GUIDANCE NOTES ON

AIR GAP AND WAVE IMPACT ANALYSIS FOR SEMI-SUBMERSIBLES

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1701 City Plaza Drive
Spring, TX 77389 USA

Foreword *(1 May 2020)*

These Guidance Notes supplement the Rules and Guides that ABS has issued for the classification of semi-submersibles, which refer to column-stabilized drilling units, column-stabilized units and column-stabilized installations. Unless the upper structure and deckhouses are satisfactorily designed for wave impact, ABS classification requirements mandate a reasonable air gap between the deck structures and the wave crests for all afloat modes of operation, taking into account the predicted motion of the semi-submersible relative to the surface of the sea. Calculations, model test results, field measurements or the combinations of these methods may be used to determine the appropriate air gap. In the event that a positive air gap is not maintained, structures should be appropriately designed to resist the expected horizontal and vertical wave impact. These Guidance Notes address the air gap analysis horizontal and vertical wave impact assessment methods and design considerations.

These Guidance Notes provide detailed procedures for the air gap analysis for semi-submersibles. The suggested method is based on direct analysis of vessel motions and wave free surface elevations using the linear diffraction-radiation analysis or the second order analysis. Guidance is also provided on the definition of the horizontal wave impact load and the strength assessment methods for those local structural members on peripheral sides of the upper structure subject to horizontal wave impact loads. For the vertical wave impact on the deck bottom structures of semi-submersibles, guidance is provided on the analysis based on the relative vertical velocities between the vessel motions and wave free surface elevations using the linear diffraction-radiation analysis with consideration of the wave asymmetry factor, the local wave crest enhancement, and the wave run-up factors.

Additional considerations may be needed for a particular case, especially when a novel design or application is being assessed. These Guidance Notes should not be considered mandatory, and in no case are these Guidance Notes to be considered a substitute for the professional judgment of the designer or analyst. In case of any doubt about the application of these Guidance Notes, ABS should be consulted.

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SECTION 1 Introduction

1 General *(1 May 2020)*

The term *Semi-Submersibles* used in these Guidance Notes refers to:

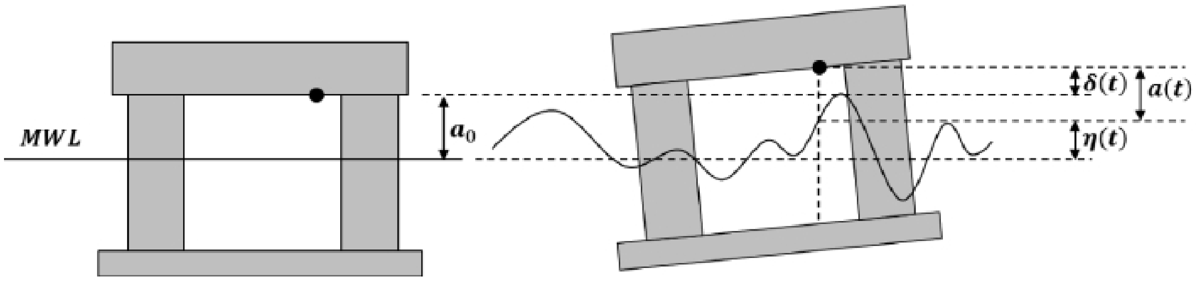
- Column-Stabilized Drilling Units and Column-Stabilized Units as defined in the ABS *Rules for Building and Classing Mobile Offshore Units (MOU Rules)*
- Column-Stabilized Installations as defined in the ABS *Rules for Building and Classing Floating Production Installations (FPI Rules)*.

A semi-submersible is a column-stabilized floating offshore structure, which consists of a deck structure with large diameter support columns attached to submerged pontoons. In the design and operation of semi-submersibles, the air gap as defined in 1/3, also known as the deck clearance, is an important design parameter. A low air gap may lead to wave impact that the structures and equipment of a semi-submersible are not designed for and could potentially cause serious damage. Conversely, an excessive air gap could increase the cost, decrease the stability, reduce payload capacity and impair global performance.

These Guidance Notes provide procedures for the air gap analysis based on either the linear or the second order potential theories, address nonlinear effects on the air gap analysis, and identify industry common design practice. The theories and equations referenced in these Guidance Notes are based on the infinite water depth assumption. However, the procedures described in these Guidance Notes are in general also applicable to finite water depth condition.

3 Definition of Air Gap *(1 October 2018)*

FIGURE 1
Air Gap Definition



An air gap or a deck clearance represents the distance between the water surface and the lowest deck of a semi-submersible. 1/3 FIGURE 1 gives a schematic view of the air gap in the absence of waves and in the presence of waves. In still water, the air gap at a particular location (x, y) on the lowest deck is denoted as $a_0(x, y)$, which is determined by the hydrostatic analysis. In the presence of waves, $\eta(x, y, t)$ denotes the free surface elevation at a particular location on the lowest deck at time instant t, and $\delta(x, y, t)$ is the corresponding vertical motion of the point on the platform at time instant t. In these Guidance Notes, a location on the structure selected for air gap calculation is referred to as a *field point*. The instantaneous air gap at a field point is:

$$a(x,y,t) = a_0(x,y) - [\eta(x,y,t) - \delta(x,y,t)] \quad (1.1)$$

The total change of air gap is a linear combination of two sources: free surface elevation, $\eta(x,y,t)$ and corresponding vertical vessel motion, $\delta(x,y,t)$, at a field point. In this document herein, the total change of air gap is referred to as the relative wave elevation, $r(x,y,t)$.

$$r(x,y,t) = \eta(x,y,t) - \delta(x,y,t) \quad (1.2)$$

Deck impact occurs if relative wave elevation, $r(x,y,t)$, exceeds the still water air gap, $a_0(x,y)$ and the air gap $a(x,y,t)$ becomes negative. The air gap analysis in these Guidance Notes is to determine the minimum air gap corresponding to the extreme relative wave elevation in storm conditions.

5 Air Gap Design Considerations

5.1 General *(1 May 2020)*

The air gap should normally be determined by an appropriate model test. Alternatively, the air gap may also be calculated using numerical approaches that account for relative motions between the semi-submersible and waves. The following items should be considered in the determination of the air gap:

i) Environmental conditions specified in the *MOU Rules* for semi-submersibles operated as mobile offshore units or the *FPI Rules* for semi-submersibles operated as floating production installations;

ii) Environmental condition headings relative to the semi-submersible;

iii) Motions of semi-submersible in six degrees of freedom;

iv) Wave crest elevation, including wave asymmetry;

v) Wave/structure interaction effects (e.g., wave enhancement, run-up);

vi) Effects of interacting systems (e.g., mooring and riser systems);

vii) Local wave crest effects as described in APR RP 2FPS, where applicable.

viii) Maximum/minimum operating drafts.

Wave enhancement effects should be appropriately considered in the air gap analysis. In the vicinity of large bodies, the free surface elevation can be enhanced by motions, diffraction, radiation, wave/current interaction effects, and other non-linear wave effects. These should be accounted for, as appropriate, in the wave action calculation and used to estimate deck clearance and freeboard. Wave run-up along the column reaching above the underside of the deck box should be considered for the vertical wave impact analysis. The structures and equipment in the path of wave run-up on column and deck box should be able to withstand the effect of wave run-up.

For semi-submersibles operating as floating production installations, the ABS *FPI Rules* require a reasonable clearance between the lowest point of the topside deck and the wave crest. A commonly referenced minimum deck clearance is 1.5 m (5 ft), see API RP 2FPS, in the case where the semi-submersible is subject to the 100-year return period environmental conditions. Consideration is also given to performing the robustness check, in which a rarer, but still possible event (e.g., 1,000-year wave crest in the Gulf of Mexico or 10,000-year return period wave crest in the North Sea) is applied and the topside deck elevations should be such that no negative air gap takes place. If the air gap criterion is not satisfied, the anticipated local and global wave forces (including slamming) in both vertical and horizontal directions, where applicable, should be suitably considered in the design of the structural strength. Guidance on the assessment of local structures on peripheral sides of the upper structure subject to horizontal wave impact is provided in Section 6. Guidance on the assessment of local structures underside of the upper structure subject to vertical wave impact is provided in Section 7.

For semi-submersibles operating as mobile offshore units, the ABS *MOU Rules* require a reasonable clearance between the lowest point of the topside deck and the wave crest, unless the upper structure and deckhouses are satisfactorily designed for wave impact. For mobile offshore units with a DP system, larger vertical motions due to rolling or pitching induced by overturning moment generated by the force from the thrusters, especially at field points near the columns and at the extreme of ends of the unit, may lead to a smaller air gap than what is predicted from numerical analysis. A static trim due to the unfavorable combination of thrust forces may be applied in the absence of a more detailed analysis.

For semi-submersibles operating as mobile offshore units but which are intended to be deployed at one site for no less than 5 years without dry docking, the air gap analysis should in general follow the requirements for floating production installations.

5.3 Storm Condition *(1 May 2020)*

For semi-submersibles operated as mobile offshore units, environmental conditions specified in the *MOU Rules* for the severe storm condition should be applied.

For semi-submersibles operated as floating production installations, environmental conditions specified in the *FPI Rules* for the design environmental condition (DEC) should be applied.

5.5 Operating Condition *(1 May 2020)*

5.5.1 Normal Operating Condition for Mobile Offshore Units

For semi-submersibles operated as mobile offshore units, environmental conditions specified in the *MOU Rules* for the normal operating condition should be applied. The draft and loading condition in the normal operating condition can be different from those in the severe storm condition.

In addition, operational restrictions other than local structural strength requirements may govern the air gap assessment for the operating condition where the non-negative air gap may be required for safe operation of equipment and safety of personnel.

The limiting sea states for non-negative air gap for normal operating condition can be derived using either the frequency-domain approach based on the linear air gap analysis (Section 4) or the time-domain approach based on the second-order air gap analysis (Section 5). For the frequency-domain approach, the short-term approach in 4/5.1 or the long-term approach in 4/5.3 can be applied.

Section 4/9 provides the recommended method based on the frequency-domain approach for derivation of limiting sea states for semi-submersible in the operating condition.

If the unit is intended to deballast in the operating condition in order to increase the still water air gap, a procedure on when and how such operation to be carried out should be included in the operation manual.

5.5.2 Design Operating Condition for Floating Production Installations

For semi-submersibles operated as floating production installations, environmental conditions specified in the *FPI Rules* for the design operating condition (DOC) should be applied.

The draft for DOC is normally the same as that for design environmental condition (DEC). Air gap and wave impact loads are normally governed by design environmental condition (DEC).

5.7 Transit Condition *(1 May 2020)*

The draft and loading condition for the semi-submersible in the transit condition can be different from those in the design environmental condition for floating production installations or the severe storm condition for mobile offshore unit. Air gap should be assessed for the transit condition.

However, for transit condition, environmental condition is in general limited by the wave impact load on the bracing members with the consideration of transit speed. The limiting environmental condition for transit condition is normally milder than that of the operating and storm conditions. If there is negative air gap issue during ocean or field transit for wet tow, the semi-submersible can be ballasted to the survival draft for safe operation. Air gap analysis for transit condition may be exempted if a procedure on when and how such operation to be carried out be included in the operation manual.

For semi-submersibles operated as mobile offshore units, environmental conditions specified in the *MOU Rules* for the transit condition should be applied.

For semi-submersibles operated as floating production installations, environmental conditions specified in the *FPI Rules* for the transit condition (ocean transit and field transit) should be applied.

5.9 Other Considerations *(1 May 2020)*

In addition, the following should be considered regarding air gap design:

- Where negative air gap occurs, global structural integrity of the strucutre should be maintained. The evaluation can be made through model test and/or advanced numerical analysis.
- Safety related equipment should remian functional after wave impact.
- Personnel safety should be managed by evacuation or restrcition to areas exposure to wave impact.
- Equipment and system should be removed from areas subject to the negative air gap. Otherwise, strengthening, protection or shielding should be considered for pontentail wave impact.
- Work areas, access areas, escape routes and safety critical equipment should not be exposed to wave run-up.

For semi-submersibles operated as floating production installations, pre-service conditions specified in the *FPI Rules* should be considered for air gap design.

7 Air Gap Analysis Methods

The following four methods may be used for the air gap analysis. It is possible, however, that other approaches may also exist.

7.1 Linear Air Gap Analysis

Linear air gap analysis relies on the linear three-dimensional diffraction-radiation theory to calculate the first order wave frequency vessel motions and free surface elevations at field points in frequency domain, and then uses a stochastic method to determine the extreme relative wave elevation. Corrections are applied to approximately account for the nonlinear effects of wave surface elevations and vessel motions.

This approach is commonly used in practical application due to its simplicity and ease of use. Empirical corrections are applied through a wave asymmetry factor and a wave run-up factor to consider the nonlinearity of the free surface elevation and the local run-up, respectively. Without these corrections, experimental observations indicate that the linear analysis may significantly under-predict the crest elevation in steep waves and, in particular, at locations very close to columns where local run-up may appear. In addition, the mean and low frequency vessel motions should be considered for the air gap design of semi-submersibles. Current industry practice, however, often ignores the mean and low frequency vessel motions for the air gap design of semi-submersibles operated as mobile offshore units. The mean and low frequency motions can be calculated by the global motion analysis in either frequency domain or time domain.

7.3 Second Order Time-Domain Air Gap Analysis

The second order air gap analysis attempts to model the nonlinear effects of vessel motions in time domain. It usually can improve the prediction accuracy and result in a better agreement with model test results. The free surface elevation can be determined through the linear diffraction-radiation theory and adjusted using empirical corrections as described above in 1/7.1. Alternatively, a second order method based on the second-order diffraction theory may be applied to model the asymmetry in the free surface elevation provided the analysis results of the free surface elevation and the correction to the wave run-up at near-column locations are validated by model tests.

7.5 CFD-Based Air Gap Analysis

Computational Fluid Dynamics (CFD) analysis can solve complex nonlinear flows around an offshore structure. This may be the most promising numerical approach to model the wave run-up effects. However, the computational cost is high and the analysis method should still be validated by the comparison with model tests.

