random sea from at least 200 components and use divisions of equal frequency. <u>Note:</u> fewer frequency components may be used provided that the divisions are shown to be sufficiently small around the wave period, the natural period & periods associated with reinforcement and cancellation.
Time Domain	Generate random sea from at least 200 components and use divisions of generally equal energy. It is recommended that smaller energy divisions are used in the high frequency portion of the spectrum, which will generally contain the reinforcement and cancellation frequencies. Each wavelet should be taken to disperse with its own linear dispersion relationship [12] Check validity of wave simulation: - correct mean wave elevation - standard deviation = $(H_s /4) \pm 1\%$ - $-0.03 <$ skewness < 0.03 - $2.9 <$ kurtosis < 3.1 - Max crest elevation = $(H_s/4)\sqrt{2\ln(N)}$ -5% to +7.5% where N is the number of cycles in the time series being qualified, $N \approx \text{Duration} / T_z$ Integration time-step less than the smaller of: $T_z/20$ or $T_n/20$ where; T_z = the zero-upcrossing period of the wave spectrum T_n = the jack-up natural period (unless it can be shown that a larger time-step leads to no significant change in results) Avoid transients in 'run-in' (≥100 secs). Ensure simulation length OK for method chosen to determine the Most Probable Maximum Extreme (MPME) response(s). <u>Note:</u> The MPME is defined in Table 7.3

<u>Table 7.2 - Recommendations for application of dynamic analysis methods (see Commentary)</u>

Method	Recommendations
General	Define the Most Probable Maximum Extreme (MPME) as the extreme with a 63% chance of exceedence (typically this is the mode or highest point on the probability density function (PDF)).This is approximately equivalent to the 1/1000 highest peak level in a 3-hour storm.
Frequency Domain	Use mean & standard deviation to determine drag-inertia parameter and use Figure C7.B.6 in Commentary Section C7.B.2.1.
Time Domain	Use mean & standard deviation to determine drag-inertia parameter and use Figure C7.B.5 or Figure C7.B.6 in Commentary Section C7.B.2.1. Simulation time of at least 60 minutes usually required to obtain stable standard deviation. or Fit Weibull distribution to distribution, for 3-hour probability level. Take results as average of MPME's from ≥ 5 simulations. Each input wave simulation to be of sufficient length for recommendations of Table 7.2 to be met (usually at least 60 minutes). See Commentary C7.B.2.2. or Use multiple 3-hour simulations and use Gumbel distribution on the extreme from each simulation. Sufficient simulations (usually at least 10) are required to obtain stable MPME of responses. See Commentary C7.B.2.3. or Use Winterstein's Hermite polynomial model, with improvements by Jensen if Kurtosis > 5. Simulation of sufficient duration to provide stable skewness and kurtosis of responses (normally in excess of 180 minutes). See Commentary Section C7.B.2.4.

Table 7.3 - Recommendations for determining MPME (see Commentary)

7.4 Fatigue

7.4.1 *General*

The fatigue of jack-ups should be considered for all new locations and operations. Jack-ups are mobile structures, generally operating in a wide range of water depths, therefore the location of the fatigue sensitive areas may vary (see Section 7.4.3). This means that fatigue damage at any member/joint or other component may not occur equally throughout the life of the unit and tends to complicate the fatigue problem.

If the original analysis carried out for the unit demonstrates that lives of critical components are adequate then a unit may not require a separate analysis if on location for a period of less than one year provided that adequate proof from a recent inspection exists showing that the unit is behaving as originally predicted.

If no original analysis and/or inspection proof is in existence then a separate analysis may be required for all operations in excess of one year. In extreme cases six months may be more appropriate if this period contains the rough winter season. Alternatively a recent assessment inspection, or proof that such an inspection (including detailed NDT) has been carried out may serve as a demonstration of the adequacy of the unit.

7.4.2 *Fatigue Life Requirements*

A fatigue analysis, if undertaken, should ensure that all structural components have (remaining) fatigue lives of more than the greater of four times the duration of the assignment or 10 years. Different (reduced) fatigue life requirements may be justified for certain items on a case by case basis where structural redundancy or ease of access for inspection and repair permit.

7.4.3 Fatigue Sensitive Areas

All structural members subject to fatigue loading are to be checked in the analysis, with emphasis on the following areas, which are likely to be the most critical. However, other areas should also be studied if they are potentially more critical:

a) The leg members and joints in the vicinity of the upper and lower guides for the operating leg/guide location(s).
b) The rack teeth of the chord.
c) The leg members and joints adjacent to the waterline.
d) The jack-frame/jackhouse and associated areas of the hull.
e) The leg members and joints in the vicinity of the leg to spudcan connection.
f) The spudcan to leg connection.

Records of inspections, damage and repair for the unit may provide guidance in the selection of critical areas.

As mentioned the fatigue analysis should consider all loading conditions that may occurduring the period under consideration and for items c) through f) the cumulative damage due to transit loadings should also be included.

7.4.4 General Description of Analysis

Suitable approaches to the analysis may be found in reference [13]. Equivalent approaches may be applied.

7 GLOSSARY OF TERMS - DETERMINATION OF RESPONSES

A	=	Equivalent axial area of a leg (see Figure 5.1), including contribution from rack teeth (see note to Section 5.6.4).
A_s	=	Effective shear area of one leg.
BS	=	Base Shear.
d	=	Distance between upper and lower guides.
D	=	Self weight and non varying loads.
DAF	=	Dynamic Amplification Factor.
D_n	=	Inertial loads due to Dynamic response.
E	=	Environmental loads.
E	=	Young's modulus for steel.
F_g	=	Geometric factor
	=	1.125 (3 leg unit), 1.0 (4 leg unit)
F_h	=	Factor to account for horizontal soil stiffness, K_{hs}, and horizontal leg-hull connection stiffness, K_{hh}.
F_{in}	=	Magnitude of inertial loadset.
F_n	=	0.5, three chord leg; = 1.0, four chord leg
F_r	=	Factor to account for hull bending stiffness.
F_v	=	Factor to account for vertical soil stiffness, K_{vs}, and vertical leg-hull connection stiffness, K_{vh}.
g	=	Acceleration due to gravity.
h	=	Distance between chord centers or pinion pitch points.
H_{det}	=	The wave height to be used for deterministic waveforce calculations, allowing for conservatisms in the theoretical predictions of higher order wave theories.
	=	1.60 H_{srp}
H_{max}	=	The maximum deterministic wave height.
	=	1.86 H_{srp}, generally.
	=	1.75 H_{srp}, in Tropical Revolving Storm areas.
H_s	=	Significant wave height (meters), including depth/asymmetry correction, according to Section 3.5.1.1.
H_{srp}	=	The assessment return period significant wave height for a 3 hour storm.
H_{2D}	=	2-D base shear transfer function.
H_{3D}	=	3-D base shear transfer function.
I	=	Second moment of area of the leg (see Figure 5.1) including contribution from rack teeth (see note to Section 5.6.4).
I_H	=	Representative second moment of area of the hull girder joining two legs about a horizontal axis normal to the line of environmental action.
k_f	=	Combined vertical stiffness of all fixation system components on one chord.
k_j	=	Combined vertical stiffness of all jacking system components on one chord.
k_u	=	Total lateral stiffness of upper guides with respect to lower guides.
K_e	=	The effective stiffness associated with one leg.
K_{hh}	=	Horizontal stiffness of leg-hull connection, generally infinite.
K_{hs}	=	Horizontal stiffness at the spudcan-foundation interface.
K_{rh}	=	Rotational stiffness representing the leg-hull connection.
K_{rs}	=	Rotational stiffness at the spudcan-foundation interface.
K_{vh}	=	Vertical stiffness of leg-hull connection.
K_{vs}	=	Vertical stiffness at the spudcan-foundation interface.
L	=	Variable loads.
L	=	Length of leg from the seabed reaction point (see Section 5.2.1) to the point separating M_{1a} and M_{1b}.

7 GLOSSARY OF TERMS - DETERMINATION OF RESPONSES (Continued)

M = Nodal masses.
M_e = Effective mass associated with one leg.
M_{hull} = Full mass of hull, including variable load.
M_{1a} = Mass of a leg above lower guide (in the absence of a clamping mechanism) or above the center of the clamping mechanism.
M_{1b} = Mass of leg below the point described for M_{1a}, including added mass for the submerged part of the leg.
MPME = Most Probable Maximum Extreme response(s). The extreme response with a 63% chance of exceedence; approximately equal to the 1/1000 highest peak level in a 3-hour storm.
n = Power constant of spreading function.
$\geq$ 2.0 for fatigue analysis.
$\geq$ 4.0 for extreme analysis.
N = Number of legs.
N = Number of cycles.
P = The mean force due to vertical dead weight and variable load acting on one leg.
$= \dfrac{M_{hull}\, g}{N}$
P_E = Euler buckling load of one leg.
$= \alpha^2 EI$
T = $0.9\ T_p$.
T_{ass} = Wave period associated with H_{max} (also used with H_{det}).
T_n = Natural period of jack-up (subject to the precautions of Section 7.3.5.4).
T_p = Peak period associated with H_{srp} (also used with H_s).
T_z = Zero-upcrossing period of the wave spectrum.
Y = Distance between center of one leg and line joining centers of the other two legs (3 leg unit).
= Distance between windward and leeward leg rows for direction under consideration (4 leg unit).
α = The minimum positive non-zero value of αL satisfying:

$$\tan(\alpha L) = \left\{ \frac{(K_{rs} + K_{rh})\alpha EI}{(\alpha EI)^2 - (K_{rs} K_{rh})} \right\}$$

α = Scalar multiplier used in establishing 2-D F_{in}.
β = Scalar multiplier used in establishing 2-D F_{in}.
φ_1, φ_2 = Structural mode shapes.
Ω = $\omega/\omega_n = T_n/T$.
ρ = Mass density of water.
θ = Angle between 2-D BSTF and dominant direction of 3-D BSTF.
ω = Wave excitation frequency = $2\pi/T$.
ω_n = Jack-up natural frequency = $2\pi/T_n$.
ζ = Damping ratio or fraction of critical damping.

