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Practice for the Analysis & Design of Marine Conductors

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Report EP 87-0160

Title : PRACTISE FOR THE ANALYSIS AND DESIGN OF MARINE CONDUCTORS

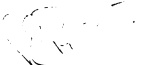
By : I.M. Hines and E. Berkel

Date : March 1989

This report presents the currently held technical opinion regarding the subject of the above title, and is endorsed as a Practice by SIPM-EP. SIPM-EPD/5 as custodian will either update/revise the document as appropriate or withdraw its status as an endorsed Practice. Any questions relating to this document should be addressed to the custodian.

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1. INTRODUCTION

Successful offshore exploration and production depends upon the provision of safe platform facilities from which to explore for and produce the available hydrocarbon resources. Whilst the structural performance of the fixed or floating structures themselves is obviously of vital importance, the integrity of the well system tubulars must also be maintained for the intended service conditions. The conductor may be considered as a separate structural system which, like the supporting structure, is also subjected to the full spectrum of environmental loads.

Conventional fixed platform facilities, even those in the deepest of waters, still rely upon relatively conventional marine conductors with above surface (deck level) wellheads through which to produce the reservoir gases and/or fluids. The conductor and well system is laterally supported by the surrounding structure at the plan bracing levels of the structure, as shown schematically in [Figure 1](#).

Much of the exploration and development drilling carried out in relatively shallow water depths, is performed using jack-up units. In this case a marine conductor is used to support and protect the well casing tubulars between the deck level of the jack-up and the seabed. A typical jack-up well configuration is shown in [Figure-2](#). The conductor is supported at the jack-up level and below the seabed. The marine conductor then comprises an end supported beam with a large free span subjected to combined lateral and axial loads.

Exploration and development drilling using jack-up units is progressing into deeper water and more hostile environments by virtue of the recent availability of significantly larger drilling units, which are claimed to be suitable for up to 100 metres water depth and for hostile environments such as the North Sea. In the past couple of years, several operating companies have asked SIPM to consider the possibility of drilling wells in water depths in excess of 70 metres, within the North Sea environment, on an all season drilling operation basis ([see Ref. C1, C2](#)). This poses interesting problems in connection with the procedures to be used for integrity assessments of both the Jack-up units and the marine conductor system. This report addresses the marine conductor system only.

A third configuration is shown in [Figure 3](#) and is a simple free standing conductor, which comprises a cantilever beam which is only supported at the seabed. In this case the lack of support at the top of the conductor introduces additional flexibility, which means that dynamic response effects predominate even in relatively shallow water depths.

In the design of the conductor system, it is necessary to provide sufficient seabed penetration and to ensure that the conductor and the well tubulars (i.e. casings) have sufficient strength and stability to accommodate the combined axial and lateral loading. Axial loading of the conductor results predominantly from the conductor self weight.

The casing tubulars will be predominantly loaded by a combination of self weight, weight of supported tubulars and the wellhead loads. Lateral loads, which generate bending stresses in both the conductor and casing strings, are induced by environmental forces, which, for flexible systems, may be amplified by the effects of dynamic response. In addition to examining the integrity of the system for prescribed storm loadings, it is also necessary to provide adequate resistance to the long-term effects of repeated loading and the consequences in terms of resulting fatigue damage. The effects of vortex induced vibrations, which may cause high stress levels and fatigue damage, is another item that should be investigated.

As with the analysis and design of the supporting structure, appropriate analysis methods and design acceptance criteria need to be adopted or developed to examine the integrity of the conductor and the well tubulars. The purpose of this document is to describe the procedures and analysis facilities which are presently available and in use for examining the in-place structural integrity of marine conductors. The document may be used as guidelines for engineers embarking upon conductor design or analysis.

[Section 2](#) deals with the assembly of the required data. The in-place storm analysis is discussed in [Section 3](#). The fatigue analysis and the vortex induced vibration analysis are contained in respectively [Section 4 and 5](#). The [Appendices A to E](#) provide simple hand calculation methods and examples, which may ease the analysis and design of marine conductors. Problems associated with the application of relevant design codes and acceptance criteria are discussed in [Appendix F](#). Throughout the report an attempt is made to highlight particular areas of uncertainty associated with predictions of the loading and the response and to examine the acceptance criteria and design codes currently used to assess the integrity of marine conductors.

A number of references have been assembled throughout the report, which should prove useful and enable the in-place problem to be tackled by individuals without previous experience of this type of problem. The report will emphasise that, despite the apparent geometrical simplicity of the marine conductor problem, an adequate assessment of the integrity of a marine conductor is far from trivial.

The existing document is restricted to the problems of analysis and design related to the in-place condition and does not examine the problems associated with the installation and required penetration or setting depth. As such it may be seen as a supplement to the existing report EF- 52510 (Conductor design and installation manual), which focuses specifically upon the installation related aspects of marine conductors.

2. ASSEMBLY OF DATA REQUIRED FOR ANALYSIS

2.1 Introduction

Where possible information and comprehensive details of all the important parameters involved, should be obtained directly from the operators. If any uncertainty exists as to the actual situation, it is recommended, if possible, to perform sensitivity analyses based on "best case" and "worst case" scenarios, which may then be expected to bracket the actual behaviour.

2.2 Data defining the well make-up

Reliable information on the well make-up is of vital importance in order to develop appropriate models for computer analysis and to establish the acting and allowable loads and stresses in the assembly tubulars. This information should include the diameter, wall thickness and yield strength of the nominal tubulars. Where optimisation of a design is required, to meet particular operating requirements, available options on size or steel type should be ascertained. The approximate locations and details of mechanical or welded connections should be established (at least for the outer conductor), in order to establish appropriate SN-curves or stress concentration factors for use in fatigue analyses ([see Section 4](#)).

In order to establish the axial loading of the tubulars ([see also Section 2.3](#)), details of the manner in which the casings are supported and the grouting operations during the well make-up are also required. The contents of all annular spaces between the assembly tubulars must be obtained in terms of the expected level and specific weight of the fluid (e.g. sea water or mud) or the grout. Grout levels, together with details of any mudline suspension system, will determine the degree of axial support and axial stiffness of the casings. Annular contents above mudline contribute to the total vibrational mass, which influences the lateral dynamic response.

Frequently, spacers or centralisers may be used to centralise the various tubulars and hence to improve the buckling stability of the internal casings. Details of the proposed locations or available options should be obtained, in order to establish the appropriate effective lengths. [Table 1](#) presents an example of a well make-up procedure for a typical offshore well drilled from a jack-up and using a 36 inch conductor. The various stages in the make-up of this well are shown in [Figure 4](#).

In the conductor shown in [Figure 4](#) and within the PETRONAS in general, it is common practice to isolate the outer marine conductor and the internal casing strings in terms of the applied axial load and the available load carrying capacity. In the completed assembly axial loads from the BOP (Blow-Out Preventer) and any other wellhead equipment are supported by the outer most casing string. For conventional wells (i.e. those without mudline suspension systems), internal casing strings and production tubings are also suspended from the top of the outermost casing and NOT from the marine conductor itself. This situation refers to the final conductor assembly; during the make-up of the well the conductor tubular may support axial loads from temporary diverter or BOF stacks. Furthermore the outermost casing is also temporary hung off from the top of the conductor prior to cementing, see the intermediate stages I and II shown in [Figure 4](#).

The above practice of separating the conductor and casing string is different from methods applied by some other oil companies, where the practice is to grout the annulus between conductor and outer casing also above mudline up to the top of the conductor. In this instant the assembly will behave differently regarding structural response and should be analysed by other techniques than included in this report (see e.g. [Ref. P8](#)).