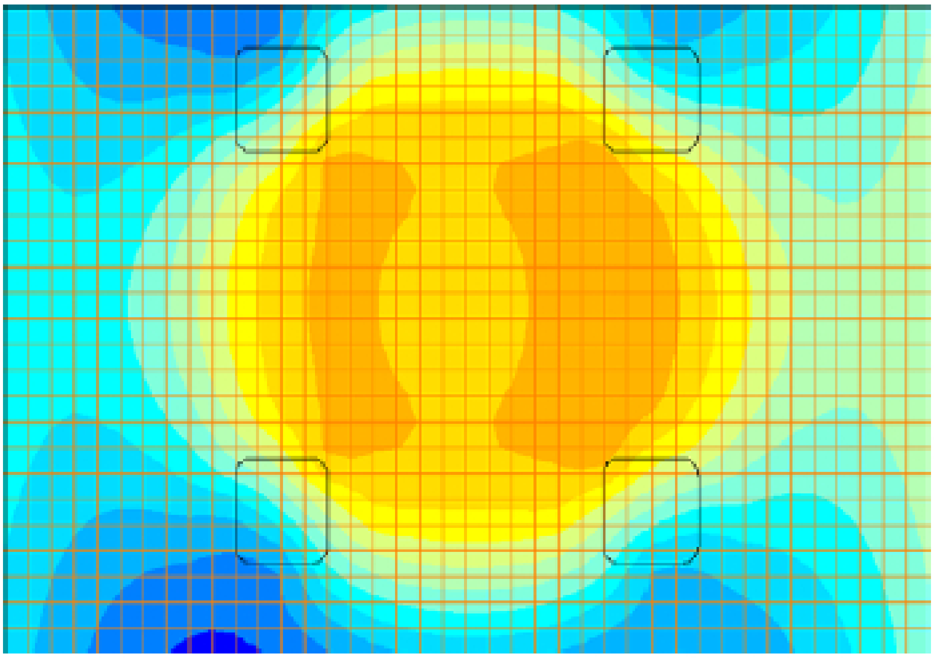
7.7 Model Test

Model test is the most direct method to predict air gap. However, model test results are usually not available in the early design stage. Even in the final design stage, model test is often conducted under limited environmental conditions. In general, model test and numerical analyses are not to replace, but are rather to complement each other. Model test data are good sources to calibrate and validate the numerical model, and in turn, numerical simulations can provide comprehensive analysis results under various environmental conditions.

9 Field Points for Air Gap Analysis *(1 October 2018)*

It is suggested that air gap analyses are performed at locations in a grid pattern around a semi-submersible. The extreme relative surface elevations at the grid points are preferably displayed with contour plots, as shown in 1/9 FIGURE 2 for an example.

FIGURE 2
Field Points in Grid Pattern



If contour plots are not available, the air gap analysis are suggested to be conducted at the following locations, as a minimum:

- Center point

- Extreme center points
- Quartering points
- Locations around column area

Additional locations may also need to be analyzed if the locations are at risk to be exceeded by crest elevation. Special attention should be paid to the up-wave columns, where the minimum air gap often occurs.

11 Scope and Overview of These Guidance Notes *(1 May 2020)*

These Guidance Notes provide the analysis procedures of air gap assessment for semi-submersibles based on either the linear analysis or the second order analysis. The theoretical background of these methods is introduced with critical parameters highlighted.

Guidance is provided on the definition of the horizontal wave impact load and the strength assessment methods for those local structural members on peripheral sides of the upper structure subject to horizontal wave impact loads. Guidance is also provided on the vertical wave impact analysis in frequency domain for semi-submersibles. The suggested method is based on the relative vertical velocities between the vessel motions and wave surface elevations using the linear diffraction-radiation analysis with consideration of wave asymmetry factors, the local wave crest enhancement, and wave run-up factors.

Section 2 describes the three-dimensional diffraction-radiation analysis used to calculate vessel motions. Both linear frequency domain and nonlinear time domain methods are addressed.

Section 3 discusses the free surface elevation induced by wave-structure interaction and addresses the wave asymmetry and wave run-up phenomenon.

Section 4 presents the linear air gap analysis procedures in frequency domain.

Section 5 demonstrates the second order air gap analysis procedures in time domain.

Section 6 presents the horizontal wave impact loads and local structural strength assessment methods for the peripheral sides of the upper structure of semi-submersibles when there are negative air gaps.

Section 7 presents the vertical wave impact loads and local structural strength assessment methods for the deck bottom or undersides of the overhanging platforms of semi-submersibles.

A diagram outlining the air gap assessment process in accordance with these Guidance Notes is given in 1/13 FIGURE 3.

13 Symbols *(1 May 2020)*

a: instantaneous air gap

a_0: still water air gap

$\widetilde{a}_0$: still water air gap reduced by the mean and low frequency heel motion components

η: free surface elevation

$\frac{d\eta}{dt}$: time derivative of free surface elevation, i.e. vertical free surface velocity

η_I: incident wave elevation

η_D: diffraction wave elevation

η_R: radiation wave elevation

η_{extre}: extreme value of the total free surface elevation

$\eta_{extre}^{(1)}$: extreme value of the first order free surface elevation

δ: vertical vessel motion

R, r: relative wave elevation

r_{in}, r_{out}: in-phase component and out-of-phase component of relative wave elevation

r_{Amp}, r_{phase}: amplitude and phase angle of relative wave elevation RAO

r_{max}: maximum relative wave elevation

r_{mean}: mean relative wave elevation

r_{WF}, r_{LF}: extreme value of wave frequency (WF) and low frequency (LF) relative wave elevation

γ: correlation factor between low frequency and wave frequency relative wave elevation

r_{extre}: most probable maximum extreme (MPME) value of relative wave elevation

t: time instant

X: vessel motion

$\dot{X}, \frac{dX}{dt}$: vessel velocity

$\ddot{X}$: vessel acceleration

X_{Amp}: amplitude of motion Response Amplitude Operator (RAO)

X_{in}, X_{out}: in-phase component and out-of-phase component of motion RAO

ω: circular wave frequency

F^a: active wave forces

N: the number of discrete regular wave trains of a wave spectrum

ρ: fluid density

M: mass matrix

M_A: added mass matrix

C: radiation damping matrix

K: hydrostatic stiffness matrix

υ: viscous damping matrix

A_{wp}: water-plane area

g: gravity acceleration

∇: displacement volume

GM_T: transverse metacentric height

GM_L: longitudinal metacentric height

(x, y, z): coordinates of a given point

(x_G, y_G, z_G): coordinate of center of gravity (COG)

ζ: critical damping ratio

T_n: natural period

β: wave heading angle

k: wave number

$H^{(2+)}$: second order sum-frequency Quadratic Transfer Function (QTF) (amplitude)

T_P: peak period

T_z: zero up-crossing period

H_s: significant wave height

ω_c: characteristic frequency of extreme waves

α: wave asymmetry factor

α_{run-up}: wave run-up factor

$S_\eta(\omega)$: wave spectrum

$S_r(\omega, \beta)$: relative wave elevation spectrum

$S_v(\omega, \beta)$: relative vertical velocity spectrum

m_n: n -th order spectral moment

σ: standard deviation

$\bar{n}$: number of positive maxima per second for a Gaussian process

ε: bandwidth parameter

ψ: location parameter of Gumbel fitting

κ: scale parameter of Gumbel fitting

Q: integrated probability of freeboard exceedance for all sea states

q: annual probability of freeboard exceedance

$n_{3h\_operating}$: number of 3-hour sea states per year with the semi-submersible in the operating condition

A_n: wave amplitude of the n-th Airy wave component

Res: first order response

$Res^{(2)}$: second order response

$Res^{(2)-}$: difference-frequency response

$Res^{(2)+}$: sum-frequency response

$R^{(2)-}$: amplitude of difference-frequency QTF

$R^{(2)+}$: amplitude of sum-frequency QTF

θ_n: phase angle of the n-th Airy wave component

$\theta^{(2)-}$: phase angle of difference-frequency QTF

$\theta^{(2)+}$: phase angle of sum-frequency QTF

p_{peak}: peak horizontal wave impact pressure

p_{v_peak}: peak vertical wave impact pressure

p_s: equivalent static horizontal wave impact pressure

p_{v_s}: equivalent static vertical wave impact pressure

f_a: design factor

f_e: elevation factor

f_s: shape and size factor

f_{s_v}: shape factor for vertical wave impact pressure

k_d: dynamic amplification factor

C_p: wave impact pressure coefficient

V_R, v: relative vertical velocity

v_{in}, v_{out}: in-phase component and out-of-phase component of relative vertical velocity RAO

v_{Amp}, v_{phase}: amplitude and phase angle of relative vertical velocity RAO

v_{WF}: extreme WF relative vertical velocity

v_{max}: maximum relative vertical velocity

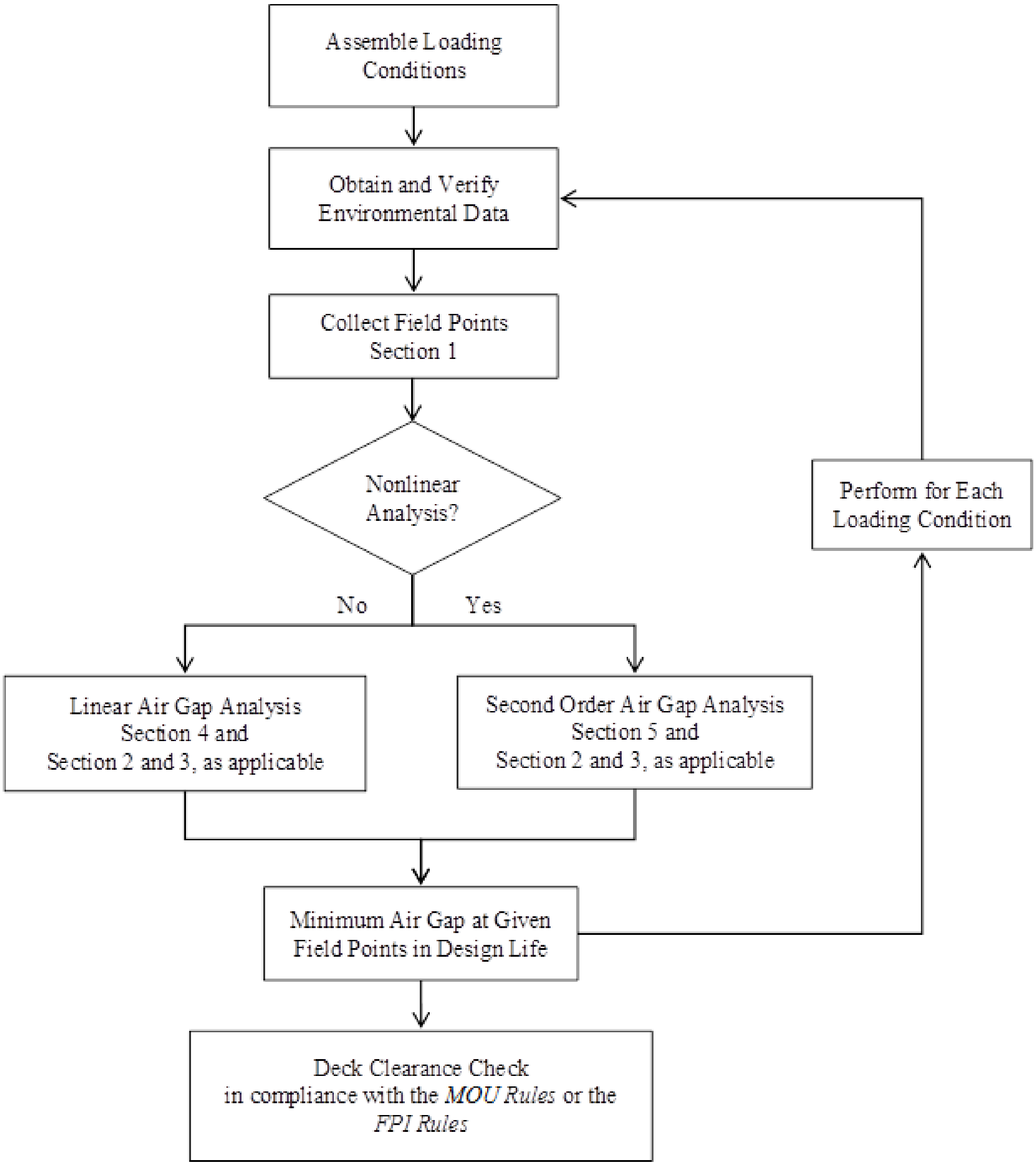
P: joint probability distribution of relative wave elevation and relative vertical velocity

σ_r: standard deviation of WF relative wave elevation

σ_v: standard deviation of WF relative vertical velocity

N_r: number of cycles of relative wave elevation in 3 hours

FIGURE 3
Overall Procedure for Air Gap Assessment *(1 May 2020)*



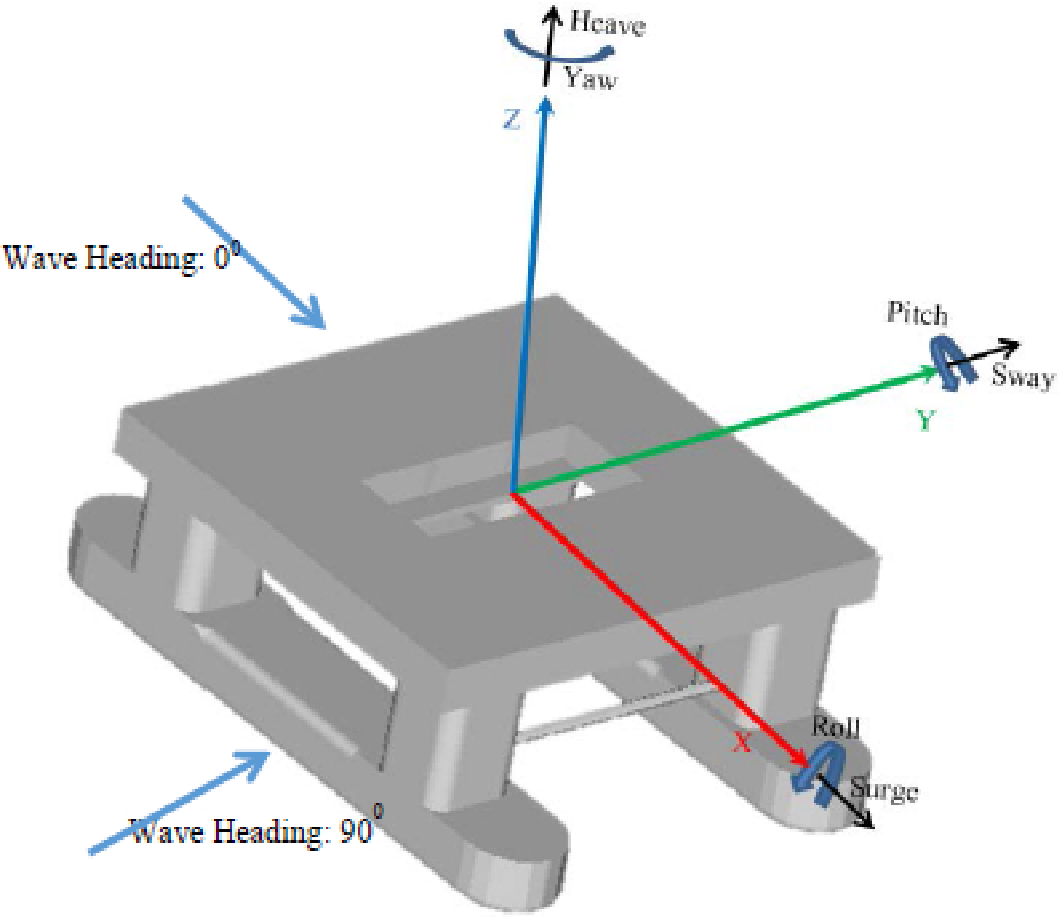
SECTION 2 Motion Responses of Semi-submersibles

1 Definition of Motions

These Guidance Notes, unless otherwise stated, utilize a right-handed Cartesian coordinate system, as shown in 2/1 FIGURE 1. The x-axis points forward; the y-axis points to port side; and the z-axis points vertically upwards. The origin is the intersection of midship and centerline in the mean free surface. The wave heading angle is measured counter-clockwise from the positive x-axis of the coordinate system. Following sea is defined as 0°, beam sea as 90°, and head sea as 180°. The wave reference point locates at the origin.