8 ACCEPTANCE CRITERIA

The acceptance checks in the following sections cover:

- Structural strength (Section 8.1),
- Overturning stability (Section 8.2),
- Foundation capacity (preload, bearing, sliding displacement and punch-through) (Section 8.3),
- Horizontal deflections (Section 8.4),
- Loads in the holding system (Section 8.5),
- Loads in the hull (Section 8.6) and
- The condition of the unit (Section 8.7).

In each check the factored resistance should equal or exceed the factored load. Thus the general form of the check is:

$$\frac{\sum(\text{Factored loads})}{\text{Factored resistance}} \leq 1.0$$

For some checks, where load and resistance vectors are considered, it may be necessary to address the interaction between the n different components. The form of the check then becomes:

$$\sum_{i=1}^{n} fn\left\{\frac{\sum(\text{Factored loads in component i})}{\text{Factored resistance to component i}}\right\} \leq 1.0$$

The required load factors are as follows:

γ_1 = 1.00 - Applies to non-varying weight loads (D)

γ_2 = 1.00 - Applies to maximum or minimum variable loads (L) applicable to check being carried out

γ_3 = 1.15 - Applies to environmental loads (E); (**provisional - see Section 1.8**);

γ_4 = 1.00 - Applies to dynamic loads (D_n) in combination with γ_3

It is assumed that the jack-up is built to recognized standards, and has been maintained as required to continue to meet those standards (see Sections 2.4.2 and 8.7). Any deterioration should be taken into account in the assessment.

8.1 Structural Strength Check

Note: Figure 8.1 provides a flowchart for member strength assessment.

8.1.1 *Introduction*

8.1.1.1 *Code Basis*

The main basis for the structural strength check is the AISC 'Load and Resistance Factor Design (LRFD) Specification for Structural Steel Buildings' [14]. The AISC LRFD specification has been interpreted and, in some cases, modified for use in the assessment of mobile jack-up unit structures. Interpretation of the code has been necessary to enable a straight-forward method to be presented for the assessment of beam-columns of non 'I' section. Development of the code has been necessary in two areas as described below:

a) A method has been established for dealing with sections constructed of steels with different material properties.

b) A method has been established for the assessment of beam columns under biaxial bending to overcome a conservatism which has been identified in the standard AISC LRFD equations.

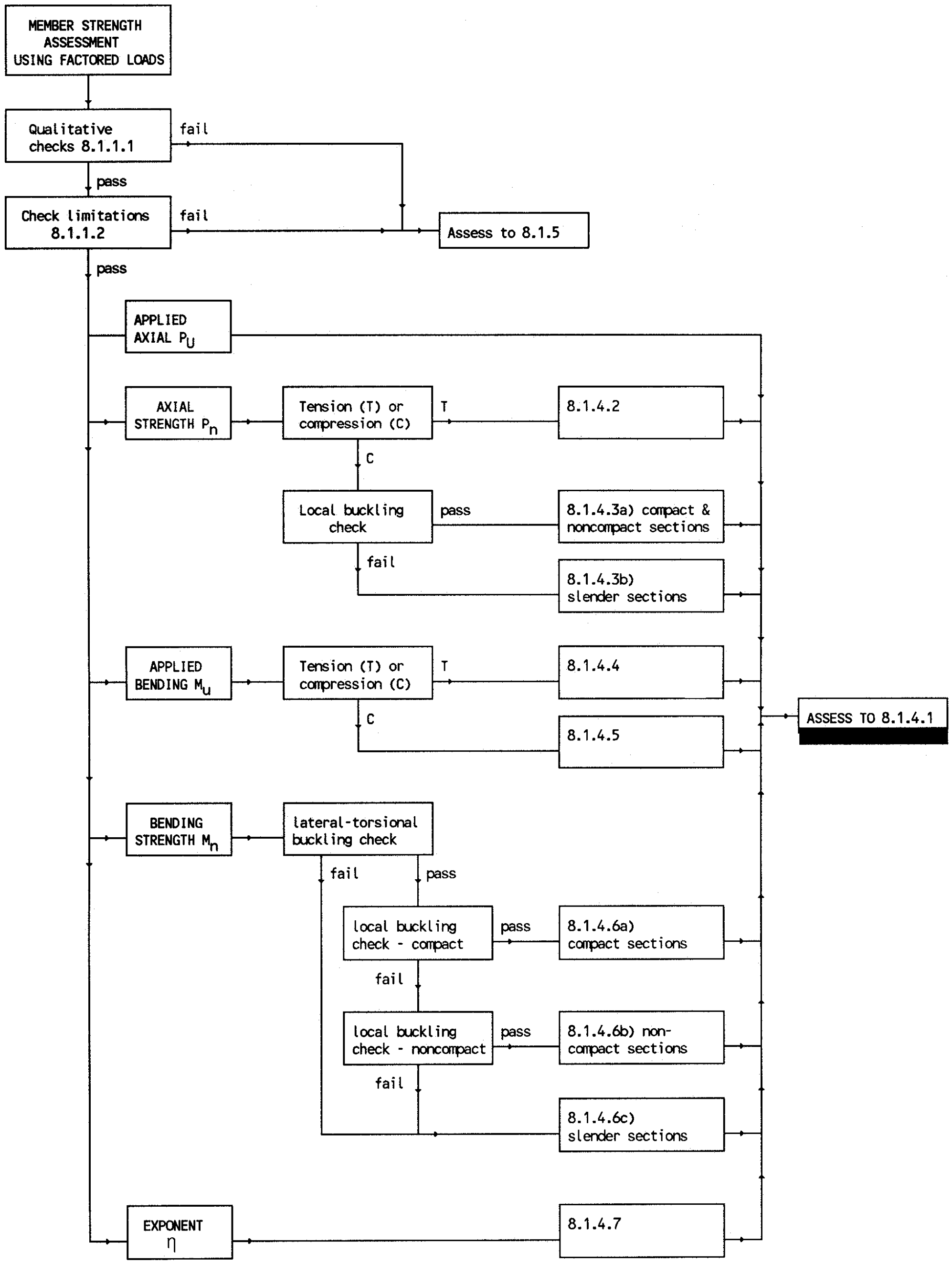


Figure 8.1: Flow chart for member strength assessment

One particular type of member geometry which is not covered at all by AISC LRFD is the high R/t ratio tubular which usually has ring frame and/or longitudinal stiffeners. Recommendations for checking such members are given in Section 8.1.5 where the user is referred to an applicable code and guidance is given on suitable load and resistance factors.

The resistance factors used in the AISC LRFD specification have been adopted.

In addition to checking the strength of members, it may be necessary to check the strength of joints between members. Recommendations for joint checking are given in Section 8.1.6 where the user is referred to an applicable code and guidance is given on suitable load and resistance factors.

8.1.1.2 *Limitations*

The structural strength check assessment described here is limited by the following criteria:

a) The geometry of structural components and members, as defined in 8.1.2, must fall reasonably within the categories described in that section.

b) In accordance with AISC LRFD Specification, Chapter A Para. A5, the minimum specified yield stress of the strongest steel comprising the components and members should not exceed:

- 65 ksi (448 MN/m^2) if (elasto-)plastic structural analysis is used to determine the member loads. For slender geometries plastic structural analysis is precluded, even if the yield stress is below 65 ksi.

- 100 ksi (690 MN/m^2) if elastic structural analysis is used to determine the member loads.

For higher strength steels within the holding system, refer to Section 8.5.

It should also be noted that the assessment has been tailored towards the types of analysis normally carried out for jack-ups. The detailed recommendations which follow focus particularly on closed section brace and chord scantlings in truss type legs.

Geometries outside the limits of Sections 8.1.2 - 8.1.4 may be checked in accordance with the recommendations of Section 8.1.5.

<u>Notes:</u>

1. Of necessity, many of the equations presented in Section 8.1 are dimensional. Such equations are quoted firstly in metric units (MN, m, MN/m^2 etc.) and then in { } in North American imperial units (kips, inches, ksi, etc.).

2. Where the member geometry may contain components of part-tubular shape it is appropriate to consider their dimensions in terms of radius and thickness (rather than diameter and thickness), and hence relevant equations have been converted to this format.

3. The AISC LRFD source equations/text are identified between [ ].

4. The terms in the equations are defined where they appear. A glossary is also provided at the end of Section 8.

8.1.2 <u>*Definitions*</u>

8.1.2.1 *Structural Members and Components*

a) <u>Structural Members</u>

For the purposes of strength assessment, it is necessary to consider the structure as comprised of structural members. Typically each structural member could be represented by a single finite element in an appropriate finite element model of the structure. Examples of members would include braces and chords in truss type legs, box or tubular legs and plating which forms a piece of structure for which the properties can readily be calculated.