2.3 Axial loading components

Accurate details of all axial loads must be compiled in order to develop appropriate computer models and to perform the design acceptance checks.

As already mentioned in [Section 2.2](#), axial loading on the outermost casing due to internally suspended casings is dependent on the way the well is made up and the extent of axial support provided by downhole grouting. Details of the downhole casing configuration and grouting levels must therefore be known. Similarly, mudline suspension of some casings may be employed either with or without casings suspended from the top of the assembly.

It is difficult to assess the precise extent of top loading on the outermost casing due to internally suspended strings which may also be supported at a lower level. The axial load due to the full buoyant weight of the suspended casing will initially act at the top of the supporting member and this load may be partially transferred to inner strings via axial shortening effects and the grouting operation. Furthermore, suspended casings may be partially supported in the drilling rig before grouting which further complicates estimation of the applied loads and the redistribution. During service, differential thermal expansions due to the flow of hot oil or cold kill fluid may still further complicate the situation.

In view of the uncertainties associated with this situation, it is recommended that, unless the true load distribution can be evaluated precisely (which is unlikely) a conservative approach should be adopted in which tension loads in suspended casings due to the buoyant self weight should not be assumed to redistribute during grouting.

However, where appropriate the load sharing resulting from axial shortening of the casing members of the assembly should be incorporated. This approach is also adopted in [Ref. C3, C4 and C5](#). The estimation of the required axial loading components will be discussed in [Section 3.4](#) in relation to the particular design acceptance check performed.

2.4 Boundary support conditions

Foundation support conditions, lateral restraint at the top of the assembly and top tension all have impact on the stiffness characteristics of the system and the effective length applicable in instability calculations.

Precise details of the extent and location of supports may be difficult to ascertain and furthermore support conditions may vary during the various stages of drilling operations. The influence of these uncertainties may be quantified to some extent by performing sensitivity analyses, in order to examine the expected range of system behaviour.

2.4.1 Support conditions at mudline

Where reliable information on the actual soil conditions is available these can be used to develop equivalent linear springs to represent the soil supports in structural analysis models (see Section 3). Where possible, specific soil data from a close borehole or recent site investigations should be used, since information obtained from even a short distance away may be inappropriate if the local soil conditions are variable. Frequently the outer conductor may be grouted into an oversize hole to a considerable distance below mudline. In such cases the soil resistance should be calculated on the basis of the outer grout diameter, but the bending stiffness of the assembly should be based on the nominal steel sections excluding the grout.

If appropriate information on the local soil conditions is not available, the concept of apparent fixity depth may be used as a first approximation (Ref. B3). In this case, the horizontal stiffness of the pile-soil interaction is represented by specifying the level below mudline at which the conductor can be assumed to be clamped. Experience has shown that a good approximation to the apparent fixity depth (i.e. the distance to the clamping point below mudline) may be as follows:

- Stiff clays : the apparent fixity depth = 3.5 - 4.5 D
- Very soft silts : the apparent fixity depth = 7.0- 8.5 D

(D = outer diameter of the pile or the grouted assembly)

General soil conditions, such as those obtained from, shallow seismic surveys may be used for guidance in the selection of the apparent fixity depth. In the absence of any details on the soil conditions the fixity depth may be set at 6 D.

For conductors supported laterally by either a conventional fixed platform, or a jack-up unit, reasonable approximations used in the development of the foundation support conditions are unlikely to have a major influence on the stiffness behaviour of the total conductor assembly and the resulting response levels. However, for freestanding conductors, where the foundation represents the only lateral support, large errors may result. In view of the variability in soil conditions and the inherent difficulties associated with the development of appropriate linearised soil support springs, sensitivity studies are recommended to bound the expected range of response levels for each conductor.

2.4.2 Lateral support above the mudline

Lateral support conditions should be established over the height of the assembly. These should include the internal support of casings (using centralisers) and external support at the plan levels of a fixed structure or the deck level support on a jack-up unit.

For a jack-up conductor at least one lateral support is provided by the jack-up unit. This generally takes the form of a guide cone attached to the top of a short cylindrical section which is located by a cantilevered bracket, or via a small spider deck beneath the drill floor. The support guides the tubular sections during installation and remains in place as a lateral restraint. A discrete rotational restraint is not usually provided. However, a second lateral restraint may be provided further up and close to the location of the BOP. In this case the two lateral supports have the ability to react the applied moment using lateral forces, which increase the overall stiffness of the conductor.

A lateral support may be modelled in the structural analysis program by the input of a true support or a stiff lateral spring at the appropriate elevation. Tolerances at the supports will usually be insignificant relative to the extreme lateral deflections of the conductor and may, therefore, be ignored. However, it is important that the precise location of every lateral support on the column is established. A lateral restraint in the region of a large mass (e.g. the BOP) will have a very significant effect on the dynamic behaviour of the structure. The exclusion of or inaccurate positioning of supports will alter the stiffness and load carrying characteristics of the system and could result in an inaccurate estimation of the bending moments and stresses in the assembly tubulars.

2.4.3 Top tension

The application of top tension may be necessary for deepwater wells drilled from jack-up rigs. Top tension of conductors is also considered for offshore structures like Tension Leg Platforms and jack-ups, which are specifically designed as a production platform.

In those cases where top tension is used it will alter the axial loading and lateral stiffness of the conductor assembly and its effect must be incorporated in the models used. Full details of the applied tension and the lateral support must therefore be obtained for incorporation in the structural analysis. This should include the precise location and magnitude of the maximum tensile load which can be applied. Differentiation between passive (so called static turnbuckle) and active (hydraulic) top tensioning systems should be considered; passive systems are unlikely to maintain the desired tension during severe storms.

2.4.4 Enforced top motions

For wells drilled from a jack-up or floating unit, motions of the top of the conductor should be considered in terms of their influence on direct stress levels and changes in the effective length assumptions for buckling stability calculations. This is discussed further in [Section 3](#).

2.5 Environmental criteria

Environmental criteria for the analysis should include details of wave and current data, tidal range and storm surge (where appropriate, related to the selected return periods), together with nominal waterdepth values and information on marine growth.

Frequently, jack-up operations will take place in areas for which the available environmental data is either sparse or non-existent. In such cases approximate data may be generated from adjacent areas by meteorological and oceanographic specialists.

2.5.1 Waterdepth

Variations in waterdepth due to tidal range and storm surge should be accounted for in the analysis. A sensitivity analysis may be performed to determine the waterdepth for which the maximum bending stress occurs in the conductor due to the design storm loading.

2.5.2 Wave data

For regular wave (deterministic) storm analysis, the "design wave" parameters are required in the form of the maximum wave height (Hmax) and associated period (Tass), both associated with a given return period. For random wave (spectral) analyses storm wave spectra must be generated (e.g. Pierson Moskowitz). These spectra are usually characterized in terms of the appropriate significant wave height (Hs) and the corresponding mean zero crossing period (Tz).

Should a spectral fatigue analysis be undertaken, then an area specific wave scatter diagram is required. This describes the joint probability of occurrence of sea-states characterised by a specified combination of significant wave height and mean zero crossing period.

In the absence of any other information it may be assumed that all directions of approach of waves have an equal probability of occurrence. However, there will be occasions, in particular in coastal waters, where this assumption may not be applicable ([see also Section 4](#)).

2.5.3 Current data

Specification of design current velocities for specified areas, environmental conditions and return periods should be carried out by, or in conjunction with, oceanographic specialists.