These Guidance Notes employ the three-dimensional diffraction-radiation analysis to calculate the motion responses to incident waves. A semi-submersible is regarded as a rigid body and the motions in six Degrees-Of-Freedom (DOF) are calculated in motion reference point, which is placed in the center of gravity (COG) of the rig. The definitions of the six motion modes, surge, sway, heave, roll, pitch, yaw, are shown in 2/1 FIGURE 1. Forward speed effects are not considered in these Guidance Notes.

FIGURE 1
Definition of Motions



The first order vessel response to unit amplitude, regular, sinusoidal wave is commonly referred to as Response Amplitude Operator (RAO), which correlates the first order vessel response amplitude with the incident wave amplitude. The corresponding second order item (i.e., Quadratic Transfer Function (QTF)) correlates the second order vessel response amplitude with the squared incident wave amplitude. Both RAO and QTF consist of two components: the in-phase component and the out-of-phase component, or alternatively, amplitude and phase angle. The time history of the first order responses and the second order responses to specified incident wave can be explicitly obtained from RAO and QTF, respectively.

3 Diffraction-Radiation Analysis

Large volume vessel responses (motions, wave loads, etc.) are typically computed using the three-dimensional diffraction-radiation analysis based on the potential flow theory, which assumes that vessel motions consist of small oscillations about an equilibrium position and that the fluid is incompressible and inviscid and the fluid flow is irrotational.

3.1 Panel Model Development

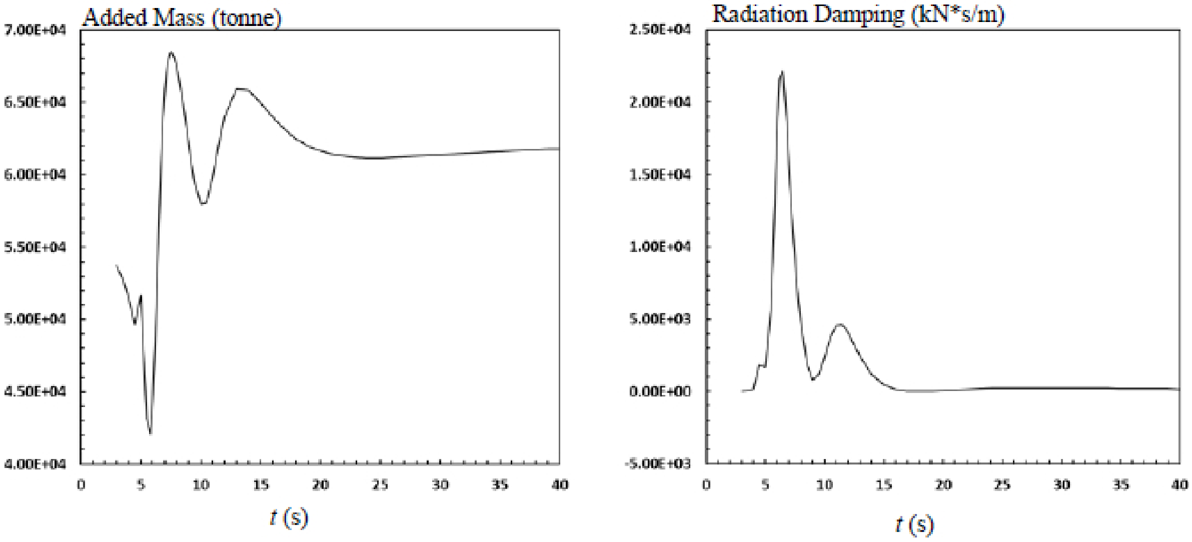
Three dimensional diffraction-radiation analysis requires the modelling of the mean wetted part of the hull surface with diffraction panels. Panel mesh should comply with modeling requirements, which depend on the software package in use. Below is a typical set of modeling requirements:

- The panel model should cover the entire mean wetted surface of the hull and should closely match the real structure.
- The panel mesh should be fine enough to resolve the radiation and diffraction waves with reasonable accuracy. A common practice is to keep the longest side of the panel mesh less than 1/7 of the smallest wave length analyzed.
- If the software in use does not allow the panels cutting through the still waterline, the diffracting panel mesh should be below the still waterline.
- The panel mesh normals should point the same direction, outward or inward depending on the software package used.
- In areas with abrupt changes in geometry, fine mesh should be applied.

3.3 First Order Hydrodynamic Load

Three dimensional diffraction-radiation analysis considers two categories of hydrodynamic load for large volume structures: active wave forces and wave radiation forces. The active wave forces consist of the incident wave force (Froude-Krylov force) and the diffraction force, which is induced by wave scattering when a vessel is considered restrained from motion. When no incident wave presents, the forced oscillation of a vessel results in the wave radiation forces. The wave radiation forces are represented by the added mass and radiation damping terms. Three dimensional potential flow theory is commonly used for solving the active wave forces, added mass and radiation damping.

FIGURE 2
Typical Added Mass and Radiation Damping in Heave Mode

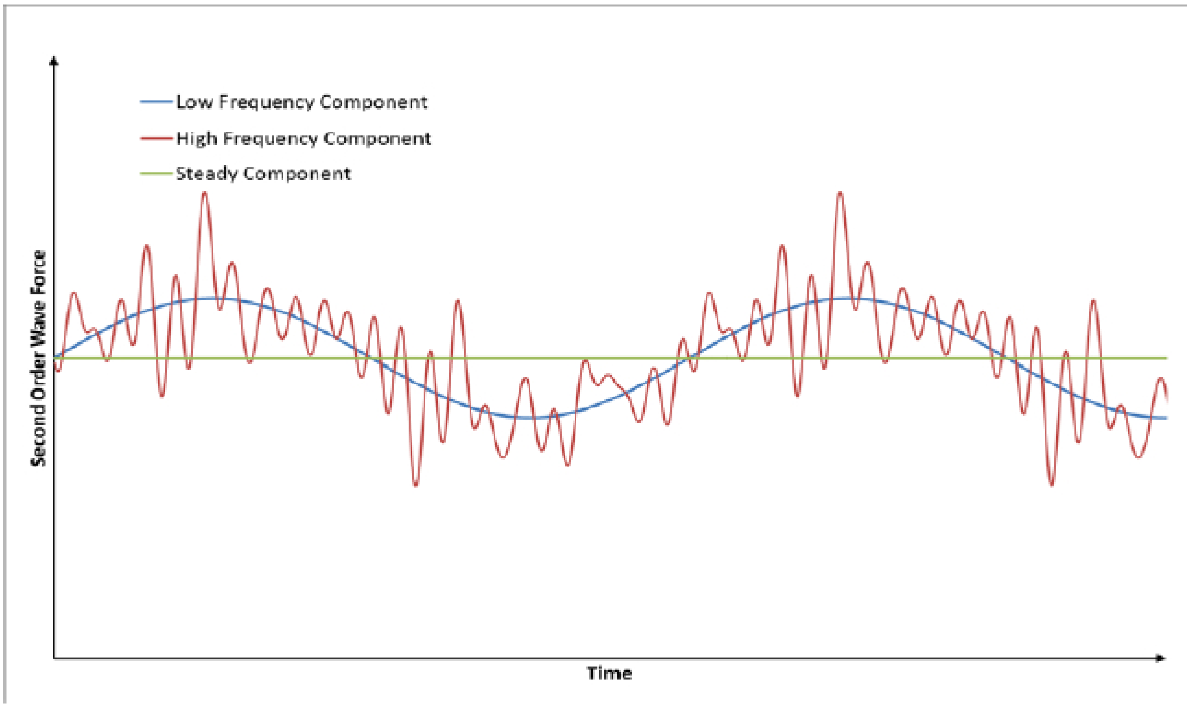


The active wave forces, added mass, and radiation damping, are independent of the mass distribution, hydrostatic stiffness and motion amplitudes of a vessel, but dependent on the wetted hull surface and wave conditions (wave frequency, water depth, fluid density, etc.). The heave mode added mass approaches constant as the wave period increases, while the heave mode radiation damping generally goes to zero for large wave periods (See 2/3.3 FIGURE 2).

3.5 Second Order Hydrodynamic Load

Compared with the first order hydrodynamic loads, the magnitudes of the second order hydrodynamic loads are smaller. They may however show prominent effects in the vicinity of resonance. The second order analysis produces the total second order wave forces by use of QTFs, consisting of three components (see 2/3.5 FIGURE 3):

FIGURE 3
Second Order Wave Force Components



3.5.1 Steady Component

The steady component, also designated as the mean wave drift force, is time-invariant. Vertical mean wave drift force may be important for air gap analysis of semi-submersibles. Both the far field solution (momentum approach) and near field solution (direct pressure integration approach) are applicable to calculate the mean wave drift forces (Faltinsen, 1990). The far field solution approach, however, can only estimate the mean wave drift forces in horizontal plane.

3.5.2 Low Frequency Component

The low frequency component of vessel response, also designated as the slowly varying wave drift force, oscillates at difference frequency. For horizontal motions, i.e. surge, sway and yaw motions of semi-submersibles, the natural periods for these modes are typically over 100 seconds, and one may take advantage of Newman's approximation (Newman, 1974) to calculate the slowly varying wave drift forces to reduce the computational time. This approach calculates the off-diagonal difference-frequency QTF values by averaging the corresponding diagonal values. For vertical motions (i.e., heave, roll and pitch motions of semi-submersibles) the natural periods are usually smaller than 60 seconds, the Newman's approximation may underestimate the slow drift motions in these modes. The solution of a full QTF matrix is recommended to be used, especially in shallow water conditions, under which the second order potential contribution increases significantly.

3.5.3 High Frequency Component

The high frequency component of vessel response, also designated as the wave springing force, oscillates at sum frequency. Wave springing forces may induce high frequency structural vibration, which is important for the fatigue performance of tension leg platforms (TLPs). However, these forces have negligible contributions to the motions of semi-submersibles.

5 First Order Wave Frequency Motion Analysis in Frequency Domain

For a steady state and linear system, the motion responses can be obtained directly in frequency domain by solving a set of linear equations, expressed as Eq. 2.1:

$$(M + M_A)\ddot{X} + (\upsilon + C)\dot{X} + KX = F^a \quad (2.1)$$

where,

F^a = active wave forces (per unit wave amplitude), refer to 2/3.3

M = 6 × 6 structural mass matrix

M_A = 6 × 6 added mass matrix, refer to 2/3.3

C = 6 × 6 radiation damping matrix, refer to 2/3.3

υ = 6 × 6 viscous damping matrix

K = 6 × 6 hydrostatic stiffness matrix

$\ddot{X}$ = accelerations (per unit wave amplitude) in six DOF

$\dot{X}$ = velocities (per unit wave amplitude) in six DOF

X = motions (per unit wave amplitude) in six DOF

5.1 Structural Mass Matrix

The diagonal elements of the structural mass matrix consist of the total mass of a vessel and the total mass moment of inertia for roll, pitch and yaw. The non-diagonal elements in the mass matrix are usually at least one order of magnitude smaller than the diagonal elements.

The total mass moment of inertia, usually identified as radius of gyration, describes the mass distribution of a vessel relative to a given axis. The mass moment of inertia relative to a given axis includes two items, local item and "parallel axis" item. The local items of large mass blocks should be considered. Otherwise, the accuracy of the motion responses will be degraded.

5.3 Hydrostatic Stiffness Matrix

The hydrostatic stiffness matrix, also called the hydrostatic restoring matrix, represents the reaction forces produced when the rigid body moves unit displacement from the initially equilibrated position in calm water.

For a freely floating structure, the hydrostatic stiffness matrix is only available for heave, roll and pitch. Some software packages are capable to generate the hydrostatic stiffness matrix based on the panel model and COG of a vessel. Alternatively, the heave, roll and pitch stiffness can be directly calculated by using water-plane area (A_{wp}) and transverse metacentric height (GM_T) or longitudinal metacentric height (GM_L). When partially filled tanks exist, $GM_L and GM_T$ should be corrected for free-surface effects.

If necessary, the stiffness matrix should take into account the additional stiffness due to mooring lines and risers.

5.5 Natural Periods

Relatively large motions are likely to occur if the wave period is close to a semi-submersible's resonance period. The natural periods of a semi-submersible are important parameters in assessing the motion characteristics of the structure. The undamped and uncoupled natural periods can be expressed as:

$$T_{ni} = 2\pi\left(\frac{M_{ii} + M_{Aii}}{K_{ii}}\right)^{1/2} = > \begin{cases} T_{n3} = 2\pi\left(\frac{M_{33} + M_{A33}}{\rho g A_{wp}}\right)^{1/2} \\ T_{n4} = 2\pi\left(\frac{M_{44} + M_{A44}}{\rho g \nabla GM_T}\right)^{1/2} \\ T_{n5} = 2\pi\left(\frac{M_{55} + M_{A55}}{\rho g \nabla GM_L}\right)^{1/2} \end{cases} \quad (2.2)$$

where

T_{ni} = natural period of heave (i = 3), roll (i = 4) and pitch (i = 5)

M_{ii} = corresponding diagonal elements in the mass matrix

M_{Aii} = corresponding diagonal elements in the added mass matrix

K_{ii} = corresponding diagonal elements in the hydrostatic stiffness matrix

GM_T = transverse metacentric height

GM_L = longitudinal metacentric height

A_{wp} = vessel's water-plane area

∇ = vessel's displacement volume

ρ = fluid density

g = gravity acceleration

The typical natural periods of six DOF for moored semi-submersibles in deep water are listed in 2/5.5 TABLE 1.

TABLE 1
Typical Natural Periods for Deep Water Semi-submersibles

Motion Modes	*Surge*	*Sway*	*Heave*	*Roll*	*Pitch*	*Yaw*
Natural Periods (s)	>100	>100	20-30	30-60	30-60	>100

5.7 Viscous Loads *(1 October 2018)*

The viscous loads induce not only viscous damping but also viscous wave exciting force. While in general the viscous wave exciting force is small, it cannot be ignored around the cancelation period of the heave motion. A Morison element model with linearization can be used to derive both the viscous damping and the viscous wave exciting force.

Due to the small water-plane area, the radiation damping of semi-submersibles is only a small portion of the total damping. The effect of viscous damping becomes prominent in the vicinity of the natural period. In particular, the heave motion amplitudes will be unrealistically large in the vicinity of resonance period if the viscous damping effect is neglected, as shown in 2/5.7 FIGURE 4.