The strengths of structural members are to be assessed according to Section 8.1.4 with the exception of structural members exceeding any of the following provisions which should be assessed according to Section 8.1.5.

i) A plain tubular with $R/t > 44{,}815/F_y$
{Imperial: $6{,}500/F_y$}
[Table A-F1.1]

ii) Any tubular with ring stiffeners with or without longitudinal stiffeners.

iii) Tubulars with longitudinal stiffeners where;
$R/t > 11{,}375/F_y$ {Imperial: $1{,}650/F_y$}
[Table B5.1]

b) <u>Structural Components</u>

A structural component is defined as a part of a structural member (see Figure 8.2). Typically, structural components are pieces of plating or tubulars such as the plates, split-tubulars and rack pieces forming a jack-up chord, or the stringers on a panel. Note that it is not always appropriate to consider fundamental structural parts as components. A plain tubular, for example is better analyzed as a member. A component should not consist of more than one material.

8.1.2.2 *Stiffened and Unstiffened Components*

A component which is stiffened along both edges is denoted a stiffened component. A component which is supported along only one edge is denoted an unstiffened component. Typically all the components forming parts of chord sections may be regarded as stiffened.

8.1.2.3 *Compact, noncompact and slender sections*

Steel sections are divided into compact sections, noncompact sections and sections with slender compression elements. Compact sections are capable of developing a fully plastic stress distribution before the onset of local buckling. Noncompact sections can develop the yield stress in compression components before local buckling occurs, but will not resist inelastic local buckling at the strain levels required for a fully plastic stress distribution. Slender compression components buckle elastically before the yield stress is achieved.

Where a distinction is required between these categories, appropriate limiting slenderness ratios have been stipulated.

8.1.3 Factored Loads

Factored loads in structural components and members are to be determined in accordance with the previous sections, using the most onerous condition for each structural component or member.

Each structural component or member should be checked for the factored load vector Q (i.e. axial load, moments and, if applicable, shears and torsion) where;

$$Q = \gamma_1.D + \gamma_2.L + \gamma_3(E + \gamma_4.D_n)$$

and

γ_1 = 1.0
γ_2 = 1.0
γ_3 = 1.15 (**provisional - see Section 1.8**)
γ_4 = 1.0

D = Member load vector due to the weight of structure and non-varying loads including:

- Weight in air including appropriate solid ballast.
- Equipment.
- Buoyancy.
- Permanent enclosed liquid.

L = Member load vector due to the maximum variable load (gravity adds to environmental loads) or minimum variable load (gravity opposes environmental loads) positioned at the most onerous center of gravity location applicable to extreme conditions as specified in Section 3.2.

E = The extreme member load vector due to the assessment return period wind, wave and current conditions (including associated large displacement effects).

D_n = Member load vector due to the inertial loadset which represents the contribution of dynamics over and above the quasi-static response as described in Section 7.3.6 (including associated large displacement effects).

8.1.4 *Assessment of Members*
- excluding stiffened and high R/t ratio tubulars

8.1.4.1 *General interaction equations*

Each structural member within the scope of Section 8.1.2 shall satisfy the following conditions:

If $P_u/\phi_a P_n > 0.2$

$$\frac{P_u}{\phi_a P_n} + \frac{8}{9}\left[\left\{\frac{M_{uex}}{\phi_b M_{nx}}\right\}^{\eta} + \left\{\frac{M_{uey}}{\phi_b M_{ny}}\right\}^{\eta}\right]^{\frac{1}{\eta}} \le 1.0 \qquad \text{[Eq. H1-1a]}$$

else

$$\frac{P_u}{2\phi_a P_n} + \left[\left\{\frac{M_{uex}}{\phi_b M_{nx}}\right\}^{\eta} + \left\{\frac{M_{uey}}{\phi_b M_{ny}}\right\}^{\eta}\right]^{\frac{1}{\eta}} \le 1.0 \qquad \text{[Eq. H1-1b]}$$

where;

P_u = applied axial load

P_n = nominal axial strength determined in accordance with Section 8.1.4.2 (tension) and 8.1.4.3 (compression).

M_{uex}, M_{uey} = effective applied bending moment determined in accordance with Section 8.1.4.4 (tension) and 8.1.4.5 (compression).

M_{nx}, M_{ny} = nominal bending strength determined in accordance with Section 8.1.4.6.

ϕ_a = Resistance factor for axial load = 0.85 for [Eq. E2.1] compression and 0.90 for tension [Eq. D1.1].

ϕ_b = Resistance factor for bending = 0.9 [Ch. F1.2]

η = Exponent for biaxial bending, a constant dependent on the member cross section geometry, determined as follows:

i) For purely tubular members, η = 2.0

ii) For doubly symmetric open section members, η = 1.0

iii) For all other geometries, the value of η may be determined by analysis as described in Section 8.1.4.7 but shall not be less than 1.0. In lieu of analysis, a value of η equal to 1.0 may be used.

The interaction equations can be used in a reduced form if one or two of the three load ratio terms in the equation are zero.

Alternatively, the more complex interaction formulations given in Section C8.1.4.7 of the Commentary may be used where applicable.

8.1.4.2 *Nominal Axial Strength of a Structural Member in tension P_n*

For a member comprising more than one component, the nominal tensile strength lies between the maximum individual tensile strength of any one component, and the sum of all the individual tensile strengths.

The nominal tensile strength of a tension component shall be the lower value from the following equations:

a) $P_{ni} = F_{yi}A_i$

b) $P_{ni} = \frac{5}{6} F_{ui}A_i$

where;

- A_i = area of component
- F_{yi} = specified minimum yield stress of component (or specified yield strength where no yield point exists)
- F_{ui} = specified minimum tensile (ultimate) strength of component
- P_{ni} = component nominal axial tensile strength

This assumes that for members in jack-up units the net section is equal to the gross section [Eq's. D1.1 and D1.2].

The total member nominal tensile strength shall be:

$$P_n = F_{min}\Sigma A_i$$

with the resistance factor

$$\phi_t = 0.90 \qquad \text{[Eq. D1.1]}$$

where F_{min} is the smallest value of F_{yi} or $\frac{5}{6}F_{ui}$ of all the components.

Note: If for any component the nominal strength is significantly different from the nominal strengths of other components, the formulation above may be conservative and alternative rational methods may be applied. An example is given in the Commentary.

8.1.4.3 *Nominal Axial Strength of a Structural Member in Compression P_n*

So long as local buckling of the components of a member is not the limiting state, the member can be treated for global loads only. Should local buckling dominate, the loads in the components must be considered. Therefore, in determining the nominal axial strength of a member in compression, a local buckling check must first be applied.

<u>Check: Local buckling</u>

The structural components which make up the cross section of a compact or noncompact section must satisfy the following criteria [Table B5.1]:

i) For rectangular components stiffened along both edges

$$b_i/t_i \le 625/\sqrt{(F_{yi} - F_r)}$$

$$\{\text{Imperial: } b_i/t_i \le 238/\sqrt{(F_{yi} - F_r)}\ \}$$

ii) For rectangular components stiffened along one edge

$b_i/t_i \le 250/\sqrt{(F_{yi})}$ {Imperial: $b_i/t_i \le 95/\sqrt{(F_{yi})}$ }

iii) For tubular sections

$R/t_i \le 11380/F_{yi}$ {Imperial: $R/t_i \le 1650/F_{yi}$}

where;

- b_i = width of a rectangular component
- t_i = thickness of a rectangular component or tube wall
- R = outside radius of the tube or tubular component
- F_r = residual stress due to welding (114 MPa, {16.5 ksi})

Members containing rectangular and tubular sections which do not meet this criteria are considered to be slender and are treated in 8.1.4.3 b) for local buckling.

a) <u>Strength assessment for Compact and Noncompact Sections</u>

The nominal axial strength of a structural member subject to axial compression and within the above stipulated restrictions regarding cross section shall be determined from the following equations:

$$P_n = A\,F_{cr} \qquad \text{[Eq. E2.1]}$$

$$F_{cr} = (0.658^{\lambda_c^2})\,F_{yeff} \qquad \text{For } \lambda_c \le 1.5 \qquad \text{[Eq. E2.2]}$$

$$F_{cr} = \left\{\frac{0.877}{\lambda_c^2}\right\} F_{yeff} \qquad \text{For } \lambda_c > 1.5 \qquad \text{[Eq. E2.3]}$$

where;

- A = gross area of section (excluding rack teeth of chords)

$$\lambda_c = \frac{K\iota}{r\pi}\left\{\frac{F_{yeff}}{E}\right\}^{1/2} \quad \text{for max. } K\iota/r \text{ from all directions} \qquad \text{[Eq. E2.4]}$$

- ι = unbraced length of member:
  - face to face for braces
  - braced point to braced point for chords
  - longer segment length of X-braces (one pair must be in tension, if not braced out of plane)
- r = radius of gyration, based on gross area of section.
- E = material Young's modulus (200,000 MN/m^2 {29,000 ksi}).
- F_{yeff} =effective material yield stress, to be taken as the minimum of (specified) yield stress or 5/6 (ultimate stress) of all components in the member unless rational analysis shows that a higher value may be used.
- K = effective length factor. Figure 8.3 provides generally recommended values for K. For the specific case of jack-up truss legs, the value of K shall be taken as follows [Table C-C2.1], unless alternative values are shown applicable by rational analysis:

	K	Assumed boundary conditions
Chord members	1.0	pinned-pinned
K-Braces & span breakers	0.8	between pinned-pinned
X-Braces	0.9	and fully built-in
Complete legs	2.0	pinned-sliding

b) <u>Strength Assessment for Members with Slender Components</u>

The nominal axial strength of a structural member subject to axial compression and outside the restrictions for a) above shall be determined from the following equations.