For purely wind-driven currents the velocity distribution may be assumed to vary linearly with depth from the specified value of the surface current to zero at the seabed. In the absence of better data the velocity profile of tidal currents may be calculated from a one-seventh power law as follows ([Ref. P1](#)):

$$V(x) = V(s) * (x/d)^{1/7}$$

In which : $V(x)$ = current velocity at distance x above mudline,

$V(s)$ = current velocity at the surface,

x = distance above mudline,

d = applied waterdepth.

2.5.4 Specification of return periods for environmental criteria

For the design of offshore structures the return period of the environmental criteria is generally selected based upon an accepted risk (or probability) that the specified design conditions will be exceeded. Typically, structural design codes and guidelines for offshore structures stipulate a 50 to 100-year return period for the storm environmental criteria (the so-called "survival conditions") for a structure with a service life of 25 to 30 years. A one-year return period is frequently employed for the analysis of the so-called "operating conditions". This approach is largely based upon risk assessment, reinforced by satisfactory operating experience.

For conductors which are to remain in service for the design life of the supporting structure (e.g. platform wells) return periods of environmental criteria used to examine the conductor are nominally the same as for the structure itself. Unless there exists a planned maintenance philosophy to replace conductor elements subject to fatigue loading at regular intervals.

For shorter-term operations, such as exploratory drilling, it may be appropriate to consider a reduction in the design storm criteria, which is consistent with the decrease in exposure duration (or service life). This is usually achieved by allowing the probability of occurrence of the design event for short-term operations, to be the same as that used for the design of a structure with a 25 to 30-year service life subjected to a 100 year return period storm wave ([see Ref. B4](#)).

The following return periods are recommended for the analysis of conductors:

- Conductors supported by a fixed structure (e.g. jackets or production jack-ups), with a service life of 25 to 30 years:
 - * operational conditions : 1 year
 - * survival conditions : 100 years
- Conductors for exploratory drilling from a jack-up; duration of the drilling operation approximately 3 to 6 months:
 - * operational conditions : 1 year
 - * survival conditions : 50 years
- Free standing conductors with a service life of 25 to 30 years:
 - * operational conditions : 1 year
 - * survival conditions : 100 years

2.5.5 Marine growth

The increase in wave and current loading on the structure due to marine growth fouling should be accounted for in the prediction of hydrodynamic loads.

Since information on the rate of marine growth is sparse (or non-existent in many areas), the application of marine growth thickness in the analysis of short-term operations remains largely a matter of engineering guesswork. In the absence of detailed information it is recommended that for operations of less than six months the marine growth may be neglected. For periods between six months and one year the marine growth thickness may merit consideration depending on the area and the season. For operations in excess of one year, marine growth in most offshore areas will be well established and the general design marine growth thickness for the area should be applied.

3. EVALUATION OF THE IN-PLACE BEHAVIOUR FOR EXTREME STORM LOADING

3.1 Introduction

This section describes the procedures presently available to predict the in-place behaviour of marine conductor assemblies under the Influence of design storm loading. The procedure may be subdivided into the following basic steps:

- i) Estimate the influence of the acting static axial loading components.
- ii) Evaluate the quasi-static or dynamic response of the assembly and the resulting bending stresses under the influence of the environmental and functional loading.
- iii) Perform design acceptance checks using interaction equations to examine the potential for buckling instability or local yielding of individual members or the entire assembly.

The key features of each of these steps are outlined in the subsequent sections. The presented procedures are based on the conventional working stress design method. However, ultimate limit state analyses might be necessary in order to determine safety margins of existing conductor assemblies. An example of an ultimate limit state analysis is presented in [Reference C6](#).

3.2 Prediction of conductor responses and bending moment distributions

The lateral loading and response of the marine conductor under the influence of combined wave and current loading can be evaluated using either a static deterministic or a random dynamic model of the environment and the subsequent response. Of these two fundamentally different approaches the first (based upon periodic design waves of fixed height and period) has been used almost exclusively for the design of fixed offshore structures. The use of the regular wave deterministic model is supported by its successful application within the industry for the design of a large number of, relatively shallow water, fixed platforms. These structures are generally rather stiff and do not experience significant dynamic response under the influence of the time varying environmental loading. In this situation the static deterministic model appears to offer a reasonable predictive tool when used in combination with existing design codes of practice.

Despite the fact that the random dynamic models are widely acknowledged to offer a better representation of the actual conditions in a real sea they have not, as yet, received widespread use for the design of fixed offshore structures. For slender structures, such as deepwater platforms or large span conductors, the natural frequencies may move within the range of significant wave energy and dynamic response may become an important design consideration. In this situation, deterministic models are considered inappropriate because design sea-state energy is associated with a single discrete wave frequency (see also [Ref. P.6](#)). If there is reason to expect that the structure may respond dynamically, then the response of the structure should be evaluated using random wave dynamic analysis procedures. These procedures account for the important relationships between the broader band frequency content of a real sea and the own natural frequencies of the structure in a consistent manner.

Both the random dynamic and static deterministic procedures have an important role to play-within the context of the marine conductor problem. For marine conductors installed within fixed offshore platforms, span lengths are generally limited so as to increase the fundamental natural frequencies and thereby avoid dynamic amplification effects in design storm conditions. In such instances the quasi-static bending stresses resulting from the extreme storm loading can be predicted using conventional deterministic models. It should be noted, however, that dynamic response due to milder everyday sea-states or vortex-induced vibrations may still need to be evaluated in view of the potential impact on maximum stress levels and cyclic fatigue damage, see also [Sections 4 and 5](#).

Slender free-standing or guyed marine conductors, or conductors used to drill wells from a jack-up unit, are generally more flexible and usually respond dynamically under the influence of extreme storm loading. For this situation the effects of the resulting mass inertial and damping forces must be taken into account and a random sea dynamic analysis is the most appropriate course of action.

Guidelines for the implementation of both options are contained in the following sections.

3.2.1 Static deterministic-analysis methods

For situations where no significant dynamic amplification effects are expected, a static deterministic analysis is considered appropriate. In general there are no well established criteria for defining precisely where the transition from pure quasi-static to significant dynamic response regimes occurs. However, it is tentatively suggested that if the fundamental mode natural period of the conductor (or the conductor span) is less than 2.5 seconds, or alternatively if the dynamic amplification of the maximum bending moment is less than 10%, then a full dynamic analysis is unnecessary and a quasi-static deterministic analysis is fully acceptable. Simple hand calculations may give a first estimate of the first natural period to be expected ([see Appendix A](#)).

The quasi-static bending stress response of a marine conductor assembly under the influence of a deterministic design wave can be estimated using hand calculations. However, it is most conveniently predicted using a computer model.

For the analysis of marine conductors the assembly may be idealised as a simple 2-D beam stick model with hydrodynamic properties which are based upon the outer member diameter of the conductor and stiffness characteristics based upon the combined section modulus of all the conductor and casing tubulars. If possible, the finite element formulation employed should consider secondary geometric stiffness effects (e.g. the P-delta effect), otherwise these must be evaluated separately.

Wave particle kinematics may be determined from the specified deterministic wave height and period using a periodic wave theory (e.g. Airy, Stokes 5th). A guidance on selection of an appropriate wave theory is presented in [Figures 5 and 6 \(acc. to Ref. P5\)](#). Hydrodynamic forces, resulting from the combined influence of co-linear steady current and wave particle kinematics for the periodic design wave should be established using the Morison wave force formulation. Wave forces should be integrated up to the instantaneous wave crest elevation. In line with present practice within SIPM a drag coefficient (C_d) of 0.7 is recommended for use with deterministic wave force predictions. An inertia coefficient (C_m) of 2.0 is considered appropriate for both deterministic and random sea conditions. Wave loading and conductor bending moment responses should be evaluated at the wave crest position corresponding to the maximum aid span moment which is established by stepping the regular wave past the conductor.