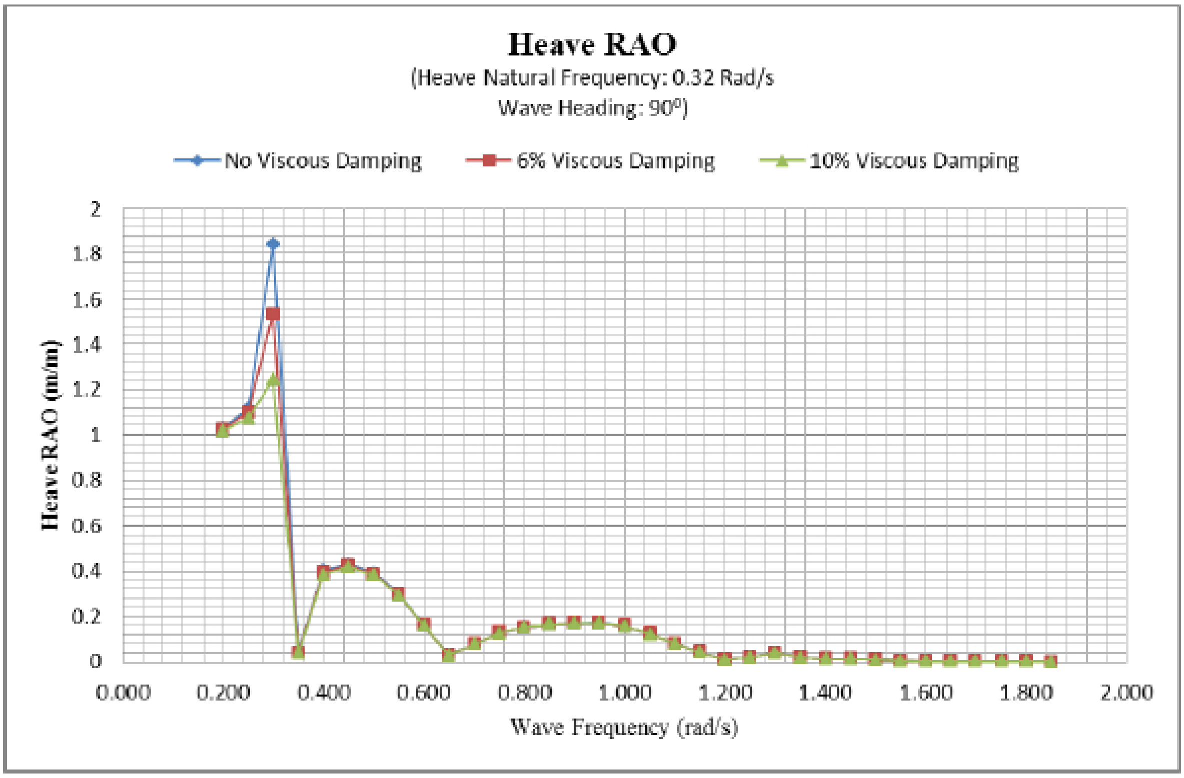
Consequently, motion analysis of semi-submersibles based on potential flow theory entails a proper heave viscous damping model. The nonlinear characteristics of the viscous damping make the numerical prediction very complex. Generally, the viscous damping value is determined through model tests and is typically expressed in the form of critical damping ratio. If model test results are not available, critical damping ratio may be assigned according to previous experience with similar designs. The equivalent viscous damping, υ, can be calculated based on Eq. 2.3.

$$\upsilon_{ii} = 2 \cdot \zeta_{ii} \cdot \sqrt{(M_{ii} + M_{Aii}) \cdot K_{ii}} \quad (i = 3, 4, 5) \quad (2.3)$$

where, ζ is the critical damping ratio and may be determined via free decay test, while M, M_A, K are corresponding mass, added mass and stiffness for heave, roll and pitch, respectively.

Damping used in the analysis may be calibrated using model test results if available. It is however noted that the damping in a storm sea state can be significantly higher than that in a free decay test in still waters. In contrast to adjusting the damping matrix, viscous damping could be more accurately simulated using Morison members. A calibrated drag coefficient is necessary to calculate the viscous drag term. Linearization of the viscous drag forces can be performed for each sea state.

FIGURE 4
Typical Heave RAO (Amplitude) under Different Levels of Heave Viscous Damping



5.9 Motion RAO

Solving the equations of motion in frequency domain produces the motion RAOs corresponding to each pair of wave frequency-heading combination. The time history of motion response corresponding to given unit incident wave can be expressed by using motion RAOs:

$$X(x_G, y_G, t) = X_{Amp}(\omega, \beta) \cdot cos[\omega t - kx_G\cos\beta - ky_G\sin\beta - \theta(\omega, \beta)] \quad (2.4)$$

where,

$X(x_G, y_G, t)$ = vessel motions at COG at instant t

x_G, y_G = coordinate of COG in x-axis and y-axis, respectively

$X_{Amp}(\omega, \beta)$ = amplitude of the motion RAO at COG

$\theta(\omega, \beta)$ = phase angle of the motion RAO at COG

β = wave heading angle

k = wave number

ω = circular wave frequency

Alternatively, RAOs can also be described with the in-phase component (X_{in}) and the out-of-phase component (X_{out}). 2/5.9 FIGURE 5 is a schematic drawing that illustrates the RAO components. The in-phase component represents the response at the moment when wave crest reaches the wave reference point while the out-of-phase component represents the response at the time instant $^1/_4$ wave period after the in-phase component. The relationships of these components are as follows:

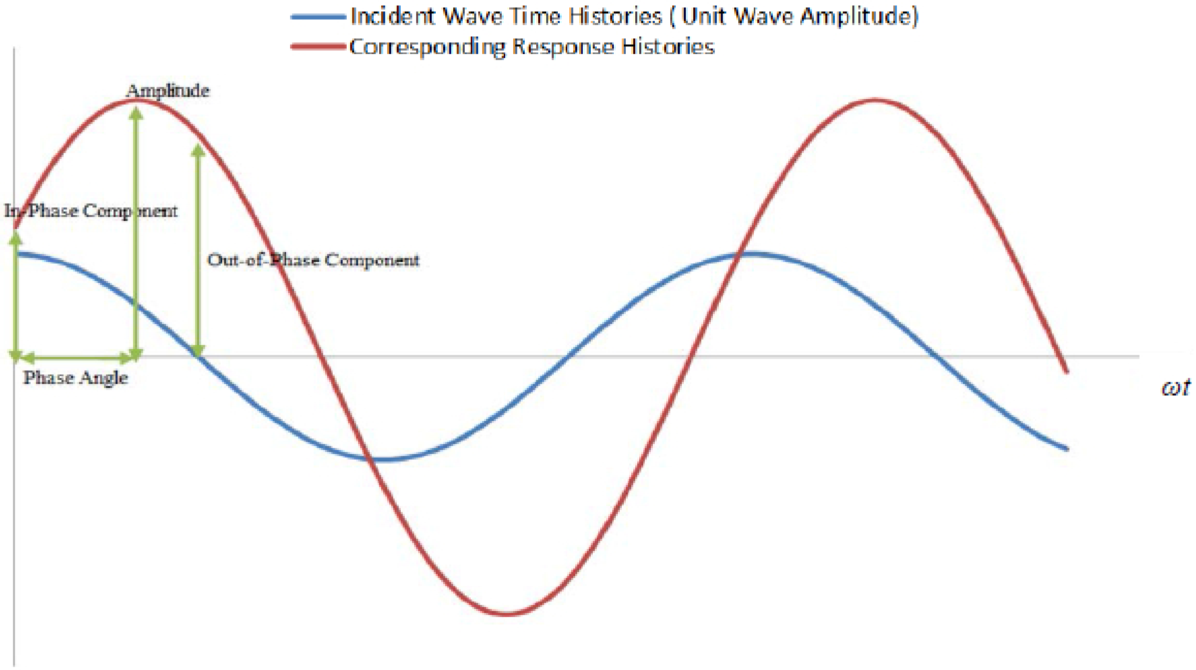
In-Phase: $X_{in} = X_{Amp} \cdot \cos(\theta)$ (2.5)

Out-of-Phase: $X_{out} = X_{Amp} \cdot \sin(\theta)$ (2.6)

Amplitude: $X_{Amp} = \sqrt{(X_{in})^2 + (X_{out})^2}$ (2.7)

Phase Angle: $\theta = \arctan(X_{out}/X_{in})$ (2.8)

FIGURE 5
RAO Components



7 Second Order Low Frequency Motion Analysis in Frequency Domain

The linear air gap analysis can be augmented with corrections considering the mean and significant low frequency vertical vessel motions. The mean and low frequency motions may be estimated by a simplified analysis in frequency domain using the spectral analysis techniques in conjunction with the difference-frequency QTFs of slowly varying drift forces and the damping of low frequency motions.

Wind induced vessel roll and pitch motions may be calculated based on one of the following two methods:

i) Wind is modeled as a steady 1-minute average speed in a given direction;

ii) Wind is modeled as a steady 1-hour average speed in combination with the wind speed fluctuation modeled by a wind spectrum in a given direction.

Wind induced motions can then be calculated in frequency domain using the spectral analysis techniques.

The wave and wind induced low frequency motions may be combined by assuming independence of the two motions. Correlations of the motions may be considered provided the correlation factor of the motions is validated through reliable sources, such as model tests.

9 Motion Analysis in Time Domain

For a freely floating semi-submersible, the vertical slowly varying wave exciting force is typically small and, thus, the linear diffraction-radiation analysis often suffices to predict motions in vertical plane. However, the nonlinear effects on the motions can be significant. For a moored semi-submersible in deep water, in particular ultra-deep water, the floater, risers and mooring system comprise an integrated dynamic system responding to environmental loads in a complex way. The nonlinear effects, particularly contributed by mooring lines and risers, could become prominent. The mean and low frequency motion responses may be considerably large. The coupled motion analysis in time domain, which allows considering nonlinear forces and stiffness functions, is recommended.

The time-domain dynamic motion analysis requires initially calculating the hydrodynamic coefficients [e.g., added mass, radiation damping, Froude-Krylov forces, diffraction forces, mean wave drift forces (see 2/3.5)], and the difference-frequency QTFs of slowly varying drift forces (see 2/3.5), based on frequency domain diffraction-radiation analysis. Since the RAO-based radiation force calculation may no longer be accurate due to nonlinear motions involve, convolution integral is recommended to calculate the radiation forces in time domain. The coupled motion equations of the hull and dynamic equations for mooring lines and, if necessary, risers, are solved simultaneously at each time step by taking into account the wave, wind and current loads acting on the hull plus the drag and inertia loads on mooring lines. The accuracy of the time domain analysis entails a carefully selected time step by a convergence check.

11 Motion Analysis Using the Combined Time Domain and Frequency Domain Approach

To reduce computational complexity, a hybrid approach is often applied to calculate only the mean and low frequency motion responses in time domain, while wave frequency vessel motions are calculated separately in the frequency domain and then combined with the mean and low frequency motions. Unlike the fully coupled time domain analysis, the low frequency damping from the vessel, mooring lines and risers should be evaluated separately and treated as an input parameter for the hybrid approach.

13 Vertical Motions at Given Points

The air gap assessment entails the vertical motions at field points. Based on the linear analysis in frequency domain or nonlinear analysis in time domain described in this section, the motions in six DOF with respect to the reference point (COG in these Guidance Notes) can be obtained. The linearized expression of vertical motion of any point on the structure can be written as,

$$\delta(x, y, z, t) = X_3(t) + (y - y_G) \cdot \sin(X_4(t)) - (x - x_G) \cdot \sin(X_5(t)) \quad (2.9)$$

where

$\delta(x, y, z, t)$ = vertical motion of given point on the structure

(x, y, z) = coordinates of the given point

x_G, y_G = coordinate of COG in x-axis and y-axis, respectively

t = time instant

$X_3(t)$ = heave motion at COG

$X_4(t)$ = roll motion at COG

$X_5(t)$ = pitch motion at COG

15 Motion Characteristics of Semi-submersibles

Semi-submersibles have small water-plane areas, which effectively reduce motion amplitudes driven by waves, especially during swell seas and storms. The favorable motion characteristic is one of the main features of semi-submersibles.

The stiffness of the horizontal plane motions (surge, sway, and yaw) are very soft, so that the natural frequencies in surge, sway and yaw are usually low and far away from the peak frequency of the wave spectrum. Thus, the wave frequency motions of semi-submersibles in the horizontal plane are considerably low. However, slowly varying wave drift loads give rise to low-frequency resonant horizontal motions. The horizontal plane motions are important considerations for the design of mooring systems and dynamic positioning systems but do not interfere with the air gap design. These Guidance Notes will not further discuss the horizontal plane motions.

The natural period of heave motion of a typical semi-submersible could be within or close to the wave frequency range, which may cause significant wave frequency heave motion. On the other hand, the natural periods of roll and pitch are usually beyond the wave frequency range occurring in the sea and therefore the wave frequency roll and pitch motions are moderate. Due to the small water-plane area of semi-submersibles, the mean and low frequency heave, roll and pitch motion responses may become significant. It is worth paying attention to the prediction of mean and low frequency motions when the margin of air gap becomes critical for a semi-submersible.

SECTION 3 Free Surface Elevation

1 Linear Free Surface Elevation

The free surface elevation at a given point includes three components,

$$\eta = \eta_I + \eta_D + \eta_R \quad (3.1)$$

where

η = total free surface elevation

η_I = incident wave elevation

η_D = diffraction wave elevation

η_R = radiation wave elevation

The radiation wave elevation, dependent on the motion amplitude, is negligible in short waves ($\omega > 1.2$ rad/s) since the corresponding heave motion amplitudes of the semi-submersible are typically very small at these wave periods. Meanwhile, viscous damping affects the radiation wave elevation in the vicinity of the resonant frequency.

3 Wave Asymmetry Factor *(1 October 2018)*

Ocean waves are inherently asymmetric (i.e., higher crests with shallower troughs) and cannot be described by the linear diffraction-radiation theory. Thus, the linear air gap analysis entails a wave asymmetry factor (α_{wa}) to account for the asymmetry of crests and troughs. The preferred approach to obtain the wave asymmetry factor is to derive the value from model tests and compare the results to the field data. If model test data is not available, these Guidance Notes recommend the following closed-form expression to estimate the wave asymmetry factor:

$$\alpha_{wa} = \eta_{extre}/\eta_{extre}^{(1)} \quad (3.2)$$

$$\begin{aligned}\eta_{extre} &= \eta_{extre}^{(1)} + H^{(2+)}\left(\eta_{extre}^{(1)}\right)^2 \\ &\approx \eta_{extre}^{(1)} + \frac{k}{2}\left(\eta_{extre}^{(1)}\right)^2 \quad (3.3)\end{aligned}$$

where

η_{extre} = extreme value of the total free surface elevation

$\eta_{extre}^{(1)}$ = extreme value of the first order free surface elevation

$H^{(2+)}$ = second order sum-frequency QTF (amplitude) of incident wave

k = wave number

Following the second order Stokes model in infinite water depth, the sum-frequency QTF of incident wave elevation can be expressed as:

$H^{(2+)} = k/2 \quad (3.4)$

The wave number, $k = \omega_c^2/g$, should be based on a wave characteristic frequency of extreme waves. ω_c can be estimated as $\omega_c = 2\ \pi\ /(0.92\ T_p)$ (Sweetman, 2001), where T_p is peak period of the sea state. The extreme value of the first order free surface elevation, $\eta_{extre}^{(1)}$, can be determined using the method similar to the one described in 4/5 for calculating the linear extreme relative free surface elevation.