$$P_n = A\,F_{cr}$$

where;

$$F_{cr} = Q(0.658^{Q\lambda_c^2})F_{yeff} \quad \text{for } \lambda_c\sqrt{Q} \le 1.5 \qquad \text{[Eq. A-B5-11]}$$

$$F_{cr} = \left\{\frac{0.877}{\lambda_c^2}\right\} F_{yeff} \quad \text{for } \lambda_c\sqrt{Q} > 1.5 \qquad \text{[Eq. A-B5-13]}$$

where λ_c is defined in Section 8.1.4.3 a) and Q is determined from the following:

i) For members comprising entirely of stiffened components [A-B5.3.b and A-B5.3.c]:

$$Q = Q_a$$

where;

$$Q_a = A_e/A \qquad \text{[Eq. A-B5-10]}$$

and

A_e is the section effective area found from:

$$A_e = \Sigma\, b_{ei}\, t_i \text{ (excluding rack teeth of chords)}$$

with

$$b_{ei} = \frac{856 t_i}{\sqrt{f_i}}\left\{1 - \frac{170}{(b_i/t_i)\sqrt{f_i}}\right\} \le b_i$$

$$\left\{\text{Imperial: } b_{ei} = \frac{326 t_i}{\sqrt{f_i}}\left\{1 - \frac{64.9}{(b_i/t_i)\sqrt{f_i}}\right\} \le b_i \right\}$$

[Eq A-B5-7]

and f_i is the calculated <u>elastic</u> stress in the component where, for the analysis, the member area is based on the actual cross sectional area but with elastic section modulus and radius of gyration based on effective area.

ii) For members comprising of stiffened and unstiffened components [A-B5.3.b and A-B5.3.c]:

$$Q = Q_a\, Q_s$$

where Q_a is determined from Section 8.1.4.3 b) i) but with the additional check that f_i for the stiffened component must be such that the maximum compressive stress in the unstiffened component does not exceed $\phi_c F_{cr}$ with F_{cr} defined in Section 8.1.4.3 b) with $Q = Q_s$ and $\phi_c = 0.85$ or $\phi_b F_{yeff} Q_s$ with $\phi_b = 0.90$.

Q_s is the lowest value for all components in the member which are stiffened along one edge determined from the following:

For $250/\sqrt{F_y} < b_i/t_i < 460/\sqrt{F_y}$
{Imperial: $95/\sqrt{F_y} < b_i/t_i < 176/\sqrt{F_y}$ }

$Q_s = 1.415 - 0.00166(b_i/t_i)\sqrt{F_y}$
{Imperial: $1.415 - 0.00437(b_i/t_i)\sqrt{F_y}$ } [Eq. A-B5-3]

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For $b_i/t_i \geq 460/\sqrt{F_{yi}}$ {Imperial: $b_i/t_i \geq 176/\sqrt{F_{yi}}$ }

$Q_s = 137{,}900/[F_{yi}(b_i/t_i)^2]$
{Imperial: $Q_s = 20{,}000/[F_{yi}(b_i/t_i)^2]$} [Eq. A-B5-4]

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<u>Note</u>: The implication of this section is that the critical components in the member will be the unstiffened components. If these buckle, then the assumed buckling lengths and hence strengths for the stiffened components will then be wrong, hence invalidating the original assumptions. This assumes that the unstiffened components are placed in the member to reduce the buckling length of the major components.

iii) For members comprising a tube alone and:

$11{,}375/F_y < R/t < 44{,}815/F_y$
{Imperial: $1{,}650/F_y < R/t < 6{,}500/F_y$}

$$Q = \frac{3790}{F_y(R/t)} + \frac{2}{3} \qquad \text{\{Imperial: } Q = \frac{550}{F_y(R/t)} + \frac{2}{3}\text{\}}$$
[Eq. A-B5-9]

8.1.4.4 *Effective Applied Moment for Members in Tension; M_{ue} (M_{uex},M_{uey})*

In many cases, the effective applied moments used in the interaction equations will not be equal to applied moments obtained in a structural analysis. This can be due to the type of structural model and /or the effective length effect on buckling. The following procedures shall be followed for the determination of the effective applied moment.

The effective applied moment for a member under axial tension shall be taken to be equal to the applied moment from an analysis including global P-Δ effects and accounting for local loading.

8.1.4.5 *Effective Applied Moment for Compression Members; M_{ue} (M_{uex},M_{uey})*

The effective applied moment for a member under axial compression shall be taken to be:

$M_{ue} = B\ M_u$ [Eq. H1.2]

where;

M_u is the applied moment determined in an analysis which includes global P-Δ/hull-sway effects and accounts for local loading. When eccentricity is not incorporated in the model, the equation for M_{ue} should be modified to include $p_u e$ due to the eccentricity, e, between the elastic and plastic neutral axes. Note: When the member considered represents the leg the requirement to include P-Δ effects in the global analysis means that the provisions of ii) below apply.

and

i) Where the individual member loads are determined from a first order linear elastic analysis i.e. the equilibrium conditions were formulated on the undeformed structure, (For example a linear analysis of a detailed truss type leg, using external loads determined from a second order analysis of a simplified global model):

$$B = \frac{C_m}{(1 - P_u / P_E)} \geq 1.0 \qquad \text{[Eq. H1-3]}$$

where:

P_E = $(\pi^2 r^2 AE)/(K\iota)^2$ with $K \leq 1.0$ and P_E is to be calculated for the plane of bending. A is defined in Section 8.1.4.3 a) and r is the radius of gyration for the plane of loading.

C_m = a coefficient whose value shall be taken as follows [Ch. H1.2a]:

i) For members not subject to transverse loading between their supports in the plane of bending $C_m = 0.6 - 0.4\ (M_1/M_2)$ [Eq. H1-4] where M_1/M_2 is the ratio of the smaller to the larger moments at the ends of that portion of the member unbraced in the plane of bending under consideration. M_1/M_2 is positive when the member is bent in reverse curvature, negative when bent in single curvature.

ii) For members subjected to transverse loading between their supports, the value of C_m can be determined from rational analysis. In lieu of such analysis, the following values may be used:
For members whose ends are restrained against
sidesway $C_m = 0.85$
For members whose ends are unrestrained against
sidesway $C_m = 1.0$

ii) Where the individual member loads are determined from a second order analysis i.e. the equilibrium conditions were formulated on the elastically deformed structure so that local P-Δ loads were also included in the analysis:

$B = 1.0$

8.1.4.6 *Nominal Bending Strength; M_n (M_{nx}, M_{ny})*

The calculation of nominal bending strength is based on the plastic properties of the section. The practice allows for hybrid sections built up from components of different yield strengths. Standard techniques shall be applied to obtain a section plastic moment in the absence of axial load, M_p, based on the individual component values which are the lesser values of F_{yi} and 5/6 F_{ui} (an example is given in the Commentary).

Lateral torsional buckling and local buckling of components must be considered.

If both tensile and compressive yielding occur during the same load cycle, it shall be demonstrated that the structure will shake down without fracture.

Check: Lateral torsional buckling (Not applicable to tubulars)
The cross sectional geometry of a member subjected to bending shall be examined for susceptibility to the limit state of lateral torsional bucking. The member cross section must satisfy the following criteria for compact sections for the nominal bending strength to be assessed under Sections 8.1.4.6 a) or 8.1.4.6 b).

$L_b/r_y \leq 25860\sqrt{(JA)/M_p}$ {Imperial: $L_b/r_y \leq 3750\sqrt{(JA)/M_p}$ }

[Table A-F1.1]

where;

- L_b = Laterally unbraced length; length between points which are either braced against lateral displacement of the compression flange or braced against twist of the cross section.
- r_y = Radius of gyration about the minor axis.
- A = Cross sectional area.
- J = Torsional constant for the section

Sections which do not satisfy this criteria are susceptible to lateral torsional buckling and are treated as having slender compression components as in Section 8.1.4.6 c).

Check: Local buckling
The cross sectional geometry of a member subjected to bending is to be examined for susceptibility to the limit state of local bucking. If local buckling is deemed to be the limit state, the nominal bending strength shall be reduced in accordance with the following paragraphs. Members with particularly slender components are covered in Section 8.1.4.6c).

For this check it is necessary to identify web components and flange components. This can be done by visual inspection, with knowledge of the major and minor axes. For example, in a split-tubular, opposed rack chord, the rack plate would be a suitable web component, and the split tubulars flanges. For a teardrop chord, the rack and side plates would be web components, and the back plate the flange. In cases of doubt, components shall be checked as both web and flange.

a) <u>Compact Sections</u>

For members in which all the components sections satisfy the following [Table B5.1]:

i) For rectangular components stiffened along both edges

$b_i/t_i \quad \leq \quad \lambda_p$

where;

$\lambda_p \quad = \quad 500/\sqrt{(F_{yi})}$ {Imperial: $\lambda_p = 190/\sqrt{(F_{yi})}$ }

ii) For rectangular components stiffened along one edge

$b_i/t_i \quad \leq \quad \lambda_p$

where;

$\lambda_p \quad = \quad 170/\sqrt{(F_{yi})}$ {Imperial: $\lambda_p = 65/\sqrt{(F_{yi})}$ }

iii) For tubular sections

$2R/t \quad \leq \quad \lambda_p$

where;

$\lambda_p \quad = \quad 14270/F_{yi}$ {Imperial: $\lambda_p = 2070/F_{yi}$}

The nominal bending strength is given by the plastic bending moment of the whole section

$M_n \quad = \quad M_p$ [Eq-A-F1-1]

where M_p is derived as discussed above.