The interaction between conductor and soil may be modelled as follows ([see also Section 2.4.1](#)):

- 1) Non-linear lateral soil springs according to the available soil data (the structural analysis program must be capable to handle this).
- 2) Equivalent linearised lateral soil springs. Stiffnesses may be calculated using the available soil data and a program that can handle the non-linear conductor and soil interaction. Usually this approach involves an iterative process.
- 3) Equivalent linearised soil spring matrix at the mudline location; the stiffnesses will be calculated using the available soil data and a non-linear conductor/soil interaction program. Usually this approach involves an iterative process and the stresses and displacement conductor below mudline have to be determined with the conductor/soil interaction program.
- 4) An apparent fixity at a certain level below mudline. Values for this level are presented in [Section 2.4.1](#). This concept is used for first approximation analyses and in situations where specific soil data are not available or unreliable.

The length of the discrete segments of the conductor below mudline for options 1, 2 and 3 may be taken between 1 and 2 times the outer diameter of the conductor or the grouted assembly.

3.2.2 Random dynamic analysis methods

3.2.2.1 Frequency and time domain methods

For flexible systems, such as free-standing conductors or for wells drilled from a jack-up unit, dynamic response in extreme storm conditions may be significant. In such cases it is necessary to account for the dynamic response characteristics and the actual frequency content of the storm within the random wave environment. The prediction of the dynamic response of systems subject to random wave excitation can be achieved using either of the two following approaches; see also [Reference P6](#) for a more detailed discussion:

i) Frequency domain solutions

- Normally conventional linear spectral analysis techniques are used, which are relatively economical and easy to apply.
- The development of linear transfer functions is required which relate the system loads and responses to the input wave spectrum. This development involves the linearisation of the non-linear drag loading.
- Using the system transfer functions, the short-term statistics of the structural responses (e.g. standard deviations, mean zero crossing periods, etc.) are estimated for the prescribed design storm spectrum.
- Once the short-term statistical responses are available, procedures for extrapolating the short-term responses to predict the extreme values necessary for design assessments are required.
- The loading will be determined using linear wave theory in which wave forces are only integrated up to mean sea-level. Free surface loading effects need to be estimated separately.

ii) Time-domain simulations

- Storm wave spectra are used to develop random time histories of wave surface elevation and to define the wave particle kinematics field influencing the loading on the conductor. The resulting irregular time histories are used to determine the fluid loading on the conductor. The response of the structure is obtained by performing numerical time integration of the dynamic (or static) equations of motion. Analysis of the irregular response time histories must then be performed to establish the statistical properties of the response (e.g. standard deviation, mean zero crossing periods, etc.).
- Random sea time-domain simulations incorporate system non-linearities explicitly without the need for system linearisations. This is achieved at a penalty of computational cost which depends upon the length and number of simulations required and the size and complexity of the model.
- The method carries an additional penalty in terms of the difficulties associated with the interpretation and application of the results. In modelling the random environment it is assumed that the wave surface elevation is a stationary, ergodic, Gaussian process. As a result of system non-linearities, the response is certainly non-Gaussian, but it is also possible that it may not be stationary in a statistical sense either. Therefore, a single realisation of the non-linear structural

response only represents one of a theoretically infinite number of such realisations. In order to develop confidence in the short-term statistical characteristics of the response it is necessary to carry out a significant number of such simulations.

- Information on the extreme values, which are required for design assessments represent the tails of the peak response distributions, and can only be obtained reliably by performing a very large number of simulations, or by performing extrapolations based upon assumed extreme value distributions, e.g. the Weibull distribution. Even for the simplest of structures this process is both expensive and time consuming (and as the distribution for a non-Gaussian response is generally not known the extrapolation process invariably contains significant uncertainties).

The computational costs and difficulties associated with prediction of extreme level responses using random time-domain analysis procedures have prompted the development of a simpler engineering approach, with which to predict the extreme load levels, while still using random dynamic analysis methods, which is imperative as discussed at the beginning of [Section 3.2.2.1](#).

This procedure consists of the prediction of short-term response statistics in the design storm; i.e. mean values, standard deviations and mean zero crossing periods of each response variable of interest, using spectral analysis techniques or time-domain simulations.

The results of limited random time-domain simulations are then used to estimate the extent of the non-Gaussianity of the responses. Using this information, and making assumptions about the nature of the peak distributions of the loading and response variables, the ratios between the most probable maximum cyclic response (MPM) and the standard deviations (SD) are determined. The extreme, design level, loads and responses are then estimated using the following relationship:

$$\text{Extreme response} = \text{Mean} + \left(\frac{\text{MPM}}{\text{SD}} \right) \text{SD} \quad (3.1)$$

The various steps to estimate the extreme "storm" response levels are outlined in the subsequent sections.

3.2.2.2 Short-term conductor response using a frequency domain dynamic analysis

For the spectral dynamic analysis of marine conductors, the assembly may be idealised as a 2D finite element stick model. This model will usually account for bending, tensioning and lateral inertia. The hydrodynamic properties will be based upon the outer diameter of the conductor tubular, while the stiffness characteristics are based upon the combination of all effective tubulars. The mass of the tubulars and the contents of the annuli may be lumped at a sufficient number of nodes or, when possible, specified as a distributed parameter. The top tension force, if present, and the weight of the tubulars should also be included in the model. The weight of the fluids in the annuli and the buoyancy force from the seawater contribute to the lateral stiffness of the considered assembly and should be included as well ([see Ref. B13, B14 and Appendix B](#)). It should be noted that the external and internal fluids have no influence on the actual axial stress in the tubulars.

Wave and current loading on the conductor will be determined using Morison's equation. The application of spectral analysis techniques requires linearisation of the non-linear hydrodynamic drag loading. The most appropriate procedure presently available is based upon a sea-state dependent, first order statistical linearisation, as developed by Borgman ([see Refs. P6 and B5](#)).

Wave force coefficients used to evaluate the fluid loading should be consistent with the nature of the model used to predict the loads. Recent experimental studies have provided a substantial database for the determination of the drag and inertia force coefficients of the Morison equation, when used for the prediction of fluid loading in random seas. Results presented in [Reference B6](#), suggest dependency of the force coefficients on the ratio of the water particle displacement to the member diameter (the so-called A/D-ratio). For probabilistic response predictions it is recommended to use the relationship between the A/D-ratio and the drag and inertia coefficients, which are presented in [Table 2](#) (according to Ref. B16). A method for calculation of the A/D-ratios, which are characteristic for the total conductor should be determined and subsequently a drag and inertia coefficient should be selected from [Table 2](#). It should be noted that the values of 1.0 for the drag coefficient and 2.0 for the inertia coefficient have been used within SIPM for most of the probabilistic response predictions.

The damping in the system may be represented explicitly in the form of linear modal damping, implicitly by employing a linearised relative velocity formulation of the Morison equation or by using a combination of the two. Experimental evidence indicates that the relative velocity force formulation overestimates the system damping unless the standard deviations of conductor displacement over a considerable part of the conductor's length exceed magnitudes of the order of half a member diameter ([Ref. B7](#)).

Within SIPM it is current practice to apply an absolute velocity formulation with a system damping of 1 to 2% of the critical damping when the standard deviations of conductor displacement are less than half a member diameter. If the standard deviations of displacement are exceeding the member radius a relative velocity formulation without any additional linear modal damping is used.