Note that Eq. 3.2 yields a location-dependent wave asymmetry factor because the diffraction/radiation effects are implicitly considered. A higher wave asymmetry factor is often found at the field points close to the column, where the minimum air gap is likely to occur. In general, the wave asymmetry factor increases as the wave steepness increases.

5 Quadratic Transfer Function of Free Surface Elevation

A second order analysis may be used to predict the free surface elevations. Unlike the linear diffraction-radiation analysis, which only requires the panel model of a wetted body surface, a discretization of the mean free surface is also needed to solve the second order problem. The linear RAOs and difference-frequency QTFs of free surface elevation are easy to converge with a relatively coarse mesh. However, this is not true for the calculation of sum-frequency QTFs. The sum-frequency QTF is associated with much shorter waves and the results are very sensitive to the discretization of the wetted hull body surface and the free surface. A convergence check should be performed. In general, the second order method for the free surface elevation analysis should be validated by model tests.

7 Wave Run-up Factor *(1 May 2020)*

At near-column locations, the run-up effect should be suitably modeled. Wave run-up is a highly nonlinear local phenomenon, sometimes accompanied by wave breaking. It is recommended to consider a wave run-up factor, α_{run-up}, for near-column locations if the three-dimensional diffraction-radiation analysis is employed.

Model test is a preferred way to obtain the wave run-up factor. CFD-based methods provide an alternative approach to determine the wave run-up factor. There are also empirical formulas available for wave run-up calculation. See, for instance, Stansberg, et al. (2010). However, engineering judgment and validation are important for the application of CFD analysis or empirical formulas for the wave run up calculation.

Wave run-up effects should be considered for the vertical wave impact analysis (see Section 7). The consideration of wave run-up effects for vertical wave impact analysis are discussed in 7/3.7.

SECTION 4 Linear Air Gap Analysis

1 Relative Wave Elevation RAO (*1 May 2020*)

As illustrated in Section 1, the relative wave elevation (r) equals the free surface elevation (η) subtract the corresponding vertical motion of the vessel (δ) at a field point. Because of the phase difference between these two terms (see 4/1 FIGURE 1), the calculation should be carried out in terms of the in- phase components (η_{in} and δ_{in}) and the out-of-phase components (η_{out} and δ_{out}), instead of the amplitudes. Meanwhile, the wave asymmetry factor (α_{wa}) and the wave run-up factor (α_{run-up}) should be taken into account. The relative wave elevation RAO, thus, can be computed based on Eq. 4.1 ~ Eq. 4.4.

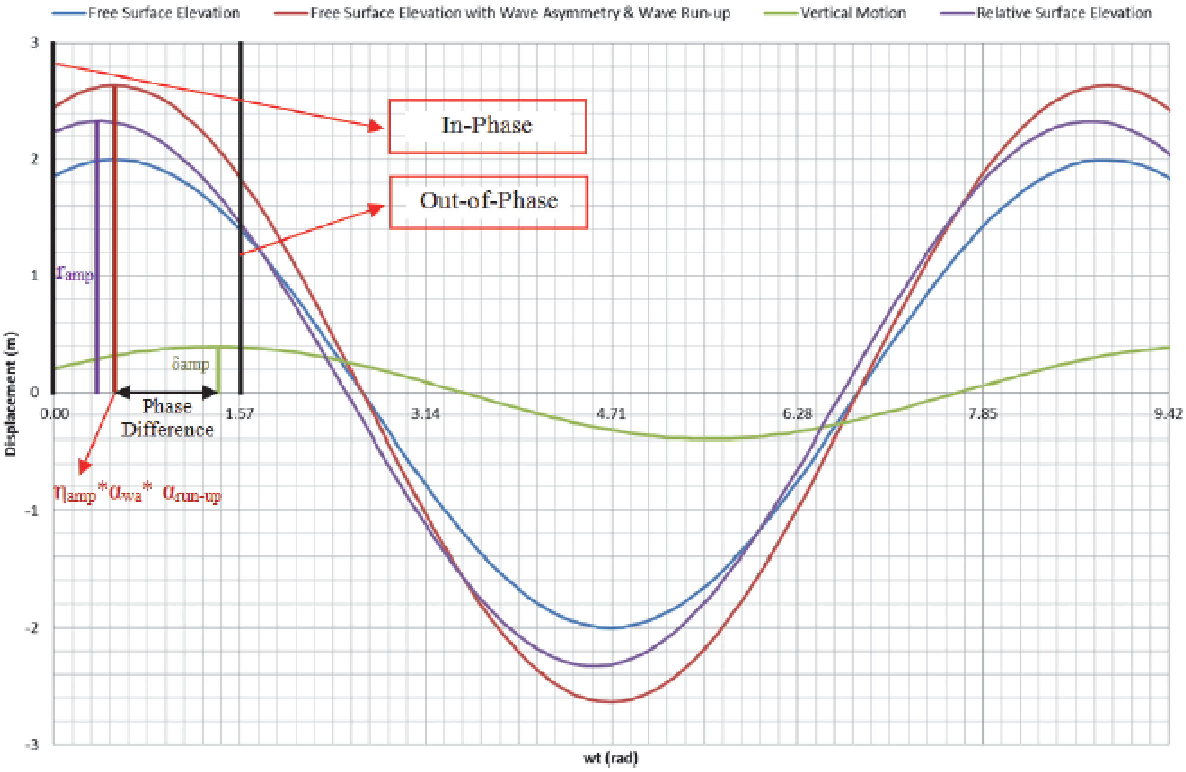
In-Phase: $r_{in}(x,y,z) = \alpha_{wa} \cdot \alpha_{run-up} \cdot \eta_{in}(x,y,z) - \delta_{in}(x,y,z)$ (4.1)

Out-of-Phase: $r_{out}(x,y,z) = \alpha_{wa} \cdot \alpha_{run-up} \cdot \eta_{out}(x,y,z) - \delta_{out}(x,y,z)$ (4.2)

Amplitude: $r_{Amp} = \sqrt{(r_{in})^2 + (r_{out})^2}$ (4.3)

Phase Angle: $r_{phase} = \arctan(r_{out}/r_{in})$ (4.4)

FIGURE 1
Relative Wave Elevation Time Histories (*1 May 2020*)



When there is forward speed, such as the transit condition, the effects of forward speed should be considered.

Hydrodynamic analysis with forward speed should be performed to derive the free surface elevation and relative wave elevation.

3 Environmental Conditions

3.1 Wave Spectrum

Ocean waves in the real world are irregular and display random behavior. Wave spectrum provides a mathematical model of the characteristics of an ocean wave system. The following spectra are among those often used in the design of semi-submersibles:

- Bretschneider spectrum or two-parameter Pierson-Moskowitz spectrum is suitable for open-sea wave conditions (e.g., the Atlantic Ocean).
- JONSWAP spectrum is derived from the Joint North Sea Wave Project (JONSWAP) and constitutes a modification to the Pierson-Moskowitz spectrum to account for the regions that have geographical boundaries limiting the fetch in the wave generating area (e.g., the North Sea).
- A two-peak spectrum, such as the Ochi-Hubble spectrum, is usually used to account for both wind sea and swell.

3.3 Sea State

A slowly varying local sea state can be assumed stationary in a short time interval (for instance, in a three-hour duration). A sea state is usually described by a wave spectrum with significant wave height (H_s) and a characteristic period, such as peak period (T_p) or zero up-crossing period (T_z). The statistical value based on a single sea state is referred to as short-term statistics.

3.5 Wave Scatter Diagram and Rosette

Long-term descriptions of the wave environment in the form of wave scatter diagram and rosette are required for long-term extreme value analysis. A wave scatter diagram recorded at a certain location over a long period (years) of time provides the probability or number of occurrence of sea states in a specified ocean area. A wave rosette describes the occurrence probability of each heading angle at a site. In case the wave rosette is not available, omni-directional sea states may be assumed in open ocean conditions. When performing the long-term extreme value analysis, estimation of damping and wave asymmetry factors used in the short-term analysis may also be applied in each individual sea state in the long-term extreme value analysis.

3.7 Wave Spreading *(1 October 2018)*

Ocean waves travel in many directions. Therefore, the combined wave system is normally short-crested. The spreading of wave directions, if applicable, may be taken into account to describe the short-crested aspect. Refer to ISO 19901-1 for further guidance.

3.9 Wind and Current

Wind and current are not considered when calculating the first order motions, but should be taken into account when calculating the mean and low frequency vessel motions.

5 Extreme Relative Wave Elevation *(1 May 2020)*

The linear air gap analysis relies on extreme response analysis to determine the extreme relative wave elevation corresponding to the minimum air gap of a semi-submersible. The statistical methods, including the short-term method and the long-term method, are applicable to predict the extreme responses when the wave enviroment is described in a spectral form.

5.1 Short-term Extreme Value Analysis *(1 May 2020)*

Short-term extreme value analysis is commonly used for the air gap analysis. The procedure described in this Subsection calculates the most probable maximum extreme value of the relative wave elevation for a given field point. Depending upon the local design and regulatory requirements, other extreme value analysis methods for predicting the extreme value with higher non-exceedance probability may also be used.

i) For each sea state represented by H_s and T_z (or T_p), a relative wave elevation spectrum, $S_r(\omega,\beta)$, should be computed from Eq. 4.5 based on the wave spectrum, $S_\eta(\omega)$, and the relative wave elevation amplitude, $r_{Amp}(\omega,\beta)$, for each wave heading, β.

$$S_r\big(\omega,\beta\ \big) = S_\eta\big(\omega\ \big)|\ r_{Amp}\big(\omega,\beta\ \big)|^2 \quad (4.5)$$

ii) The spectral moment of the relative wave elevation spectrum equals,

$$m_n = \int_0^\infty \omega^n S_r(\omega,\beta)dw \quad (4.6)$$

where m_n is the n-th order spectral moment. The standard deviation $\sigma = \sqrt{m_0}$.

iii) Assume that the relative wave elevation is a Gaussian stochastic process with zero mean, the peaks of the response follow Rayleigh distribution. Thus, the wave frequency extreme relative wave elevation, r_{WF}, to occur in 3 hours becomes,

$$r_{WF} = \sqrt{2\cdot\ln(3\cdot 3600\cdot\bar{n})}\sqrt{m_0} \quad (4.7)$$

$\bar{n}$ is the number of positive maxima per second for a Gaussian process,

$$\bar{n} = \frac{1}{2\pi}\sqrt{\frac{m_2}{m_0}} \quad (4.8)$$

iv) Where applicable, mean and low frequency motions should be considered in accordance with 4/5.5.

v) Select the maximum relative wave elevation among the wave headings considered for each sea state. Finally, select the maximum relative wave elevation among all the sea states considered. The corresponding minimum air gap is the target output.

5.3 Long-term Extreme Value Analysis *(1 May 2020)*

For semi-submersibles operating as floating production installations, the long-term statistical approach can also be used for the air gap analysis. The supplementary use of a short-term statistical approach to consider the design storm event is recommended. The larger of the two values obtained from the long-term statistical approach and the short-term statistical approach should be taken as the extreme response. Mean and low frequency motions should be considered in the air gap analysis.

The procedures for the long-term extreme value analysis are described in 4/7 of the ABS *Guide for 'Dynamic Loading Approach' for Floating Production, Storage and Offloading (FPSO) Installations*.Alternatively, the contour line approach may be used. For a given return period, the environmental contour line can be derived. Several sea states along the lines should be selected for the air gap analysis with the consideration that *i*) a sea state with the highest wave may not be the most critical sea state; and *ii*) a sea state with a smaller wave height but higher wave steepness may be more critical for the air gap analysis.

For semi-submersibles operating as mobile offshore units, depending upon the local design and regulatory requirements, long-term analysis methods for air gap analysis may also be used based on the annual probability of freeboard exceedance. When the long-term statistical approach is used for the air gap analysis to derive maximum relative wave elevation for mobile offshore units, the supplementary use of a short-term statistical approach is recommended. The larger of the two values obtained from the long-term statistical approach and the short-term statistical approach should be taken as the extreme response.

The long-term analysis may be used for derivation of the limit environmental condition for mobile offshore units as presented in 4/9.3.

5.5 Consideration of Mean and Low Frequency Vertical Motions *(1 May 2020)*

The maximum relative wave elevation, r_{max}, at a given field point can be separated into three components: the mean relative wave elevation (r_{mean}), the extreme wave frequency (r_{WF}) relative wave elevations, and the extreme low frequency (r_{LF}) relative wave elevations. When the mean and low frequency vertical motions are significant, the total maximum relative wave elevation can be calculated by:

$$r_{max} = r_{mean} + \sqrt{r_{WF}^2 + r_{LF}^2} \quad (4.9)$$

The above combination of the extreme wave frequency and low frequency relative wave elevations is based on the assumption that the two components are independent from each other. Correlations of the wave frequency and low frequency components may be considered provided the correlation factor of two components are validated through reliable sources, such as model tests. The total maximum relative wave elevation with consideration of the correlations by correlation factor (γ) between low frequency and wave frequency relative wave elevation can be calculated by:

$$r_{max} = r_{mean} + \sqrt{r_{WF}^2 + r_{LF}^2 + 2\gamma r_{WF}^2 r_{LF}^2} \quad (4.10)$$

In the absence of detailed mean and low frequency motion analysis or model tests for semi-submersibles operated as mobile offshore units, the extreme low frequency motion may be taken as a 5 degree heel amplitude applied in the same direction as the extreme wave frequency heel motion (roll, pitch or the combination of the two).

For semi-submersibles operated as floating production installations, the mean and low frequency motion should in general be determined through the motion analysis as recommended in 2/7, 2/9 and 2/11 or model tests.

Depending on the effectiveness of ballasting operation, the mean relative wave elevation, r_{mean}, may need to take into account the variation of static trim in the operating condition for which the air gap is assessed. In the absence of detailed analysis or model tests, a One (1) degree static trim may be applied in any possible directions.

Mean heel motion due to wind and wave loads is usually based on the consideration of ballasting to even keel.

The mean relative wave elevation, r_{mean}, may need to take into account the variation of static trim in the operating condition for which the air gap is assessed. In absence of the documented accuracy in ballasting to even keel, a one (1) degree static trim may be applied in any possible directions.

When the accuracy of ballasting is documented in the operations manual, the static trim can be derived based on the accuracy in the measurement of roll and pitch motions.

7 Summary of Linear Air Gap Analysis Procedures

7.1 General *(1 May 2020)*

The general procedure outlined below is recommended for the air gap analysis of semi-submersibles. Where the nonlinear wave/structure interaction effects are insignificant, the air gap analysis carried out in accordance with this procedure can normally give reasonable results.