<u>Note:</u> Where significant plastic hinge rotations are required the section must remain stable after rotation through an appreciable angle. In such cases, to achieve this requirement, the limitations of ii) and iii) above should be reduced to:

ii) $\lambda_p \quad = \quad 135/\sqrt{(F_{yi})}$ {Imperial: $\lambda_p = 52/\sqrt{(F_{yi})}$ }

iii) $\lambda_p \quad = \quad 11000/F_{yi}$ {Imperial: $\lambda_p = 1600/F_{yi}$}

b) <u>Noncompact Sections</u>

For members in which all the components do not satisfy the previous criteria but satisfy the following [Table B5.1]:

i) For rectangular components stiffened along both edges

$b_i/t_i \quad \leq \quad \lambda_r$

where;

$\lambda_r \quad = \quad 625/\sqrt{(F_{yi} - F_r)}$ {Imperial: $\lambda_r = 238/\sqrt{(F_{yi} - F_r)}$ }

$F_r \quad = \quad 114$ MN/m^2 {16.5 ksi} residual stress

ii) For rectangular components stiffened along one edge

$b_i/t_i \quad \leq \quad \lambda_r$

where;

$\lambda_r \quad = \quad 278/\sqrt{(F_{ywj} - F_r)}$ {Imperial: $\lambda_r = 106/\sqrt{(F_{ywj} - F_r)}$ }

$F_{ywj} \quad =$ web component yield stress.

$F_r \quad = \quad 114$ MN/m^2 {16.5 ksi} residual stress.

iii) For tubular sections

$2R/t \quad \leq \quad \lambda_r$

where;

$\lambda_r \quad = \quad 61850/F_{yi}$ {Imperial: $\lambda_r = 8970/F_{yi}$}

The nominal bending strength is given by an interpolation between the plastic bending moment and the limiting buckling moment:

$$M_n = M_p - (M_p - M_r)\left\{\frac{\lambda - \lambda_p}{\lambda_r - \lambda_p}\right\}_h \qquad \text{[Eq. A-F1.3]}$$

where;

M_p = Section Plastic Moment.

$_h$ = subscript referring to the component which produces the smallest value of M_n.

λ = b/t or 2R/t as applicable for component h.

λ_p is determined for component h from 8.1.4.6 a).

λ_r is determined for component h from 8.1.4.6 b).

M_r is the limiting buckling moment of the section defined as follows:

For bending of non-tubular sections about the major axis, the lesser of

$M_r = F_l S$ (flange buckling) [Table A-F1.1]

$M_r = R_e F_{yfj} S$ (web buckling) [Table A-F1.1]

where;

F_l = the smaller of $(F_{yfj} - F_r)$ and F_{ywj}

S = minimum section elastic modulus for plane of bending under consideration.

For bending of non-tubular sections about the minor axis;

$M_r = F_{yfj} S$ (flange buckling) [Table A-F1.1]

For bending of tubular sections: [Table A-F1.1]

$$M_r = \left\{\frac{2068}{R/t} + F_y\right\}S \qquad \left\{\text{Imperial: } M_r = \left\{\frac{300}{R/t} + F_y\right\}S\right\}$$

F_{yfj} = yield stress of flange component.

R_e = hybrid girder reduction factor [A-G2]

= 1.0 if components are of the same material

otherwise:

= $[12 + a_r(3m - m^3)] / (12 + 2a_r) \le 1.0$

a_r = ratio of total web area to area of compression flange.

m = ratio of web component yield stress to flange component yield stress which gives smallest value of R_e.

c) Slender Sections

The nominal bending strength of members including components which do not satisfy the above criteria for compact and noncompact sections or for lateral torsional buckling shall be determined in accordance this section.

The nominal bending strength of a member is given by the limiting flexural bending moment:

$$M_n = S\,F_{cr}$$

where S is the elastic section modulus for the plane of bending under consideration and F_{cr} is the lowest value from (where appropriate):

i) Doubly symmetric members (lateral torsional buckling)

$$F_{cr} = 6.895\,\frac{C_b X_1 \sqrt{2}}{\lambda}\left\{1+\frac{X_1^{\,2} X_2}{2\lambda^2}\right\}^{1/2} \qquad \text{[Table A-F1.1(b)]}$$

$$\{\text{Imperial: } F_{cr} = \frac{C_b X_1 \sqrt{2}}{\lambda}\left\{1+\frac{X_1^{\,2} X_2}{2\lambda^2}\right\}^{1/2}\}$$

where;

C_b = $1.75 + 1.05(M_1/M_2) + 0.3(M_1/M_2)^2 \le 2.3$ where M_1 is the smaller and M_2 the larger end moment in the unbraced member; M_1/M_2 is positive when the moments cause reverse curvature.

X_1 = $(\pi/S)\sqrt{(EGJA/2)}$

X_2 = $(4C_w/I_y)(S_x/GJ)^2$

E = Modulus of elasticity (200,000 MN/m^2 {29,000 ksi}).

G = Shear modulus of elasticity (77,200 MN/m^2 {11,200 ksi}).

J = Torsion constant for section.

A = Cross-sectional area (excluding rack teeth).

I_y = Second moment of area of section about minor axis.

S_x = Elastic section modulus for major axis bending.

C_w = Warping constant.

λ = L_b/r_y

r_y = Radius of gyration about the minor axis

ii) Singly symmetric members (lateral torsional buckling) [Table A-F1.1(c)]

$$F_{cr} = \frac{393{,}000 C_b}{S L_b}\,\{B_1 + \sqrt{(1+B_2+B_1^{\,2})}\}\,\sqrt{(I_y J)} \le F_y$$

$$\{\text{Imp'l: } F_{cr} = \frac{57{,}000 C_b}{S L_b}\,\{B_1 + \sqrt{(1+B_2+B_1^{\,2})}\}\,\sqrt{(I_y J)} \le F_y\}$$

where;

$$B_1 = 2.25\left\{2\left(\frac{I_c}{I_y}\right)-1\right\}\left\{\frac{h}{L_b}\right\}\left\{\frac{I_y}{J}\right\}^{1/2}$$

$$B_2 = 25\left\{1-\left(\frac{I_c}{I_y}\right)\right\}\left\{\frac{h}{L_b}\right\}^2\left\{\frac{I_c}{J}\right\}$$

h = web depth.

I_c = second moment of area of compression flange about the section minor axis

C_b = as for doubly symmetric sections.

iii) Doubly and singly symmetric members (flange local buckling)

[Table A-F1.1(g)]

F_{cr} = $77{,}220/\lambda^2$ {Imperial: $F_{cr} = 11{,}200/\lambda^2$}

where;

λ = b_i/t_i for flange(s)

iv) For tubular members (Local buckling) [Table A-F1.1]

F_{cr} = $33{,}610/(R/t)$ {Imperial: $F_{cr} = 4{,}875/(R/t)$}

where;

R = radius of tubular

t = wall thickness of tubular

8.1.4.7 <u>Determination of η for non-tubular sections</u>

The general interaction equation requires that applied bending moments are resolved into components in two perpendicular axes (X,Y). For elaborate sections such as chords, these axes may be selected on the basis of section geometry and not on load incidence. Therefore neither of these axes need be coincident with the angle of load. The use of the exponent η is necessary to ensure that the effective nominal bending strength of the section is not significantly influenced by this choice of axes.

To determine a suitable value of η the following process is applied:

1. For angles q = 0°, 30°, 45°, 60°, 90°, to the X-axis, obtain the allowable bending strengths M_{nq}.
2. Assume loads incident at angle q=30°, the limiting bending moment in the absence of axial load is $M_{uq} = M_{nq}$. When the section is non-compact M_{nq} is a function of M_r and a suitable analysis in line with that of Section 8.1.4.6 b), should be applied in determining M_r for non-principal axes.
3. Resolve this limiting moment into limiting components M'_{uex} and M'_{uey} about the X and Y axes:

   $M'_{uex} = M_{uq} \cos q$

   $M'_{uey} = M_{uq} \sin q$
4. The interaction ratio for the (X,Y) axis pair is required to give the same result as if the X-axis were lined up with the q-axis:

   i.e. $$\left\{\left(\frac{M'_{uex}}{M_{nx}}\right)^{\eta} + \left(\frac{M'_{uey}}{M_{ny}}\right)^{\eta}\right\}^{\frac{1}{\eta}} = \frac{M_{uq}}{M_{nq}}$$

   Since M_{nx} and M_{ny} are by definition M_{nq} for q = 0° and 90° respectively and so are known, the only unknown in the above identity is η. This can be determined from the graph in Figure 8.4 or by numerical means if preferred. Figure 8.4 is based on the ratio M_{uq}/M_{nq} being equal to unity, and will produce conservative results when axial loads are present.
5. Step 4 yields a value of η suitable for loads from 30° to the X-axis. Steps 2 to 4 are repeated for q = 45° and 60° to obtain a range of values of η.
6. The value of η for use in subsequent assessment shall be the least of the above determined values, but not less than 1.0.

This method includes some approximation. Since bending will not be along or perpendicular to a plane of symmetry, deflection will not necessarily be at the same angle as the applied moment. This effect is second order.