Frequency domain analysis tools do not integrate the applied wave loading up to the instantaneous wave crest position and, therefore, do not include the influence of the free surface wave force loading effects. In an attempt to compensate for this, it is present practice to multiply the standard deviations of loading and response by an additional factor based upon the relative change in waterdepth associated with the design storm. The factor applied is the ratio of the waterdepth plus half the significant wave height, divided by the original waterdepth. This relationship was developed from comparisons between the standard deviations of conductor response obtained from random wave simulations when including and excluding the free surface wave loading effects. Comparisons were made for a limited number of conductors drilled from jack-ups in North Sea waterdepths exceeding 50 m ([e.g. Ref. C2](#)).

It should be recognised that the approximation of the free-surface effect is nothing more than an engineering postulate, obtained from a limited set of comparisons for a rather specific set of environmental conditions. In similar and deeper waters the approach is expected to give a reasonable estimate of the free-surface loading effects and any errors inherent in the method are unlikely to have a major influence on the total response levels. However, the application of this approach in shallower waterdepths, where the influence of the free-surface effects is likely to become very significant should be considered with some caution. In these situations, it is recommended where possible to obtain the short-term response predictions from the results of random time domain simulations including free-surface loading effects.

The soil/conductor interaction may be represented by approach 2, 3 or 4 from [Section 3.2.1](#). In principle, non-linear behaviour of soil support curves implies that the soil spring stiffness values are sensitive to load level. This change in foundation support with load level may be significant for free-standing conductors where the foundation provides the only lateral support. For conductors supported by jack-up units or fixed platforms the variation of the foundation support conditions with the applied loading is unlikely to have a major influence on the design storm response levels ([see Ref. C3](#)).

3.2.2.3 Short-term conductor response using time domain simulations

As described in [Section 3.2.2.1](#) the time histories of response created by time domain simulation tools may be used to establish short-term responses of a conductor also. Some uncertainty exists whether a single realisation of the non-linear structural response is sufficiently accurate to represent a theoretically infinite number of such realisations which may not be stationary in a statistical sense. However, comparison of short-term statistics of response which have been obtained with frequency domain and time domain tools showed that both methods may be used to estimate the short-term response statistics (e.g. mean and standard deviation). It is recommended to use a minimum simulation time of 1800 sec for this purpose.

The structural and hydrodynamic model for the time domain simulations will be similar as for the frequency domain analysis (see previous section), with the exception that non-linearities such as non-linear hydrodynamic force, and free surface effects will be included. Theoretically there is no limitation on the non-linear phenomena which may be included (e.g. non-linear conductor soil interaction). The drawback is usually the capability limits of the computer program and the costs.

3.2.2.4 Prediction-of most-probable maximum cyclic responses

Having obtained the short-term cyclic response levels it is necessary to extrapolate these, in order to obtain the most probable maximum cyclic responses occurring during a typical three-hour storm duration.

In order to facilitate predictions of the extreme response values from the short-term response predictions use has been made of the theoretically known probabilistic description of Morison type wave force loading when acting on an element of a single vertical pile ([Ref. B8](#)). For a linear Gaussian model the Most Probable Maximum value (MPM) is approximately 3.7 times the Standard Deviation (SD) value, assuming a three-hour storm having approximately 1000 peaks. However, if the process is non-linear and hence the response is non-Gaussian (i.e. as a result of drag loading), the ratio of the MPM and the SD of wave force may be significantly higher; as high as 8.6 in a totally drag dominant system.

[Figure 7](#) shows the ratio of the most probable maximum and the standard deviation of wave force, when predicted by the Morison equation for an element of a vertical pile, as a function of the parameter K (acc. to [Ref. B11](#)). The K parameter is proportional to the ratio of the standard deviations of the drag and inertia force components (see [Ref. B8](#)). For small values of K (i.e. the inertia dominant situation: Gaussian response) the MPM/SD ratio is 3.7, whilst for large values of K the ratio tends to the upper limit of 8.6.

For most practical situations a mixture of linear and non-linear contributions is evident in the loading and the actual MPM/SD ratio will be somewhere within this range. It should also be noted that the mix, and therefore, the degree of non-Gaussianity, will vary from one location on the conductor to another.

Using the information on the peak value distributions for the Morison applied loading, approximate methods have been proposed, to enable extrapolation to the peak values for structural responses. This extrapolation is achieved by making assumptions regarding the nature of peak distributions. It is postulated that the probabilistic description of the structural response variables in the amplitude domain may be directly related to those of the Morison wave force formulation for a single vertical pile. It must be stressed that these procedures remain engineering postulates which are an attempt to incorporate the effects of the non-Gaussian nature of the response on the resulting statistical distributions. It is almost certain that such postulates are not strictly valid. However, at present there is no theoretical or experimental evidence available to resolve above difficulties; by virtue of the existing lack of a better model the above approach is, therefore, pragmatically adopted.

In order to establish the appropriate value of the MPM/SD ratio, some limited random time domain simulations should be employed. Within the computer program, time histories of wave kinematics and wave loading will be developed from the prescribed storm wave spectra using inverse Fourier transforms. Time histories of structural responses are then developed by performing numerical time domain integration of the dynamic equations of motion. A statistical analysis of the response time histories determines the standard deviations and peak distributions of response parameters including non-linear effects (like Morison drag loading, free surface effects, P-delta effects, etc.). This information may then be used to establish the ratio of most probable maximum to standard deviation of any response as follows.

The probability density function of peak response amplitudes of the total force on a vertical pile as predicted by the Morison equation, is controlled by the drag/inertia K parameter, as described above. A measure of the non-Gaussianity in a random analysis may also be obtained with the so-called Kurtosis parameter (i.e. the ratio of the fourth moment about the mean of the probability density function to the square of the second moment about the mean, Ref. B8). It can be shown that the Kurtosis of the Morison wave force on an element of a vertical pile is related to the K parameter as follows (Ref. B8):

$$\chi = \frac{105K^4 + 18K + 3}{(3K^2 + 1)^2} \quad (3.2)$$

It is now assumed that the relationship between Kurtosis a parameter for the force on an element of a pile is also applicable for a general response variable (e.g. the conductor bending moment). The probability density function of the peak response can now also be expressed in terms of the Kurtosis and related to the K parameter as follows:

$$K = \left[\frac{(\chi - 3) + \left(\frac{26(\chi - 3)^{\frac{1}{2}}}{3} \right)}{(35 - 3\chi)} \right]^{\frac{1}{2}} \quad \text{for } 3 < \chi < \frac{35}{3} \quad (3.3)$$

This relationship is shown in [Figure 8](#).

The Kurtosis parameter of a particular response variable should be determined by performing a time domain simulation of the conductor assembly in the prescribed sea-state, with a minimum simulation time of 1800 sec. Using the computed Kurtosis, the drag/inertia parameter K may be determined from eq. (3.3) and hence the relevant MPM/SD ratio of the response variable may be estimated using [Figure 7](#).

As a first estimate of the most probable maximum of a response variable, an alternative procedure is recommended which does not require the estimation of the Kurtosis using random wave, time domain simulations. In this approach, the K parameter is estimated using the standard deviations of the response variable under the influence of drag loading only and inertia loading only. The required standard deviations of response may be estimated by performing separate frequency domain analyses with the

force coefficients (C_m and C_d) alternately set equal to zero. In the case of a vertical pile, for which a statistical linearisation of the drag forces has been performed the K parameter may subsequently be estimated as follows:

$$K = \frac{\sigma_D}{\sigma_I} \sqrt{\frac{\pi}{8}} \quad (3.4)$$

Where, σ_D = standard deviation of response for drag only Morison loading

σ_I = standard deviation of response for inertia only Morison loading

3.3 Axial loading effects

Unlike the cyclic environmental forces, which act laterally, the axial loads within the conductor assembly are pure static loads, which result from the influence of the wellhead equipment and the self weight of the tubulars and their annular contents. As discussed in [Section 2.3](#), the prediction and the influence of axial loading effects is different depending upon whether the local strength or instability of the assembly are being examined. Therefore, the estimation of the required axial loading components will be discussed in relation to the particular design acceptance check being performed in the following section.