7.3 Key Steps for Linear Air Gap Analysis *(1 May 2020)*

i) Assemble loading conditions and collect environmental wave data (See the *MOU Rules* or the *FPI Rules*).

ii) Define field points for air gap analysis.

iii) Collect the following parameters:

- Geometry of the hull surface
- Viscous damping/critical viscous damping ratio
- Floating position of the vessel (draft, trim, etc.)
- Weight distribution, total mass and COG of the vessel
- Radius of gyration for roll, pitch and yaw of the vessel
- Waterplane area, GM_L and GM_T, not compulsory if the software package is capable to calculate these parameters from the hydrodynamic model and COG
- Additional stiffness due to mooring lines and risers if applicable

iv) Develop the hydrodynamic model for linear diffraction-radiation analysis. The guidance on modeling is provided in 2/3.1.

v) Before performing the diffraction-radiation analysis, the numerical model should be statically balanced. The displacement, trim and draft, COG, Center of Buoyancy (COB), GM_L and GM_T should be checked. The still-water air gap can be determined using the statically balanced model.

vi) Perform the linear diffraction-radiation analysis to obtain the motion RAOs at COG and free surface elevation RAOs at field points for each combination of wave heading and wave frequency.

vii) Calculate the vertical motion RAOs at field points for each combination of wave heading and wave frequency (See 2/5).

viii) Collect wave asymmetry factor and wave run-up factor from model test results. If model test is not available, the wave asymmetry factor and wave run-up factor can be estimated in accordance with 3/3 and 3/7, respectively.

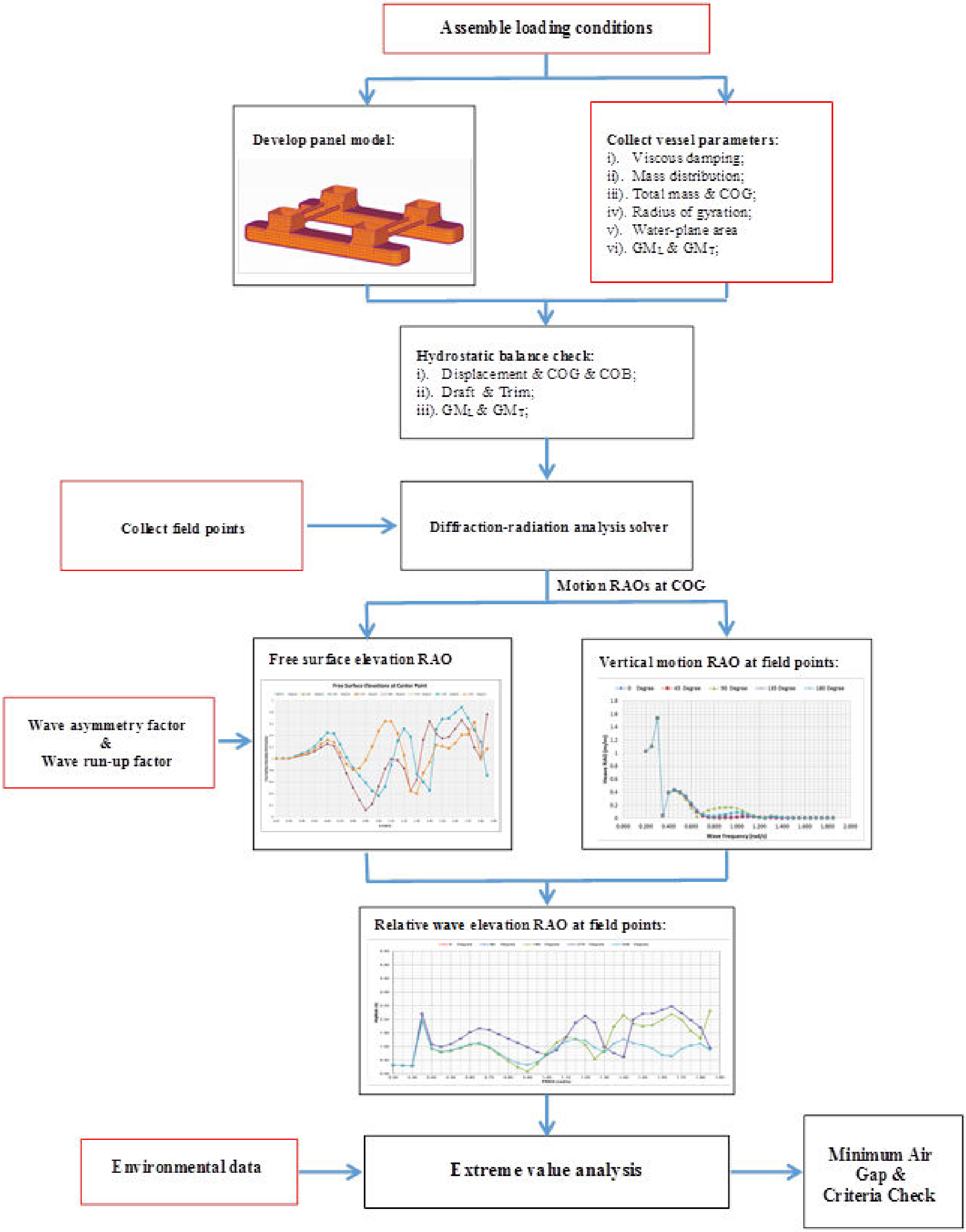
ix) Calculate the relative wave elevation RAOs at field points for each combination of wave heading and wave frequency with the wave asymmetry and wave run-up taken into consideration (See 4/1).

x) Calculate the extreme relative wave elevations at field points using an appropriate statistical approach (See 4/5). The mean and low frequency motions should be considered (See 4/5.5).

xi) The minimum air gaps at field points can be obtained by subtracting the maximum relative wave elevations from still-water air gap.

4/7.3 FIGURE 2 provides a flowchart of the linear air gap analysis procedures as described above.

FIGURE 2
Flowchart of Linear Air Gap Analysis *(1 May 2020)*



9 Limiting Sea States for Operating Condition *(1 May 2020)*

The limiting environmental condition for semi-submersibles in the operating condition is usually given in terms of the limiting sea states, which are represented by either significant wave heights (H_s) as a function of peak period $\left(T_p\right)$ or a single significant wave height (H_s). The limiting sea states can also depend on wave directions.

This Subsection describes both the short-term and long-term approaches for calculating the limiting sea states that enable a semi-submersible to maintain the non-negative air gap.

9.1 Short-term Approach

When deriving limiting sea states for non-negative air gap in the normal operating condition using the short-term approach, limiting sea states can be first selected along the environmental contour lines, then an iteration of the air gap analysis can be applied to identify the limiting significant wave height (H_s) for each peak period $\left(T_p\right)$ that results in non-negative air gap.

Mean and low frequency motions should be considered in the air gap analysis following 4/5.5.

The same procedure can be used to derive limiting sea states for different wave directions. Directional environmental contour line can be used when deriving directional limiting sea states.

9.3 Long-term Approach

When deriving limiting sea states for non-negative air gap in the normal operating condition using the long-term approach, the limiting sea states can be derived first based on the short-term approach according to 4/9.1. The probability of exceedance of a specified relative wave elevation in the operating condition can then be calculated from a long term analysis integrating over all sea states below the limiting sea states where the semi-submersible is at operating draft. Where there are multiple operating drafts, the same procedure can be applied for each draft.

When the semi-submersible operated with heading restrictions, wave scatter diagram together with its associated directional probability distribution (wave rosette) should be used.

When the semi-submersible operated with no heading restrictions, an omni-directional wave scatter diagram should be used. In open ocean conditions, an equal probability distribution of wave headings can be assumed.

The maximum relative wave elevation can be calculated based on the following long-term analysis method:

i) The maximum wave frequency (WF) relative wave elevation can be calculated following the ABS *Guide for 'Dynamic Loading Approach' for Floating Production, Storage and Offloading (FPSO) Installations*. (see 4/5.3). The total maximum relative wave elevation can be calculated using the method given in 4/5.5.

The limiting sea states can be adjusted such that the total maximum relative wave elevation, r_{max}, at each field point does not exceed the freeboard.

ii) Alternatively, the long-term distribution of maximum relative wave elevation in a 3-hour sea state is the integration of the short-term cumulative extreme value distribution of maximum relative wave elevation in each sea state over all sea states weighted by the joint probability distribution for the sea states. The short-term extreme value distribution of the maximum relative wave elevation in a given 3-hour sea state can be modelled by a Gumbel distribution.

Mean and low frequency motions should be considered in the air gap analysis based on the method given in 4/5.5.

The total maximum relative wave elevation, r_{max}, in a 3-hour sea state at each field point from the long-term analysis can be obtained from the following equation:

$$Q(r_{max} \geq a_0) = \frac{q}{n_{3h\_operating}} \quad (4.11)$$

where

Q = integrated probability of freeboard exceedance for all sea states

q = annual probability of freeboard exceedance

$n_{3h\_operating}$ = number of 3-hour sea states per year while the semi-submersible in the operating condition

For a given annual probability of exceedance, the long-term limiting sea states can be determined from Equation (4.11) through an iteration of the air gap analysis by adjusting the limiting sea states.

SECTION 5 Second Order Time Domain Air Gap Analysis

1 Applications *(1 May 2020)*

The second-order time domain air gap analysis provides a more consistent and accurate approach to model the nonlinear effect of wave free surface elevation and vessel motion. The mean and low frequency motions under the combined effect of wind, wave, current, and mooring and riser systems can be calculated more accurately in time domain than in frequency domain. The nonlinear wave free surface elevations can be taken into account either using the first order wave free surface elevation RAOs modified by a wave asymmetry factor or through the second order wave free surface elevation QTFs converted into time domain.

The analysis method described in this Section is based on the following assumptions:

- The wave frequency motions are approximately linear. The wave frequency motions can be calculated in time domain together with the mean and low frequency motions. Alternatively, the wave frequency motions can also be calculated in frequency domain together with the wave free surface elevations using the linear diffraction-radiation analysis. Viscous effects and mooring and risers system effects can be considered approximatedly through linearlization in the frequency domain analysis.
- The diffracted and radiated wave free surface elevations are calculated based on either the linear diffraction-radiation theory or the second-order diffraction theory in frequency domain in terms of RAOs and QTFs.

It is noted however, when nonlinear effects are significant, the above assumptions may no longer hold. For such cases, the nonlinear effects should be investigated using either the direct solution of the nonlinear diffraction problem in time domain or CFD analysis. Validation through model tests is recommended. Specifically, the following nonlinear effects should be taken into considerations:

- Nonlinearity of wave free surface elevation that can not be predicted using the second-order diffraction theory. Nonlinear wave effects in shallow water, shallow water wave effects on top of pontoons, and nonlinear wave effects of wave run-up near columns are usually investigsted through model tests and CFD analysis. These nonlinear effects may be included in the analysis by additional factors, such as wave run-up factors.
- Nonlinearity of vessel motions, especially large motions, in the wave frequency range. Such nonlinear motions could be induced by nonlineear hydrostatic stiffness or Mathieu instability due to heave-pitch coupling.

3 Responses to Irregular Waves *(1 May 2020)*

The summation of the decomposed linear airy wave components from the wave spectrum produces the irregular wave free surface profile for time domain analysis, i.e.:

$$\eta(x, y, t) = \sum_{n=1}^{N} A_n \cos(\omega_n t - k_n x \cos\beta - k_n y \sin\beta - \theta_n) \quad (5.1)$$

where

η = wave free surface elevation of specified point (x, y) at time instant t

A_n = wave amplitude of the n-th Airy wave component

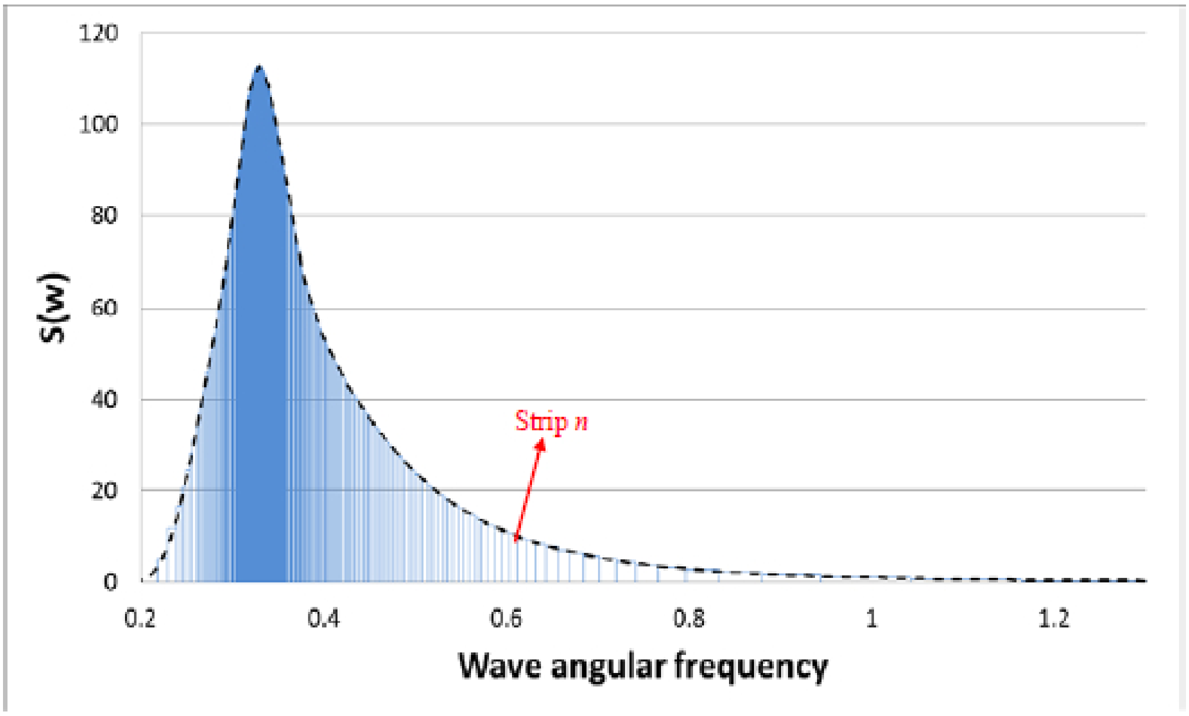
ω_n = wave frequency of the n-th Airy wave component

k_n = wave number of the n-th Airy wave component

θ_n = phase angle of the n-th Airy wave component

β = wave heading angle

FIGURE 1
Illustrating the Decomposition of Wave Spectrum

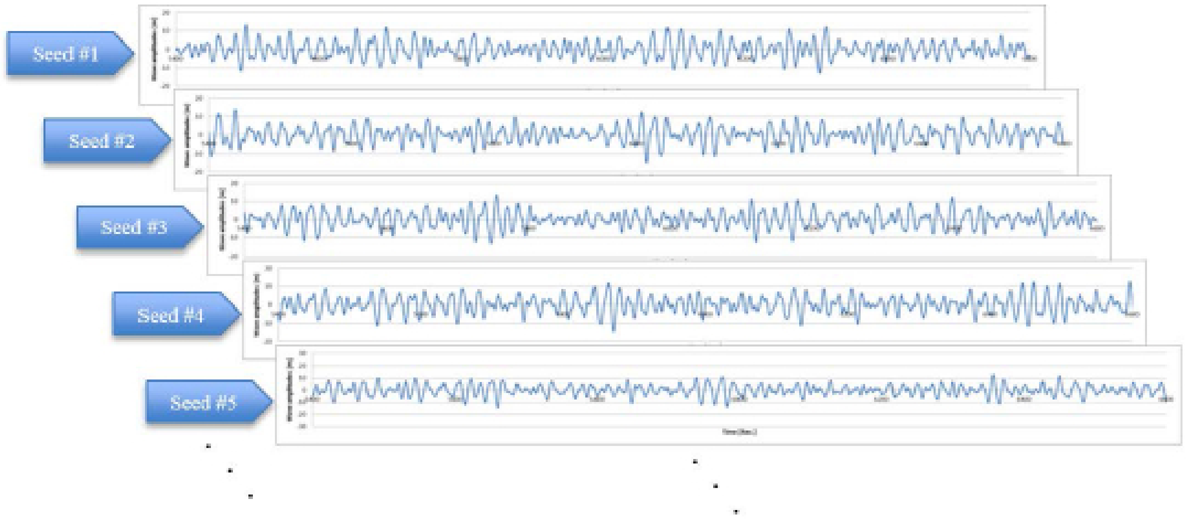


The wave spectrum is divided into strips with each strip having a center frequency (ω_n) corresponding to the frequency of one Airy wave component, as illustrated in 5/3 FIGURE 1. The wave amplitude of the n-th Airy wave component can be determined based on following formula.

$$\frac{1}{2}A_n^2 = S_\eta(\omega)\,\Delta\,\omega \quad (5.2)$$

The phase angle, θ_n, should be selected at random between 0 and 2π so that the wave free surface profile generated is random. In engineering practice, the random free surface profile is actually pseudo-random in order to maintain the reproducibility of analysis results. A common practice is to use a random seed number to represent one series of phase angle sequence. Different random seeds produce different wave profile time histories; however, the same random seed always generates the same wave profile time history. At least 200 Airy wave components should be considered in order to achieve the Gaussianity of the generated wave profile. 5/3 FIGURE 2 demonstrates the wave profile time histories generated based on same wave spectrum but different random seeds. Because of linearity, it is applicable to analyze the response to each Airy wave component separately.