Note: An alternative, more detailed approach, involving modified interaction equations is
presented below for a number of typical chord configurations.

8.1.4.8 *Plastic Interaction Curve Approach*

Alternatively, interaction equations and curves for generic families of chords are presented in Figures C8.1.8 - C8.1.11 in the Commentary. These are taken from Dyer [19] and based on the interaction approach proposed by e.g. Duan & Chen [20]. It should be noted that the curves and equations are based on axial load applied at the 'center of squash' which is defined as the location at which the axial load produces no moment on the yielded section. For chords without two axes of symmetry (triangular and tubular with offset rack) this is offset from the elastic centroid when the section is comprised of materials of differing yield strengths. Before a section is checked it is necessary to correct as appropriate moments by the axial load times the offset distance between the elastic centroid (used in the structural analysis) and the 'center of squash'. This offset, together with other geometric data for the members of each family of chord is presented in Tables C8.1.1 to C8.1.4 in the Commentary. The effective applied moment may then be calculated from:

$$M_{uex} = B_x(M_{ux} + P_u.e_y)$$
$$M_{uey} = B_y(M_{uy} + P_u.e_x)$$

The interaction equations are based on ultimate capacity. It is therefore necessary to introduce the required resistance factors. This is achieved by defining:

$$P_y = F_1.\phi_a.P_n$$
$$M_{px} = F_2.\phi_b.M_{nx}$$
$$M_{py} = F_2.\phi_b.M_{ny}$$

where; F_1 = 1.0, unless alternative values are justified by analysis.
F_2 = 1.0, unless alternative values are justified by analysis.

The ratio of P_u/P_y, M_{uex}/M_{px} and M_{uey}/M_{py} shall be determined for the condition under consideration. The user should then enter the plastic interaction curves with the M_{uex}/M_{px} and M_{uey}/M_{py} ratios. The allowable value for P_u/P_y may then be determined. A measure of the interaction ratio can then be obtained as the ratio between the actual and allowable values of P_u/P_y.

The user should note that the equations for sections with only one axis of symmetry depend on the sign of the moment about the Y-Y axis (given in the Figures). The sign convention should be observed with care.

The equations are based on lower bound data from each family of chord shape and will therefore tend to be conservative. More accurate results will be obtained from the individual consideration of the chord in question.

[NOTE: At present Figures C8.1.8 - C8.1.11 in the Commentary cover only fully plastic section strength considerations, and their use for a beam-column member is based on the assumption that the member being evaluated is sufficiently short/compact that elasto-plastic stability (buckling at large strains) is not a consideration. Violating this assumption may lead to errors on the unsafe side. Updated information covering elasto-plastic stability may be generated in the future, and should preferentially be used for member evaluations.]

8.1.5 *Assessment of other member geometries*

It is recommended that other member geometries are assessed using the relevant provisions of AISC LRFD [14] or, for stiffened or high R/t ratio shell members, the DNV Rules for fixed offshore installations in conjunction with the DNV Classification note on Buckling Strength Analysis of Mobile Offshore Units [15].

For these geometries, the nominal strength/resistance factors shall be the same as given in the relevant codes, but the load cases and factored loads should be determined in accordance with Section 8.1.3 rather than using the factors in the reference.

8.1.6 *Assessment of member joints*

It is recommended that the assessment of joints of members which form a truss structure be carried out in accordance with AISC LRFD [14] or API LRFD [16] as appropriate for the joint under consideration. The factored loads should be determined in accordance with Section 8.1.3, rather than using the factors in the references.

[NOTE: At present Figures C8.1.8 - C8.1.11 in the Commentary cover only fully plastic section strength considerations, and their use for a beam-column member is based on the assumption that the member being evaluated is sufficiently short/compact that elasto-plastic stability (buckling at large strains) is not a consideration. Violating this assumption may lead to errors on the unsafe side. Updated information covering elasto-plastic stability may be generated in the future, and should preferentially be used for member evaluations.]

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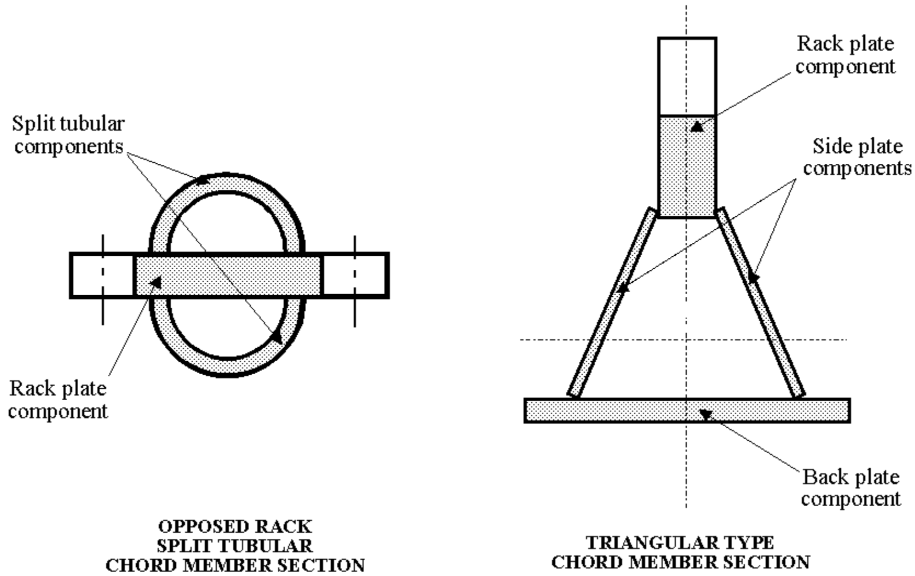


Figure 8.2: Typical members and components

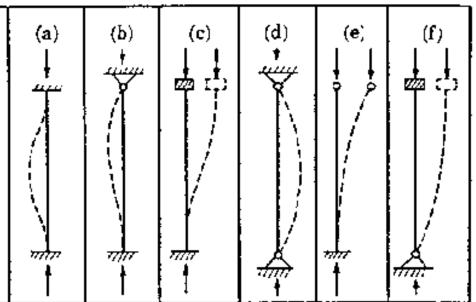
	(a)	(b)	(c)	(d)	(e)	(f)
Buckled shape of column is shown by dashed line						
Theoretical K value	0.5	0.7	1.0	1.0	2.0	2.0
Recommended design value when ideal conditions are approximated	0.65	0.80	1.2	1.0	2.10	2.0
End condition code	Rotation fixed and translation fixed Rotation free and translation fixed Rotation fixed and translation free Rotation free and translation free					

Figure 8.3: Effective Length Factors (from AISC-LRFD [14])

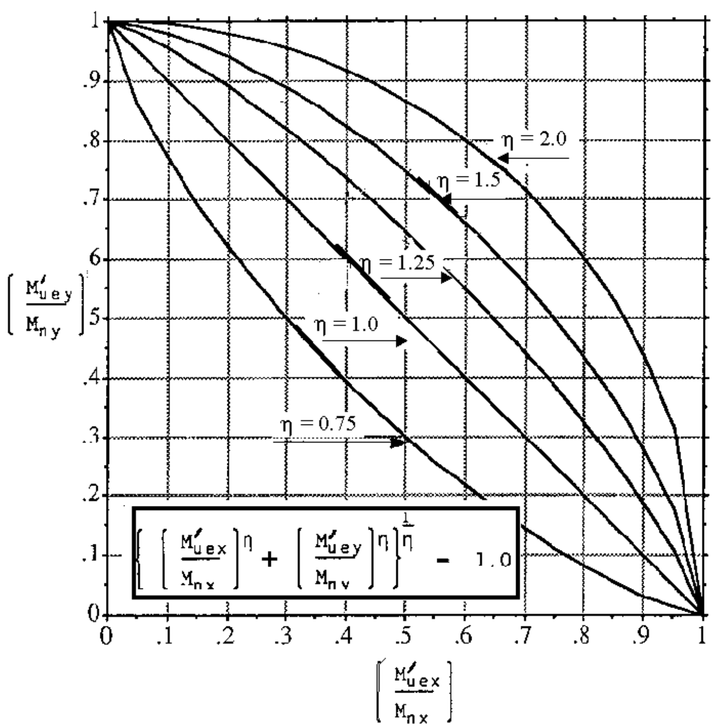


Figure 8.4: Chart for Determination of η

8.2 Overturning Stability

8.2.1 For independent leg jack-ups the assumed overturning axis shall be the most critical axis passing through any two spudcan reaction points as defined in Section 5.2.

8.2.2 The overturning moment shall be calculated from the components of environmental loading, resolved normal to the overturning axis, times the vertical distance from the point of action of the component to the overturning axis.

The overturning stability should be checked for the overturning moment M_O caused by the following factored loads:

$$M_O = \gamma_3(M_E + \gamma_4.M_{Dn})$$

where;

γ_3 = 1.15 (**provisional - see Section 1.8**)

γ_4 = 1.0

M_E = The extreme overturning moment due to the assessment return period wind, wave and current conditions (see note).

M_{Dn} = The dynamic overturning moment due to the inertial loadset which represents the contribution of dynamics over and above the quasi-static response as described in Section 7.3.6 (see note).

8.2.3 The unit shall be shown to satisfy the following overturning requirements:

$$M_O \leq \phi_1.M_D + \phi_2.M_L + \phi_3.M_S$$

where;

M_D = The stabilizing moments due to weight of structure and non-varying loads (at the displaced position resulting from the factored loads - see note) including:

- Weight in air including appropriate solid ballast.
- Equipment.
- Buoyancy.
- Permanent enclosed liquid.