3.4 Application of design code acceptance checks

In the subsequent sections, the implementation of the necessary buckling instability or local strength interaction code checks are described. These are essentially based upon the AISC/API working stress equations (Refs. P1, P2), but have been adapted where required to suit the particular features of the marine conductor problem, as discussed in more detail in [Appendix F](#).

3.4.1 Local yielding (strength) interaction checks

In principle, each member of the assembly (or at the least the conductor and outermost casing) should be checked for the tendency to local yielding, using the AISC strength interaction equation as follows:

$$\frac{f_a}{F_a} + \frac{f_b}{F_b} \leq 1.0 \quad (3.5)$$

Where, f_a = acting axial stress

f_b = nominal bending stress

F_a = allowable axial stress

F_b = allowable bending stress

The axial stress in the outer conductor will be due to self weight loads and a possible top tension force. The self weight component of the conductor itself should be computed on the basis of the self weight in air. The determination of the axial stresses due to the buoyant self weight of the casings which are hung off from the top of the outermost casing and also grouted at a lower level is subject to some uncertainties due to the installation sequence of the casing scheme and differential thermal effects when producing.

In view of the uncertainties, the acting stresses in the casing tubulars, should be determined in a conservative manner using the buoyant weights of the full length of the suspended casings. Buoyant weights should be determined from the submerged volume of steel and using the appropriate drilling mud density. Assuming the casing tubulars to be suspended from the top of the outer casing prior to grouting, the resulting loads should then be distributed to the appropriate tubulars in proportion to their relative axial stiffnesses and taking due account of the individual member axial shortening effects. This distribution should also take place for loads from the wellhead, which will usually be supported by all the casings. This requires a careful review of the member axial loads as shown in the example presented in [Appendix B](#) (taken from [Ref. C4](#)).

Bending stresses in well tubulars should include the influence of lateral environmental loads, the effects of any eccentricities in the casing assembly (if it has not been centralised) and if necessary dynamic response. The influence of environmental loads and dynamic response may be established using the procedures described in [Section 3.2](#). The effect of eccentricity of suspended casings on the bending stresses in these casings, can be determined using hand calculations based upon worst case eccentricity assumptions as described in [Appendix C](#).

Variations in the axial and bending stresses over the height of the conductor imply that the local strength interaction check should be considered at a number of locations along the assembly. However, the AISC local strength check is specifically intended for those locations where instability cannot occur. It is, therefore, suggested that local yield interaction checks are only developed at the support locations along the assembly, e.g. within the section of the conductor below mudline, at the deck level support of a jack-up, or at plan levels in a fixed structure.

The nominal allowable axial stress for compression loading in the absence of instability is 60% of the yield stress. For compact hollow sections, the nominal allowable bending stress is 66% of the yield stress. In accordance with the API code these nominal values may be increased by one-third when the incident loading is due in part or in total to stresses resulting from extreme design event environmental loads.

3.4.2 Examination of the buckling instability

The interaction equations specified in the AISC/API codes have been developed for single, uniform wide flange columns, subject to axial loads and bending moments applied at the member ends. In contrast, the marine conductor assembly consist of a series of concentric tubular members which are subjected to relatively small axial loading at the top of the assembly (i.e. from wellhead loads and the BOP stack), significant distributed loads from self weight of the tubulars and the annular contents, and lateral loads from the environmental forces. The relative proportions of axial loading in each of the tubulars in the assembly may also be markedly different, depending upon how the well is made up. This is particularly so if, as is common practice in the Shell group, the inner casing strings are hung from the outermost casing prior to cementing, and the outer casing also supports the wellhead loads. In this situation the outermost tubular, the marine conductor itself, carries very little axial loading; its primary function is to support the inner strings laterally and to protect them from the environmental loading.

The casing strings, either individually or collectively, have the biggest potential for instability (within the conductor), until they contact the conductor inner wall. Depending upon the annular spacing, the extent of any centralisers employed and the resultant curvature of the inner strings, this displacement may or may not render the strings unserviceable. Finally, if the applied axial loading is sufficiently large, the entire assembly may buckle. Clearly, stability of marine conductors is considerably more complex than that of the idealised, uniform, and loaded column. It is therefore suggested to use the following modified interaction ratio equations, which are discussed in more detail in [Appendix F](#).

If no top tension force is applied the buckling stability of the complete assembly should be checked as follows:

$$\frac{P_{act}}{P_{allow}} + \frac{f_{b'}}{F_b} \leq 1.0 \quad (3.6)$$

Where,

- P_{act} = acting (or equivalent) end axial loading contributing to instability
- P_{allow} = allowable end axial load
- $f_{b'}$ = acting bending stress in the conductor tubular, including second order (P-delta) effects due to axial loading and displacement
- F_b = allowable bending stress in the conductor tubular in accordance with [Ref.P1](#)

In this equation, the first term relates directly to the axial load effects but only considers the end axial forces which tend to cause instability. As described in [Appendix F](#), axial casing and conductor loads arising from portions of the assembly hanging below the lateral foundation support do not cause instability of individual members, or of the assembly ([see also Ref. P8](#)). If the conductor is embedded in the soil, the support point may be calculated using the apparent fixity depth concept ([see Section 2.4.1](#)) or the point below mudline where the maximum bending moment acts may be used. The estimation of the acting and allowable end load is discussed in more detail in [Section 3.4.2.1](#).

If the outer tubular of the considered assembly is also axially loaded by forces, which do not contribute to the instability of this assembly, the buckling stability should be checked as follows:

Axial load (instability)		Axial stress		Bending stress		
$\frac{P_{act}}{P_{allow}}$	+	$\frac{f_{ai}}{F_{ai}}$	+	$\frac{f_{b'}}{F_b}$	≤	1.0

(3.7)

Where,

- P_{act} = acting (or equivalent) end axial load contributing to instability
- P_{allow} = allowable end axial load
- f_{ai} = axial stress due to those axial loads which do not cause instability but do give rise to direct stresses (f_{ai} is caused by portions of the assembly hanging below the lateral foundation support)
- F_{ai} = allowable axial stress in accordance with [Ref.P1](#)
- $f_{b'}$ = acting bending stress in outer tubular including second order(P-delta) effects due to axial loading and displacement
- F_b = allowable bending stress of outer tubular in accordance with [Ref.1](#)

Cases which should be checked with equation 3.7 are e.g.:

- In order to check the buckling stability of the casing strings alone, without the conductor tubular, as an additional check to equation 3.6.
- In order to check the buckling stability of the casing string without the conductor tubular, in case top tension force is applied to the conductor tubular.
- In order to check the buckling stability of the assembly in the installation phase when the outer casing is temporarily hung off the conductor tubular.
- In order to check the buckling stability of an assembly, which has the casing strings grouted to the outer conductor over its full length (see Ref. P8). This is not common practice in the Shell Group.

The estimation of the acting and allowable end load is discussed in more detail in [Section 3.4.2.1](#). The calculation of f_{ai} , should be based upon loads arising from portions of the assembly hanging below the lateral foundation support (see [Section 3.4.2.2](#)). For the outer casing this support is usually the top of grout in the annulus between conductor and outer casing.