FIGURE 2
Wave Profile Time Histories Generated by Different Random Seeds



The first order response to an irregular wave can be expressed as:

$$Res(x,y,t) = \sum_{n=1}^{N} A_n R(\omega_n,\beta)\cos[\omega_n t - k_n x\cos\beta - k_n y\sin\beta - \theta_n - \theta(\omega_n,\beta)] \quad (5.3)$$

where $R(\omega_n,\beta)$ and $\theta(\omega_n,\beta)$ are the RAO amplitude and phase angle, respectively.

Similarly, the second order response to an irregular wave can be written as:

$$Res^{(2)}(x,y,t) = Res^{(2)-}(x,y,t) + Res^{(2)+}(x,y,t) \quad (5.4)$$

$$Res^{(2)-}(x,y,t) = \sum_{m=1}^{N}\sum_{n=1}^{N} A_m A_n R^{(2)-}(\omega_m,\omega_n,\beta)\cos \left[(\omega_m - \omega_n)t - (k_m - k_n)x\cos\beta - (k_m - k_n)y\sin\beta - (\theta_m - \theta_n) - \theta^{(2)-}(\omega_m,\omega_n,\beta)\right] \quad (5.5)$$

$$Res^{(2)+}(x,y,t) = \sum_{m=1}^{N}\sum_{n=1}^{N} A_m A_n R^{(2)+}(\omega_m,\omega_n,\beta)\cos \left[(\omega_m + \omega_n)t - (k_m + k_n)x\cos\beta - (k_m + k_n)y\sin\beta - (\theta_m + \theta_n) - \theta^{(2)+}(\omega_m,\omega_n,\beta)\right] \quad (5.6)$$

where

$Res^{(2)}(x,y,t)$ = second order response at time instant t

$Res^{(2)-}(x,y,t)$ = difference-frequency response at time instant t

$Res^{(2)+}(x,y,t)$ = sum-frequency response at time instant t

$R^{(2)-}(\omega_m,\omega_n,\beta)$ = amplitude of difference-frequency QTF

$\theta^{(2)-}(\omega_m,\omega_n,\beta)$ = phase angle of difference-frequency QTF

$R^{(2)+}(\omega_m,\omega_n,\beta)$ = amplitude of sum-frequency QTF

$\theta^{(2)+}(\omega_m,\omega_n,\beta)$ = phase angle of sum-frequency QTF

For the air gap analysis for semi-submersibles, the effect of the sum-frequency second-order responses (see 2/3.5.3) is normally insignificant and thus can be ignored.

5 Time Histories of Relative Wave Elevation *(1 May 2020)*

According to Eq. 1.2, time histories of relative wave elevation at field points equal time histories of the free surface elevation at field points subtract corresponding time histories of vessel's vertical motion. In general, a minimum three-hour length time history is recommended to simulate the response to a stationary sea state.

5.1 Time Histories of Vertical Motions at Field Points

These Guidance Notes only consider mean and low frequency motion responses for semi-submersibles because the high frequency motion responses are several orders of magnitude smaller. The following two methods may be used to compute vertical motion time histories at field points:

5.1.1 Nonlinear Motion Analysis in Time Domain

When mean and low frequency motions are of critical concerns for the air gap assessment, the nonlinear motion analysis in time domain is preferred. Reference is made to 2/9. This approach can provide more accurate modeling of the nonlinear effects but requires more computational effort.

5.1.2 Hybrid Approach

In order to reduce computational effort, only the low frequency motions are computed in time domain and, subsequently, combined with the wave frequency motions calculated separately by converting the wave frequency motion RAOs in frequency domain to time domain. See also 2/11. The procedures are described as follows:

i) Calculate the first order vertical motion RAOs at field points for each combination of wave heading and wave frequency in accordance with Section 2.

ii) Convert the first order vertical motion responses in frequency domain to time domain following the procedure given in 5/3.

iii) Calculate the low frequency motion responses in time domain. Wave frequency motion response time histories and low frequency motion response time histories should correspond to same incident wave profile generated with the same random seed.

iv) Combine the wave frequency vertical motion time histories with the low frequency vertical motion time histories to obtain the total vertical motion time histories at field points. The initial transient response at the beginning portion should be truncated in order to establish stable statistical peak values.

5.3 Time Histories of Free Surface Elevation at Field Points *(1 May 2020)*

The free surface elevation can be determined through converting the first order solutions in terms of RAOs in frequency domain (see 3/1) to time domain and applying the wave asymmetric factor defined in 3/3 and, where applicable, the wave run up factor in 3/7. For simplicity, when wave frequency motions are calculated in frequency domain in terms of RAOs, the wave frequency vertical motion RAOs may be included in the relative wave elevation RAOs. The relative wave elevation RAOs can then be converted into time domain and added to the time histories of low frequency vertical motions from the time domain simulations.

Alternatively, if the free surface elevation QTFs can be determined, the procedures to compute the time histories of free surface elevation at field points up to the second order are outlined below:

i) Calculate RAOs, difference-frequency QTFs and sum-frequency QTFs of free surface elevation (See Section 3).

ii) Generate the incident wave time series using Eq. 5.1 for specified sea states. The random seed should be the same with the one used in generating vertical motion time histories at field points.

iii) Determine the first order free surface elevation time series, the difference-frequency free surface elevation time series and the sum-frequency free surface elevation time series using Eq. 5.3, Eq.

5.5 and Eq. 5.6, respectively. The time histories of free surface elevations at field points up to the second order are the summation of the respective linear and second order components. In contrast to motion responses, the high frequency free surface elevations could be larger than the low frequency components and cannot be ignored in steep waves.

It should be noted that the second order analysis as described above in this Subsection is not adequate to account for the local wave run up at locations close to columns. A suitable wave run-up factor should be applied.

7 Prediction of Extreme Relative Wave Elevation in Time Domain

(1 May 2020)

The maximum relative wave elevation observed directly from the time history during certain simulation period varies as the random waves initiated by the random seeds are different from simulation to simulation. Minimum air gaps of field points correspond to the most probable maximum extreme (MPME) relative wave elevation at field points, which has 63.2% chance of being exceeded by the maximum of any three-hour storm. If a time series of a response is a realization of a narrow band Gaussian process whose maxima follows Rayleigh distribution, the MPME values are readily available as discussed in 4/5. Due to the second order contributions, however, the relative wave elevation time history becomes a non-Gaussian process.

Various extreme value analysis methods have been developed to calculate the MPME value of a non-Gaussian process (ISO 19905-1). The commonly used Gumbel fitting method computes the MPME through fitting the maximum values of ten 3-hour simulations. The cumulative distribution function is expressed as:

$$F(r_{max};\psi,\kappa) = \exp\left[-\exp\left(-\frac{r_{max}-\psi}{\kappa}\right)\right] \quad (5.7)$$

where

$F(r_{max};\psi,\kappa)$ = probability that the 3-hour maximum will not exceed value r_{max}

r_{max} = maximum relative wave elevation

ψ = location parameter

κ = scale parameter

The MPME value discussed herein corresponds to an exceedance probability of 63.2% in an extreme probability distribution function. The MPME response (r_{extre}), thus, equals:

$$r_{extre} = \psi - \kappa\ln(-\ln(F(r_{extre}))) = \psi - \kappa\ln(-\ln(0.368)) \approx \psi \quad (5.8)$$

Both maximum likelihood method and method of moment can yield ψ and κ. Gumbel fitting method normally lead to reliable and stable extreme value but requires time-consuming computations since at least 10 three-hour simulations with different random seeds are required.

If the Winterstein/Jensen method and Weibull fitting method (see ISO 19905-1) are employed to compute MPME values, multiple simulations are recommended.

Depending upon the local design and regulatory requirements, other extreme value analysis methods for predicting the extreme value with higher non-exceedance probability may also be used.

SECTION 6 Horizontal Wave Impact Analysis *(1 October 2018)*

1 Application *(1 May 2020)*

In the case where the negative air gap occurs along the peripheral sides of the upper structure of a semi-submersible, the horizontal wave impact should be considered in assessing the local structural strength of the upper structure.

In this Section, the methodology and procedure for prediction of horizontal wave impact load and wave impact analysis are provided for the local structural strength assessment for the semi-submersibles subjected to negative air gap. The assessment conditions include, as applicable, the storm condition (see 1/5.3), operating condition (see 1/5.5) and transit condition (see 1/5.7). For environmental conditions with a return period longer than 100 years, model tests or detailed numerical analyses may be used to derive the horizontal wave impact loads.

The horizontal wave impact typically occurs on a localized area of exposed structural component. When the negative air gap is significant and could lead to the wave impact affecting a significant part of the upper structure or causing excessive green water on deck, the global effects of wave impact on global motions, global structural strength, mooring system integrity and stability may need to be assessed through detailed numerical analysis and/or model test.

The wave impact criteria specified in Appendix 5B-A1-1 of the *FPI Rules* apply to floating production installations, which include Column Stabilized Installations. Those criteria consider the wave impact as a solitary, abnormal event not directly related to the air gap and are applicable to outward facing portions of structures located between the still water level and the maximum wave crest elevation of the design sea state. These Guidance Notes provide direct analysis approaches for assessing the structural strength of the semi-submersible upper structure subject to horizontal wave impact due to the occurrence of negative air gap.

A negative air gap could also occur under the deck structure and cause vertical wave impact, which is addressed in Section 7.

The wave loads due to green water on deck are not covered in these Guidance Notes.

3 Horizontal Wave Impact Loads

3.1 General

When experimental data or detailed calculations are not available, the horizontal wave impact loads specified in this Subsection can be used to assess the local structural strength of peripheral sides of the upper structure. The following aspects are considered for the determination of the horizontal wave impact loads:

- Intensity and time duration of impact pressures
- Spatial distribution of impact pressures
- Geometry of the strucutral members subject to wave impact loads

3.3 Peak Impact Pressure *(1 May 2020)*

Experimental observations indicate that the measured horizontal wave impact pressure is dependent on the location and size of the measurement area (i.e. load averaging area). In the peak impact pressure, p_{peak}, as defined below in Equation (6.1), spatial adjustment coefficients are introduced to account for the variation of wave impact pressure with respect to the elevation, shape and size of a structural component. For small diameter tubular members of an open-truss in a topside deck, the impact pressure is often multiplied by the member's width to obtain the impact force per unit length along the member. An example is provided in Appendix A1 to demonstrate the horizontal wave impact load calculation for different structural components.

$$p_{peak} = k f_a f_e f_s H_s \qquad (6.1)$$

where

p_{peak} = peak horizontal wave impact pressure, kN/m^2 (tf/m^2, lbf/ft^2)

k = 65.5 (6.7, 417.0)

f_a = design factor

= 1.25

f_e = elevation factor, as defined in 6/3.3 TABLE 1 and 6/3.3 FIGURE 1

f_s = shape and size factor, as defined in 6/3.3 TABLE 2

H_s = significant wave height of the design sea states, in m (m, ft)

TABLE 1
Elevation Factor, f_e

Structural Member Elevation, z/r_{max}^{1,2}	f_e
$1.3 \le z/r_{max}$	0.02
$1.0 \le z/r_{max} < 1.3$	$-1.267\,z/r_{max} + 1.667$
$0.85 \le z/r_{max} < 1.0$	$-4.0\,z/r_{max} + 4.4$
$0.5 \le z/r_{max} < 0.85$	1.0
$0.2 \le z/r_{max} < 0.5$	$2.0\,z/r_{max}$
$z/r_{max} < 0.2$	0.4

Notes:

1 Elevation (z) is the vertical distance from the center of an individual structure member to the still water level.

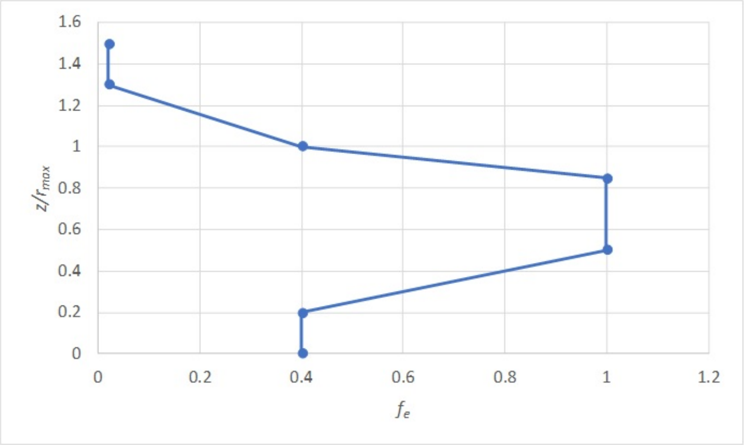
2 r_{max} is the maximum relative wave elevation, (see 4/5.5 and 5/7).

TABLE 2
Shape and Size Factor, f_s

Structural Member	f_s
Small tubular member (braces and truss members)	0.5
Plating and stiffener	1.0

Structural Member	f_S
Girder of stiffened panel	0.656
Girder of stiffened cylindrical shell	0.492

FIGURE 1
Elevation Factor, f_e



3.5 Wave Impact Load for Local Structural Analysis *(1 May 2020)*

The horizontal wave impact loads are impulse loads that can cause dynamic structural responses. To account for these dynamic effects due to wave impact on local structures, two alternative methods may be applied to the structural finite element analysis:

i) Time-domain dynamic structural analysis of the local structure subject to the dynamic wave impact pressure

ii) Static local structural analysis using the equivalent static horizontal wave impact pressure

3.5.1 Dynamic Local Structural Analysis

A dynamic analysis of the local structure should be performed in the time domain. The time history of the wave impact pressure is applied to calculate the dynamic structural responses.