M_L = The stabilizing moment due to the most onerous combination of minimum variable load and center of gravity applicable to extreme conditions as specified in Section 3.2 (at displaced position - see note).

M_S = The stabilizing moments due to seabed foundation fixity (these shall not be taken into account unless specific calculations for the location and the spudcan concerned show that a significant contribution from seabed fixity may be expected).

ϕ_1 = R.F. for dead load moments (M_D) = 0.95

ϕ_2 = R.F. for live load moments (M_L) = 0.95

ϕ_3 = R.F. for seabed moments (M_S) = 0.95

Note: It may be convenient to consider the reduction in dead and live load stabilizing moment caused by the displacement resulting from the factored loads as an increase in the overturning moment, rather than as a reduction in the stabilizing moment.

8.3 Foundation assessment

The foundation assessment shall be carried out in a step-wise manner until the requirements of the current stage are satisfied when it is not necessary to proceed further. The philosophy is described in Section 6.3 and shown in Figure 6.9.

8.3.1 *Step 1 - Preload and Sliding checks*

Step 1a - Preload check

8.3.1.1 A preload check shall be used to verify the adequacy of the leeward leg foundation. The acceptance criteria for the windward leg are discussed in Section 8.3.1.5.

8.3.1.2 The preloading capability should be checked for the vertical leg reaction Q_v caused by the following factored loads:

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

where;

γ_1 = 1.0
γ_2 = 1.0
γ_3 = 1.15 **(provisional - see Section 1.8)**
γ_4 = 1.0
V_D = Vertical leg reaction due to the weight of the structure and non-varying loads including:
- Weight in air including appropriate solid ballast.
- Equipment or other objects.
- Buoyancy.
- Permanent enclosed liquid.

V_L = Vertical leg reaction due to maximum variable load positioned at the most onerous center of gravity location applicable to extreme conditions as specified in Section 3.2.
V_E = Extreme vertical leg reaction due to the assessment return period wind, wave and current conditions (including associated large displacement effects).
V_{Dn} = Vertical leg reaction due to the inertial loadset which represents the contribution of dynamics over and above the quasi-static response as described in Section 7.3.6 (including associated large displacement effects).

8.3.1.3 The preload capacity shall be shown to be sufficient to satisfy the following requirements:

$$Q_v \leq \phi_p.V_{Lo}$$

where;

V_{Lo} = Vertical leg reaction during preloading
ϕ_p = R.F. for foundation capacity during preload
= 0.9 (see Commentary)

8.3.1.4 In dense sands (i.e. with maximum bearing area not mobilized) and in clayey soils the preload check may be applied if the leeward leg horizontal reaction $Q_H < 0.1V_{Lo}$ (with Q_H determined in accordance with the equations of Section 8.3.1.5). For a spudcan fully embedded in sand the preload check may be applied if the leeward leg horizontal reaction $Q_H < 0.03V_{Lo}$. In all other cases a pinned condition bearing capacity check of the foundation shall be carried out in accordance with Section 8.3.2 (see Commentary).

8.3.1.5 *Step 1b - Sliding Resistance - Windward Leg(s)*

a) The sliding capacity of the windward leg(s) should be checked for the horizontal leg reaction Q_H caused by the following factored loads:

$$Q_H = \gamma_3(H_E + \gamma_4.H_{Dn})$$

in association with:

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

where;

γ_1 = 1.0
γ_2 = 1.0
γ_3 = 1.15 **(provisional - see Section 1.8)**
γ_4 = 1.0

H_E, V_E = The extreme horizontal and vertical leg reactions due to the assessment return period wind, wave and current conditions (including associated large displacement effects).

H_{Dn}, V_{Dn} = The horizontal and vertical leg reactions due to the inertial loadset which represents the contribution of dynamics over and above the quasi-static response as described in Section 7.3.6 (including associated large displacement effects).

V_D = Vertical leg reaction due to the weight of the structure and non-varying loads including:

- Weight in air including appropriate solid ballast.
- Equipment or other objects.
- Buoyancy.
- Permanent enclosed liquid.

V_L = Vertical leg reaction due to the minimum variable load positioned at the most onerous center of gravity location applicable to extreme conditions as specified in Section 3.2.

b) The foundation shall be shown to satisfy the following capacity requirements:

$$Q_H \leq \phi_{Hfc}.F_H$$

where;

F_H = foundation capacity to withstand horizontal loads when load Q_V is acting

ϕ_{Hfc} = R.F. for horizontal foundation capacity (see Commentary).
= 0.80 (effective stress - sand/drained).
= 0.64 (total stress - clay/undrained).

8.3.2 *Step 2a - Capacity check - pinned foundation*

8.3.2.1 The bearing capacity of the leeward leg should be checked for the leg reaction vector Q_{VH}, relative to the still water leg reaction vector, caused by the following factored loads:

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

where;

γ_1 = 1.0
γ_2 = 1.0
γ_3 = 1.15 **(provisional - see Section 1.8)**
γ_4 = 1.0

VH_D = Vector of vertical and horizontal leg reaction due to the weight of structure and non-varying loads (allowing for structural deformation) including:

- Weight in air including appropriate solid ballast.
- Equipment or other objects.
- Buoyancy.
- Permanent enclosed liquid.

VH_L = Vector of vertical and horizontal leg reaction (allowing for structural deformation) due to maximum variable load positioned at the most onerous center of gravity location applicable to extreme conditions as specified in Section 3.2 (including associated large displacement effects).

VH_E = Vector of extreme vertical and horizontal leg reaction due to the assessment return period wind, wave and current conditions (including associated large displacement effects).

VH_{Dn} = Vector of vertical and horizontal leg reaction due to the inertial load set which represents the contribution of dynamics over and above the quasi-static response as described in Section 7.3.6 (including associated large displacement effects).

8.3.2.2 The leeward leg foundation shall be shown to satisfy the following capacity requirements:

$$Q_{VH} \leq \phi_{VH}.F_{VH}$$

where;

F_{VH} = foundation capacity to withstand combined vertical and horizontal loads taken as a vector from the still water load vector in the same direction as Q_{VH}.

ϕ_{VH} = R.F. for foundation capacity (see Commentary).
= 0.90 - Maximum bearing area not mobilized.
= 0.85 - Penetration sufficient to mobilize maximum bearing area.

8.3.2.3 The windward leg foundations should be checked according to the requirements of Section 8.3.1.5.

8.3.3 *Step 2b - Capacity check - with foundation fixity*

8.3.3.1 The foundation capacity of the leeward and windward legs should be checked for the leg reaction vector, including the associated can moment, Q_{VHM}, relative to the still water leg reaction vector, caused by the following factored loads:

$$Q_{VHM} = \gamma_1.V_HM_D + \gamma_2.VHM_L + \gamma_3(VHM_E + \gamma_4.VHM_{Dn})$$

where;

γ_1 = 1.0
γ_2 = 1.0
γ_3 = 1.15 (**provisional - see Section 1.8**)
γ_4 = 1.0

VHM_D = Vector of vertical and horizontal leg reaction and spudcan moment due to the weight of the structure including non-varying loads (allowing for structural deformation and large displacement effects) including:

- Weight in air including appropriate solid ballast.
- Equipment or other objects.
- Buoyancy.
- Permanent enclosed liquid.

VHM_L = Vector of vertical and horizontal leg reaction and spudcan moment (allowing for structural deformation and large displacement effects) due to maximum (leeward leg) or minimum (windward leg) variable load positioned at the most onerous center of gravity location applicable to extreme conditions as specified in Section 3.2.

VHM_E = Vector of extreme vertical and horizontal leg reaction and spudcan moment due to the assessment return period wind, wave and current conditions (including associated large displacement effects).

VHM_{Dn} = Vector of vertical and horizontal leg reaction and spudcan moment due to the inertial loadset which represents the contributions of dynamics over and above the quasi-static response as described in Section 7.3.6 (including associated large displacement effects).

8.3.3.2 The leg reaction vector Q_{VHM} shall be checked to satisfy the yield surface as defined in 6.3.4.

8.3.3.3 The windward and leeward leg foundations shall also be shown to satisfy the bearing capacity and sliding capacity requirements of 8.3.2.

8.3.4 *Step 3 - Displacement check*

If the factored loads on any footing exceed the factored capacity discussed above a further assessment may be performed in order to show that any additional settlements and/or the associated additional structural loads are within acceptable limits. See Section 6.3.5.

8.3.5 *Punch-through*

The selection of factors of safety against punch-through should be made using sound engineering judgment, accounting for the accuracy of the available soil data and the magnitude of any possible sudden penetration (see Commentary).

When the possibility of punch-through exists during the installation and preloading phases it may be applicable to consider the magnitude of possible sudden penetration in comparison with the structural capability of the unit to resist punch-through.

If the possibility of punch-through remains once the unit has been installed on location and elevated to the operational airgap the evaluation should account for long term effects (e.g. cyclic degradation).

8.4 Horizontal Deflections

When working close to or over a platform the assessor shall, if required by the platform owner, provide the extreme deflections of the jack-up to the platform owner (see Section 5.5.1 of the GUIDELINE).

8.5 Loads in the Holding System

8.5.1 The holding system (elevation and/or fixation system) is deemed to be the system which forms the load path connecting the hull to the legs.

8.5.2 The loads in the holding system shall not exceed those specified by the manufacturers, unless the basis of the limitations and the equivalent reference stress levels are stated, when the factored applied load may be compared with the ultimate capacity multiplied by a R.F. (ϕ) of 0.85.