Since the second term in eq. 3.7 relates to the direct stresses caused by those axial load components which do not contribute to instability, it may be expressed in terms of the ratio of stresses, without the transformation to loads which was required for the instability term.

In both the eqns. 3.6 and 3.7 the final term represents the ratio of the acting and allowable bending stresses. The acting stress should include the influence of second order axial load/deflection affects (P-delta) which may be calculated explicitly using finite element programs which incorporate geometric stiffness terms or estimated using the bending stress amplification approach prescribed in Section 1.6 of the AISC code (see also [Appendix F](#)).

3.4.2.1 Estimation of acting and allowable end loads

At the various stages in the make-up of the well, it is required to estimate the applied axial loading effects and compare these with the allowable values for either single members, or for a number of concentric tubulars. This can be performed manually, by making assumptions about the effective lengths of the tubulars under consideration, as follows:

- Applied axial loads (P_{act})

In line with the development in [Appendix F](#), the combined effects of the wellhead loads and the distributed self-weight forces should be expressed in terms of an equivalent axial force acting at the end of the member. The part, which represents the effects of distributed weight may be estimated on the basis of virtual work considerations for an assumed or calculated buckling mode shape. [Appendix D](#) gives the required calculations for a simple beam having uniform distributed weight, for several of the most common end support conditions and the corresponding assumed node shapes. The applied axial load will then be calculated as follows:

$$P_{act} = P_{top} + \alpha * W \quad (3.8)$$

Where,

- P_{top} = sum of the axial loads from wellhead equipment, BOP, etc. acting at the top of the assembly
- W = sum of the distributed axial loads which contribute to the instability of the assembly under consideration (i.e. the sum of the non-buoyant tubular weights above the mudline support)
- α = a factor which represents the properties of the total distributed weight to be applied at the end of the member in order to achieve equivalent work done during instability (see Appendix D).

Determination of the actual applied top loads can be performed easily from the wellhead load data. Estimation of the second term, representing the equivalent end load effect of distributed weight is less straightforward and requires a more careful treatment of the various elements.

As discussed previously, for a centralised casing assembly axial loads arising from portions of the casing below the lateral foundation support do not contribute to instability of the system and do not need to be considered. However, the distributed weight of the well tubulars above this support do contribute to instability; these must be included in the estimation of the equivalent end axial load. For the purpose of estimating the distribution loads which contribute to instability it is recommended to use the sum of the non-buoyant weights of all the well tubulars for the above mudline sections of the assembly.

The above estimation of the equivalent acting axial end load can be performed manually. However, if computer programs are available to estimate the Euler elastic buckling characteristics of the assembly, a second approach is possible. This will be based upon changes in the end axial force required to induce elastic Euler buckling for the actual assembly (including all self weight and annular fluid contents) and for a hypothetical situation comprising the same assembly but without the influence of distributed self-weight loads.

- Allowable axial loads (P_{allow})

The allowable axial load for the assembly under consideration may be determined using a finite element program, which calculates the Euler elastic buckling load, incorporating the effect of the external and internal fluids on the lateral bending stiffness (see Ref. B13).

For manual approximations of the Euler buckling load, it is first necessary to estimate in appropriate slenderness ratio for the section of the column which may undergo instability. This must be based upon the effective slenderness ratio of the column when translated to an equivalent pin ended beam under axial end load condition.

$$\text{Slenderness ratio} = \frac{kL}{r}$$

k = effective length factor

L = column length for instability

r = radius of gyration = $\sqrt{I/A}$ where:

I = second moment of area

A = cross section area

Appropriate k-factors for a variety of end support conditions may be found in the AISC commentary Section 1.8, which are also shown in [Table 3](#).

For a single member, the threshold slenderness for the transition from elastic (Euler) to inelastic buckling is defined by the critical slenderness ratio C_c which is related to a proportional stress limit of 50% of yield (see Appendix F or Ref. B1).

$$C_c = \sqrt{\frac{2\pi^2 E}{F_y}} \quad (3.9)$$

Where,

F_y = material stress
 E = Young's modulus

For $k1 / r > C_c$ (Elastic buckling) the allowable axial load is the Euler load divided by a factor of safety of 1.92 (23/12):

$$P_{allow} = \frac{12 \pi^2 E I A}{23 (k1)^2} \quad (3.10a)$$

For $k1 / r \leq C_c$ inelastic buckling occurs and then

$$P_{allow} = \frac{F_y}{SF} * \left[1 - \frac{\left(\frac{k1}{r} \right)^2}{2 C_c^2} \right] * A \quad (3.10b)$$

Where,

A = the cross-section area of the member
 SF = AISC safety factor given by:

$$\frac{5}{3} + \frac{3}{8} \left(\frac{k1 / r}{C_c} \right) - \frac{1}{8} \left(\frac{k1 / r}{C_c} \right)^3 \quad (3.11)$$

For an assembly comprising a series of concentric members, the allowable axial load is estimated from the sum of the allowable loads for each of the members.

In dealing with composite concentric tubular assemblies, the well make-up must be reviewed to identify the possible modes of instability and to establish the appropriate effective lengths for each of the members being considered. In cases where there is no axial load transfer between the outer conductor and the internal string potential exists for the casing to buckle within the outer conductor, in either the fundamental or higher buckling modes.

For the examination of instability of the casing within the outer conductor, the effective length of the casing members should be based upon the positions of centralisers (or other supports) used to locate the outermost casing within the marine conductor. For the examination of instability of the whole assembly, the effective length of all members within the assembly should be based upon the support conditions for the outer conductor only since these will ultimately control the mode of global instability of the entire assembly.

In the AISC code, the allowable axial load is only related directly to the Euler load for cases where elastic stability controls, i.e. $KL/r < C_c$. This criterion corresponds to a uniform compressive axial stress within the member which is less than half of the yield stress. For a single member, application of this criterion imposes no difficulty, however, it cannot be applied directly to a series of concentric tubulars in which the various conductor and casing members are loaded to varying levels. In this situation it is necessary to examine how the equivalent axial Euler loads would be carried by the members of the assembly if they were applied to the top of the assembly.

In cases where the outer conductor is not grouted to the inner casings above mudline the only contribution to direct axial stress in the conductor is from its own self-weight forces. These are unlikely to result in stresses which exceed the 50% of yield criterion except for wells in very deep waterdepths. From the knowledge of the well make-up it is possible to estimate the individual stresses in each member of the casing assembly on the basis of their respective section areas. If any of the members in the assembly are stressed beyond half of yield, it should be assumed that this member (or members) do not contribute to the elastic buckling resistance of the total assembly. An estimate of the allowable load may then be obtained from the Euler load as follows:

$$P_{\text{allow}} = P_{\text{euler}} * \frac{I_1}{I_2} * \frac{12}{23} \quad (3.12)$$

I_1 = second moment of area of the assembly excluding those members of the casing which violate the assumed proportional stress limit of 50% of the yield stress

I_2 = second moment of area of the entire assembly

3.4.2.2 Influence of axial loads which do not cause instability

Axial loads from downhole tubulars do not contribute to instability as a result of the so-called Bowden cable effect (see [Appendix F](#) and Ref. P8). However, they do not give rise to direct stresses in the casing tubulars and may also load the outer conductor if the outer casing is grouted to the conductor above mudline. The effects of these axial loads are represented in terms of the direct stresses which they cause and this effect is included in the term f_{ai}/F_{ai} in interaction eq. 3.7.

Axial stresses due to the downhole tubulars should be determined using the procedures outlined in [Section 3.4.1](#) for completion of the strength check. In this instance, however, the stresses should be based only on the axial loads caused by the sections below mudline. The contributions from the above mudline sections have been accounted for in the first term of the interaction equation ($P_{\text{act}}/P_{\text{allow}}$) when considering the components which contribute to instability.

3.4.2.3 Locations to be considered for stability checks

Some doubt exists with respect to the precise locations for which the instability check should be performed. Whilst the actual acting axial loads in the well tubulars are not constant, the equivalent end axial forces required in the equations do not vary along the length in the formulations described here. However, the bending moment does vary along the length of the assembly. Therefore the maximum bending stress may occur anywhere along the conductor depending upon the applied loading and the support conditions. It is recommended that the instability interaction ratio should be examined at the location(s) corresponding to the maximum moment occurring between the supports and not at the supports themselves, since these points define the locations where deflections are limited and hence instability is not likely to occur (see also Ref. P8). For a free-standing conductor, the maximum bending moment will occur below mudlines. In this case this location will be appropriate to perform the instability interaction ratio check.

3.5 Influence of jack-up unit motions

For conductors drilled from a jack-up unit, the motions of the jack-up itself under the influence of the environmental storm loads, may have a detrimental influence on the stress levels in the member of the conductor assembly. A detailed assessment of the influence of the motions of the jack-up unit requires a complete analysis of the jack-up and coupled conductor response, including any dynamic effects. Clearly this would demand a significant increase in the scope of the analysis which is unlikely to be justified in most cases; a possible exception would be the analysis of conductors for jack-up production units. For other cases, a simpler approach is recommended in which the sensitivity of conductor stresses to enforced top motions is examined. The jack-up motions are imposed as static top displacements of varying amplitudes. Stresses are computed, using simplifying assumptions regarding the end support conditions and deflected shapes. Examples of the approach are contained in Refs. C3, C4 and C5.

3.6 Available computer tools

3.6.1 Static deterministic analysis

Within SIPM the computer program 'SESAM' is available for static deterministic analyses as discussed in Section 3.2.1. This program is suitable to carry out quasi-static, linear elastic analysis of arbitrary shaped offshore structures. For the analysis of conductors simple, 2-D beam stick models can be employed.

3.6.2 Random dynamic analysis

For dynamic analysis of conductors in the frequency domain as discussed in Section 3.2.2.2, the in-house developed computer program 'FREERISE' (Ref. B16) is used within SIPM. This program uses classical linear spectral analysis techniques. A finite element 2-D stick model with lumped parameters is employed, enabling variations in element mass, stiffness and wave loading properties along the length of the equivalent beam. Element weight and associated P-delta effects are also included in the formulation. The program includes an eigenvalue extraction subroutine, enabling the fundamental modes of vibration and the buckling modes to be calculated.

Time domain simulations of conductor responses under random wave loading as discussed in the Sections 3.2.2.3 and 3.2.2.4, are performed within SIPM with the in-house developed computer program 'MARIANTO' (Ref. B17). This program develops time histories of wave kinematics and wave loading from the prescribed storm wave spectra. Time histories of structural responses are then developed by performing numerical time domain integration of the equations of motion. A statistical

analysis of the response time histories provides, standard deviations and peak distributions of response parameters including non-linear effects (e.g. Morison drag loading, free surface effects, P-delta effects, etc.).

The SESAM computer program, which is within SIPM available as well, is also capable to perform dynamic analysis of conductors in both the frequency and time domain. However, at this moment in time statistical cost processing of intermediate results, in order to design conductors according to methods described in this report, is not (yet) available within the SESAM computer program.

4 EXAMINATION OF FATIGUE DURABILITY

4.1 Introduction

For relatively flexible conductor systems such as free-standing conductors, conductors used to drill wells from jack-up units, or conductors installed in fixed structures with large bay heights, dynamic amplification of response and the associated cyclic fatigue damage may become a critical design condition even for relatively short duration operations. Even for conductors or conductor spans of more modest lengths, cyclic bending stresses resulting from lateral environmental loading (or vortex-induced vibration) may also be an important design consideration. An assessment of fatigue durability is, therefore, considered an essential design requirement for relatively flexible conductor systems.

This section examines the procedures and tools which are suitable for the examination of cyclic fatigue damage under the influence of environmental wave action. The fatigue damage resulting from vortex-induced vibrations is not discussed here; see [Section 5](#).

4.2 Outline of the procedure

The assessment of the fatigue sensitivity of a structure can be carried out in a number of ways but the most rational and cost-effective approach to the problem involves the use of a probabilistic fatigue analysis using linear spectral analysis techniques ([Ref. B9](#)). This analysis procedure involves the following basic steps:

- a) Specification of a statistical description of the wave environment for the area of interest in the form of a wave scatter diagram. This defines the joint probability of occurrence of sea-states of given significant wave height and mean zero crossing period.
- b) Derivation of stress transfer functions relating the amplitudes of the hot spot stress variations to the wave amplitude for each location of the member under consideration.
- c) Processing of the stress transfer functions for each sea-state in the wave scatter diagram to arrive at short-term and long-term stress range distributions for the member locations of interest.
- d) Use of the long-term stress range distribution and an appropriate S-N curve to find the accumulated damage and hence the calculated fatigue life for each member location using Miner's rule.

The basic features of the procedure are illustrated in [Figure 9](#).

While simpler approaches are available, it is recommended, in view of the potential influence of dynamic response, to assess the fatigue durability of conductors using the above described procedure. The basic steps b), c) and d) are discussed in the subsequent sections.

4.3 Calculation of bending stress transfer functions

Use of the conventional spectral analysis techniques requires that the amplitudes of the response variations should be linearly related to the wave amplitude. However, the actual wave loading, and hence the response of the conductor, are non-linearly related to the wave height largely due to the influence of velocity squared drag forces. The influence of these non-linear components increase in significance with the severity of the sea-state. For fatigue calculations, the constant wave steepness linearisation gives the best overall prediction of fatigue damage (see Ref. B10).

Linearised stress transfer functions should be established for the member location(s) corresponding to the minimum bending stress occurring over the length of the conductor, including the region below mudline. The current is assumed to be included in the mean load level and is assumed to have a negligible effect on the cyclic parts of the stresses. Therefore, the transfer functions should not incorporate the influence of current.

Since the linearisation, and hence the resulting transfer functions are sea-state dependent it may be necessary to develop a separate set of response transfer functions for each and every sea-state in the scatter diagram or, more practically, for selected blocks of sea-states. Whilst this represents the most rigorous approach it is difficult from the viewpoint of data handling and implementation and also incurs much higher costs. As a result a simpler approach is recommended which makes use of a single response transfer function for all sea-states. The sea-state selected, and hence the resulting transfer functions are then considered to characterise the response for all sea-states in the scatter diagram.

As a first approximation it is suggested to develop the response transfer functions using a relatively severe sea-state such as the one-year condition. This will accentuate the influence of drag loading and should provide an upper bound transfer function for use in the subsequent fatigue damage calculations.

Alternatively, the response transfer functions may be developed using the "so-called" centre of fatigue damage sea-state (Ref. B10). The centre of fatigue damage sea-state is determined by processing the scatter diagram with a constant unit stress transfer function. For every sea-state in the wave scatter diagram a damage figure may be calculated to produce a corresponding damage scatter diagram. The damage figure D per sea-state is calculated as follows (see Ref. B10):

$$D = \frac{p(H_S, T_Z) \times (H_S)^{2m}}{T_Z}$$

Where,

$P(H_S, T_Z)$ = probability of occurrence of the considered sea-state

H_S = significant wave height of the considered sea-state

T_Z = mean zero crossing period of the considered sea-state

m = inverse log-log slope of an appropriate S-N-curve