8.5.3 he stresses in the structural members connecting the holding system to the hull shall be in accordance with the requirements of Section 8.1.

8.6 Hull

8.6.1 It is assumed that the jack-up hull is designed and built to the structural/scantling requirements of a recognized Classification Society and carries a valid Class Certificate.

8.6.2 For jack-ups where 8.6.1 does not apply it shall be shown that the hull has adequate strength to withstand appropriate combinations of dead load, variable load, environmental load, deflections, preload conditions and dynamics effects.

8.7 Structure Condition Assessment

The objective of the site specific assessment is to ensure an appropriate level of structural reliability of the jack-up in the elevated condition. To achieve this, account must be taken of any deterioration in the jack-up structure (see Section 1.3.4 of the GUIDELINE). The condition of the structure is the responsibility of the owner and is deemed to be satisfactory if the jack-up has valid class certification as described in Section 2.4.2. Normally the owner can thus provide the assessor with all the information required to satisfy the structure condition requirement.

In special cases (usually at the option of the operator), an on site structural inspection may be required to assess the condition of the jack-up. Guidance for such an on site structural inspection is given in the Commentary. In the event that the results of this inspection reveal deterioration of the structure, due account of such deterioration shall be taken into account in the assessment.

8 GLOSSARY OF TERMS - ACCEPTANCE CRITERIA

a_r	=	Ratio of total web area to area of compression flange.
A	=	Cross sectional area of a member (excluding rack teeth).
A_e	=	Section effective area (excluding rack teeth).
A_i	=	Area of a component in a member.
b_{ei}	=	Effective width of a component.
b_i	=	Width of a rectangular component.
B, B_1, B_2	=	Factors used in determining M_u for combined bending and compressive axial load.
B_x, B_y	=	Moment amplification factors.
C_b	=	Bending coefficient dependent on moment gradient.
C_m	=	Coefficient applied to bending term in interaction formula for prismatic members dependent upon column curvature caused by applied moments.
C_w	=	Warping constant.
D	=	Dead load vector due to the self-weight of the structure and non-varying loads.
D_n	=	The load vector due to the inertial loadset which represents the contribution of dynamics over and above the quasi-static response (including associated large displacement effects).
e , e_x, e_y	=	Eccentricity between elastic and plastic neutral axes.
E	=	Load due the to assessment return period wind, wave and current conditions (including associated large displacement effects).
E	=	Modulus of elasticity (200,000 MN/m^2 {29,000 ksi}).
f_i	=	Component compressive stress.
F_{cr}	=	Critical stress.
F_H	=	Foundation capacity to withstand horizontal loads when Q_V is acting.
F_{min}	=	The smaller value of F_{yi} and $(5/6)F_{ui}$ of all the components (in a member).
F_r	=	Residual stress due to welding (114 MN/m^2).
F_{VH}	=	Foundation capacity to withstand combined vertical and horizontal loads.
F_{VHM}	=	Foundation capacity to withstand combined vertical, horizontal and moment loads.
F_y	=	Minimum specified yield stress or specified yield strength.
F_{yh}	=	Minimum yield stress or specified yield strength of component with highest b/t ratio.
F_{yeff}	=	Effective material yield stress for consideration of axial buckling.
F_{yi}	=	Minimum specified component yield stress or specified yield strength.
F_{ywj}	=	Minimum specified web yield stress or specified yield strength.
F_{yfj}	=	Minimum specified flange yield stress or specified yield strength.
F_{ui}	=	Component material ultimate strength.
G	=	Shear modulus of elasticity.
$_h$	=	Subscript referring to the component which produces the smallest value of M_n.
h	=	Web depth.
H_{Dn}	=	Horizontal leg reaction due to inertial loadset representing dynamics.
H_E	=	Horizontal leg reaction due wind wave and current.
I_c	=	Second moment of area of compression flange.
I	=	Second moment of area of section.
I_x	=	Second moment of area of section about major axis.
I_y	=	Second moment of area of section about minor axis.
J	=	Torsional constant for the section.

8 GLOSSARY OF TERMS - ACCEPTANCE CRITERIA (continued)

K = Effective length factor.
ι = Unbraced length of member; face to face for braces, braced point to braced point for chords.
L = The load vector due to the maximum or minimum variable load positioned at the most onerous center of gravity location applicable to extreme conditions.
L_b = Laterally unbraced length; length between points which are either braced against lateral displacement of compression flange or braced against twist of the cross section.
m = Ratio of web component yield stress to flange component yield stress which gives smallest value of R_e.
M_D = Stabilizing moment due to self weight.
M_{Dn} = Overturning moment due to inertial loadset representing dynamics.
M_E = Extreme overturning moment due to wind, wave and current.
M_L = Stabilizing moment due to most onerous combination of variable load and center of gravity.
M_S = Stabilizing moment due to seabed foundation fixity.
M_n, M_{nx}, M_{ny} = Nominal bending strength.
M_{nq} = Allowable bending strength about axis q.
M_O = Factored overturning moment.
M_p = Section plastic moment.
M_{px} = Plastic moment capacity about member x-axis.
M_{py} = Plastic moment capacity about member y-axis.
M_r = Limiting buckling moment of section.
M_u = Applied moment determined in an analysis which includes global P-Δ effects and accounts for local loading.
M_{ue}, M_{uex}, M_{uey} = Effective applied bending moment.
M'_{uex}, M'_{uey} = Limiting components of applied bending moment.
M_{uq} = Assumed limiting bending moment about axis q in absence of axial load.
M_1 = Smaller end moment of a member.
M_2 = Larger end moment of a member.
P_E = Euler buckling strength.
P_u = Applied axial load.
P_n = Nominal axial strength.
P_{ni} = Component nominal axial strength.
P_y = Axial yield strength.
q = Angle of load heading with respect to defined X axis.
Q = Factored load vector.
Q = Full reduction factor for slender compression components.
Q_a = Reduction factor for slender stiffened compression components.
Q_H = Factored horizontal leg reaction.
Q_s = Reduction factor for slender unstiffened compression components.
Q_V = Factored vertical leg reaction.
Q_{VH} = Factored leg reaction vector of vertical and horizontal loads.
Q_{VHM} = Factored leg reaction vector of vertical, horizontal and moment loads.
r = Radius of gyration.
r_x = Radius of gyration about the major axis.
r_y = Radius of gyration about the minor axis.
R = Outside radius of the tube or tubular component.
R_e = Hybrid girder reduction factor.

8 GLOSSARY OF TERMS - ACCEPTANCE CRITERIA (continued)

S	=	Elastic section modulus.
S_x	=	Elastic section modulus for major axis bending.
S_y	=	Elastic section modulus for minor axis bending.
t	=	Thickness of tubular member or tubular section.
t_i	=	Thickness of a rectangular or tubular component..
V_D	=	Vertical leg reaction due to self weight.
V_{Dn}	=	Vertical leg reaction due to inertial loadset representing dynamics.
V_E	=	Vertical leg reaction due wind wave and current.
V_L	=	Vertical leg reaction due to maximum or minimum variable load at most onerous center of gravity.
V_{Lo}	=	Vertical leg reaction during preloading.
VH_D	=	Vector of vertical and horizontal leg reaction due to self weight.
VH_{Dn}	=	Vector of vertical and horizontal leg reaction due to inertial loadset representing dynamics.
VH_E	=	Vector of vertical and horizontal leg reaction due wind wave and current.
VH_L	=	Vector of vertical and horizontal leg reaction due to maximum variable load at most onerous center of gravity.
VHM_D	=	Vector of vertical, horizontal and moment leg reaction due to self weight.
VHM_{Dn}	=	Vector of vertical, horizontal and moment leg reaction due to inertial loadset representing dynamics.
VHM_E	=	Vector of vertical, horizontal and moment leg reaction due wind wave and current.
VHM_L	=	Vector of vertical, horizontal and moment leg reaction due to maximum or minimum variable load at most onerous center of gravity.
X_1,X_2	=	Beam buckling factors.
Z_i	=	Component plastic modulus.
γ	=	Load factor.
γ_1	=	Load factor for dead load vector.
γ_2	=	Load factor for variable load vector.
γ_3	=	Load factor for environmental load vector.
γ_4	=	Load factor for inertial load vector due to dynamic response.
λ,λ_c	=	Column slenderness parameter.
λ_p	=	Limiting slenderness parameter for compact component.
λ_r	=	Limiting slenderness parameter for noncompact component.
η	=	Exponent for biaxial bending.
ϕ	=	Resistance factor.
ϕ_a	=	Resistance factor for axial load.
ϕ_b	=	Resistance factor for bending.
ϕ_c	=	Resistance factor for axial load (compression).
ϕ_{Hfc}	=	Resistance factor for foundation to withstand horizontal loads when Q_V is acting.
ϕ_p	=	Resistance factor for foundation during preload.
ϕ_t	=	Resistance factor for axial load (tension).
ϕ_{VH}	=	Resistance factor for foundation to withstand combined vertical and horizontal loads.
ϕ_{VHM}	=	Resistance factor for foundation to withstand combined vertical, horizontal and moment loads.

8 GLOSSARY OF TERMS - ACCEPTANCE CRITERIA (continued)

ϕ_1 = Resistance factor for dead load moments (M_D).
ϕ_2 = Resistance factor for live load moments (M_L).
ϕ_3 = Resistance factor for seabed moments (M_S).

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