A time history of dynamic wave impact pressure can be determined by:

i) Using the peak pressure given in 6/3.3 and a series of combinations of the peak impact pressure rise time and the total wave impact time duration. The time history of impact pressure can be approximated by a triangular shape as shown in 6/3.5.1 FIGURE 2. The typical rise time of the peak impact pressure is in the range from 10 ms to 20 ms. The total impact time duration can be taken from 20 ms to 40 ms. Different combinations of the rise time and the total wave impact time duration should be evaluated to identify the one that leads to the most unfavorable structural responses.

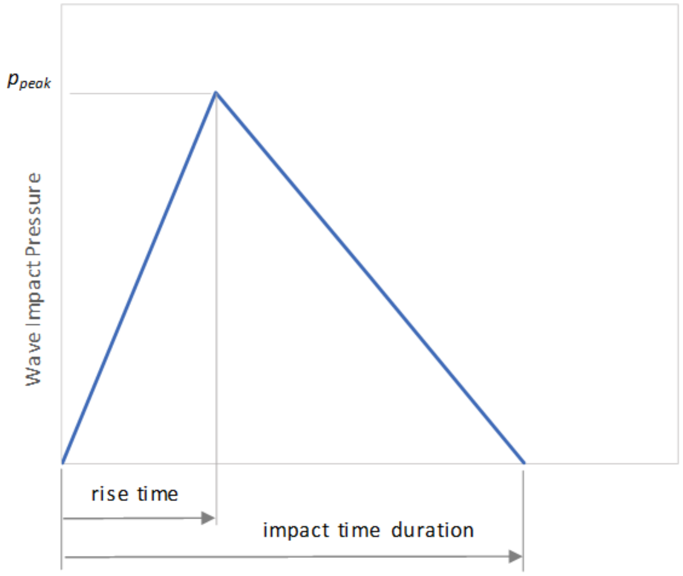
ii) Numerical simulations (see 6/3.7) and/or model tests (see 6/3.9)

The dynamic wave impact pressure should be applied within the extension of the upper structure boundaries and as described below:

- *For calculating responses in shell plating and stiffeners*: A uniform distribution on a square load patch of 3 m by 3 m (9.8 ft by 9.8 ft) applied at the center of the stiffener span and on the shell plating with an equal width on either side of the stiffener.

- *For calculating responses in girders*: A uniform distribution on a square load patch of 6 m by 6 m (19.7 ft by 19.78 ft) or of the size of girder span, whichever is larger, applied at the center of the girder span and on the shell plating with an equal width on either side of the girder.

FIGURE 2
Time History of Horizontal Wave Impact Pressure



3.5.2 Static Local Structural Analysis *(1 May 2020)*

The equivalent static horizontal wave impact pressure is calculated as the peak impact pressure multiplied by a dynamic amplification factor, i.e.:

$$p_s = k_d p_{peak} \qquad (6.2)$$

p_s = equivalent static horizontal wave impact pressure

k_d = dynamic amplification factor (not to be smaller than 1.0)

p_{peak} = peak horizontal wave impact pressure, as defined in 6/3.3

where

The dynamic amplification factor k_d can be derived by performing structural analyses using the dynamic wave impact pressure time history (see 6/3.5.2) and the peak horizontal wave impact pressure load, respectively. If such analyses are not available, the dynamic amplification factor may be taken as:

- k_d = 1.3 for plating, stiffeners and girders
- k_d = 1.75 for small tubular members (e.g., braces and truss members).

The equivalent static horizontal wave impact pressure should be applied within the extension of the upper structure boundaries and as described below:

- *For calculating responses in shell plating and stiffeners:* A uniform distribution on a square load patch of 3 m by 3 m (9.8 ft by 9.8 ft) applied at the center of the stiffener span and on the shell plating with an equal width on either side of the stiffener.

- *For calculating responses in girders:* A uniform distribution on a square load patch of 6 m by 6 m (19.7 ft by 19.78 ft) or of the size of girder span, whichever is larger, applied at the center of the girder span and on the shell plating with an equal width on either side of the girder.

3.7 Numerical Analysis of Wave Impact Loads

A direct calculation based on the nonlinear hydrodynamic analysis or CFD analysis can be used to predict the horizontal wave impact load for the local structural strength assessment. For wave-structure interactions, a coupled fluid dynamic and structural dynamic model may be used. Compressibility of water/air and elasticity of the structure may also be considered. Sea states selected for the numerical analysis should reflect the applicable design conditions described in 6/1 and include the critical sea states identified in the air gap analysis. Due to the random nature of wave impact, it is essential that the number of wave impact events simulated in the numerical analysis should be sufficient to allow reliable statistical analysis.

The numerical analysis method for predicting the horizontal wave impact load should be well documented and validated by model tests.

3.9 Model Test for Wave Impact Loads

Model test results can be used to derive wave impact loads. Sea states selected for the model test should reflect the applicable design conditions in Subsection 6/1 and include the critical sea states identified in the air gap analysis. Due to the random nature of wave impact, it is essential that the number of wave impact events collected from a model test program should be sufficient to allow reliable statistical analysis.

3.11 Effects of Hydroelasticity

Hydroelasticity may be considered in deriving the wave impact pressure time history. Model tests should be performed to verify the modeling of hydroelasticity. Not considering hydroelasticity is normally conservative for structural strength assessment.

5 Structural Strength Assessment and Design Consideration

(1 May 2020)

Where the negative air gap along the peripheral sides of the upper structure is identified, the wave impact criteria specified in Appendix 5B-A1-1 of the *FPI Rules* in conjunction with the wave height for the applicable design conditions described in 6/1 can be used to determine the scantling of the semi-submersible upper structure. In addition, for the outfacing upper structures located above the maximum relative wave crest height (i.e., $z/r_{max} > 1.0$), the scantling can be determined using the peak horizontal impact pressure defined in 6/3.3 along with the factor of safety $f_a = 1$.

The structural strength of the semi-submersibles upper structure subject to horizontal wave impact can be assessed using the direct analysis approach:

- The design conditions are defined in 6/1.
- The equivalent static horizontal impact load or dynamic impact load is defined in 6/3.
- Structural responses should be determined by the nonlinear finite element analysis using an implicit or explicit solver and with consideration of geometric nonlinearity and material nonlinearity.
- The material nonlinearity can be simulated using an elasto-plastic material model with the von Mises yield function. Other material models or yield functions may also be used based on the case-by-case review.
- The finite element model should consist of plate elements capable of modeling bending. Triangular plate elements should be avoided in the area of interest if possible.
- The mesh size and incremental time step should be determined by sensitivity study to verify that the strain converges within an acceptable tolerance. The mesh size should also be small enough to capture relevant failure modes.

- The post yield capacity of a structural member can be utilized in the strength check based on the case-by-case review. Stiffened panels should satisfy the structural member compactness and detailing requirements in 5B-A1-1/9 of the *FPI Rules*.
- When the main support to a plate stiffener is not a flexural element (such as a girder), it may be a deck, flat or bulkhead. In such a case, the in-plane strength and buckling resistance of the deck, flat or bulkhead should be sufficient such that the stiffener's supports and boundary conditions will perform as intended in design. The ABS *Guide for Buckling and Ultimate Strength Assessment for Offshore Structures* should be used in the assessment of stiffened panels comprising affected decks, flats and bulkheads.

Where the negative air gap along peripheral sides of the upper structure is identified, windows should in general not be installed within the horizontal wave impact zone measured up to $z/r_{max} = 1.3$. However, side scuttles provided with hinged inside deadlights may be installed in this zone. Deadlights should be capable of being closed and secured watertight to resist the horizontal wave impact pressure at the position where the side scuttle is installed.

SECTION 7 Deck Bottom Vertical Wave Impact Analysis
(1 May 2020)

1 General

1.1 Applications

In the case where negative air gap occurs underneath the upper structure, the deck box or topside deck structure and structural members with bottom surfaces exposed to waves are subject to deck bottom impact. These structures include bottom structures of the deck and overhanging structures or platforms, such as lifeboat platform and windlass platform of a semi-submersible. Vertical wave impact loads should be considered in the local structural strength assessment. In addition, wave impact loads on structures and equipment due to wave run-up near the column should be considered.

In this Section, the methodology and procedure for prediction of deck bottom vertical wave impact load and wave impact analysis are provided for the local structural strength assessment for the semi-submersibles subjected to negative air gap. The assessment conditions include, as applicable, the storm condition (see 1/5.3), operating condition (see 1/5.5) and transit condition (see 1/5.7). For environmental conditions with a return period longer than 100 years, model tests or detailed numerical analyses may be used to derive the vertical wave impact loads.

Vertical wave impact loads may have both global and local effects for the semi-submersible design. The vertical wave impact load is critical for local structural details because the force acts over a small area, leading to high local pressures. Wave run-up and local wave crest enhancement effects are considered as local effects. When the negative air gap is significant such that a large area of the deck bottom are under vertical wave impact loads, the global effects of vertical wave impact on global motions, global structural strength, mooring system integrity and stability may need to be assessed through detailed numerical analysis and/or model test.

The horizontal wave impact analysis is discussed in Section 6.

1.3 Deck Bottom Impact

Deck bottom impact may occur if relative wave elevation, $r(x,y,t)$, exceeds the still water air gap, $a_0(x,y)$. When the air gap becomes negative, water will impinge on the underside of the deck bottom structures. Impulse loads with high pressure peaks occur during the impact on the deck bottom.

For deck bottom impact, the impact velocity is taken as the relative vertical velocity between the wave surface and the deck bottom. The magnitude of the deck bottom impact pressure can be calculated by:

$$p = \frac{1}{2}\rho C_p V_R^2 \qquad (7.1)$$

where

ρ = water density

C_p = wave impact pressure coefficients

V_R = relative vertical velocity

Structures and equipment with a bottom surface in the path of wave run-up near the column can be subject to wave impact loads associated with wave run-up. The effect of wave run-up should preferably be assessed through model test. Numerical or empirical methods may be used provided that validation against model test is available.

3 Relative Vertical Velocity

3.1 General

The relative vertical velocity is the vertical free surface velocity relative to the deck bottom structure. The vertical free surface velocity can be obtained by taking the total derivative of the wave elevation with respect to time. Similarly, the relative vertical velocity can be taken as time derivative of the relative wave elevation.

The relative vertical velocity for a given point on the underside of structure exposed to wave can be calculated as:

$$V_R = \frac{d\eta}{dt} - \left(\frac{dX_3}{dt} + (y - y_G)\frac{dX_4}{dt} - (x - x_G)\frac{dX_5}{dt}\right) \qquad (7.2)$$

where

η	=	wave free surface elevation of specified point (x, y) at time instant t
$\frac{d\eta}{dt}$	=	time derivative of free surface elevation, i.e. vertical free surface velocity
(x, y, z)	=	coordinates of the given point
x_G, y_G	=	coordinate of COG in x-axis and y-axis, respectively
X_3	=	heave motion at COG
X_4	=	roll angular motion at COG
X_5	=	pitch angular motion at COG
$\frac{dX_3}{dt}$	=	heave velocity at COG
$\frac{dX_4}{dt}$	=	roll angular velocity at COG
$\frac{dX_5}{dt}$	=	pitch angular velocity at COG

3.3 Relative Vertical Velocity RAOs

The relative vertical velocity RAOs can be derived from the relative wave elevation RAOs. Based on Equation (4.1) through Equation (4.4) in 4/1, the relative wave elevation at specified point (x, y) in a unit amplitude sinusoidal wave with frequency ω can be expressed as:

$$r(x, y, t) = r_{in}\cos(\omega t) + r_{out}\sin(\omega t) \qquad (7.3)$$

By taking a time derivative of the Equation (7.3), the relative vertical velocity can be written as:

$$v(x, y, t) = \frac{dr}{dt} = \omega \cdot r_{out}\cos(\omega t) - \omega \cdot r_{in}\sin(\omega t) \qquad (7.4)$$

The relative vertical velocity RAOs, can be computed as below.

In-Phase:

$$v_{in} = \omega \cdot r_{out} \qquad (7.5)$$

Out-of-Phase:

$$v_{out} = -\omega \cdot r_{in} \qquad (7.6)$$

Amplitude:

$$v_{Amp} = \omega \cdot r_{Amp} \qquad (7.7)$$

Phase Angle:

$$v_{phase} = \arctan\left(-{}^{r_{in}}\!/_{r_{out}}\right) \qquad (7.8)$$

3.5 Extreme Relative Vertical Velocity

3.5.1 Short-term Extreme Value Analysis

The maximum relative vertical velocity due to wave frequency (WF) relative motion for a given sea state can be based on short-term extreme value analysis.

Maximum relative vertical velocity should be estimated when air gap $a(x, y, t)$ becomes non-positive as shown below:

$$r_{max} \geq a_0 \qquad (7.9)$$

The contribution to the total relative wave elevation due to wave frequency motion (r_{WF}) should be separated from the contributions due to mean and low frequency (LF) components. Based on 4/5.5, it can be derived that vertical wave impact will occur when

$$r_{WF} \geq \widetilde{a}_0 \qquad (7.10)$$

$$\widetilde{a}_0 = a_0 - (r_{max} - r_{WF}) \qquad (7.11)$$

where

$\widetilde{a}_0$ = still water air gap reduced by the mean and low frequency components

Assume that the wave-induced response, i.e. wave frequency response is a Gaussian stochastic process with zero mean, the peaks of the response follow Rayleigh distribution. The joint probability distribution of relative wave elevation (R, r) and relative vertical velocity (V_R, v), can be expressed by the following equation:

$$P(R \geq r, V_R \geq v) = \exp\left(-\frac{r^2}{2\sigma_r^2}\right) \ \exp\left(-\frac{v^2}{2\sigma_v^2}\right) \qquad (7.12)$$

For a given sea state, the extreme relative vertical velocity occurs once in three hours is,

$$P(R \geq r_{WF}, V_R \geq v_{WF}) = \exp\left(-\frac{\widetilde{a}_0^2}{2\sigma_r^2} - \frac{v_{WF}^2}{2\sigma_v^2}\right) = \frac{1}{N_r} \qquad (7.13)$$

Thus, the wave frequency (WF) extreme relative vertical velocity based on the most probable maximum extreme value (MPME), v_{WF}, to occur in 3 hours becomes:

$$v_{WF} = \sigma_v \sqrt{2 \cdot \left[\ln(N_r) - \frac{\widetilde{a}_0^2}{2\sigma_r^2} \right]} \qquad (7.14)$$

where

v_{WF} = extreme WF relative vertical velocity

σ_r = standard deviation of WF relative wave elevation

σ_v = standard deviation of WF relative vertical velocity

N_r = number of cycles of relative wave elevation in 3 hours

Depending upon the local design and regulatory requirements, the extreme value with a higher non-exceedance probability may also be used.

The response spectra of the relative wave elevation and relative vertical velocity can be calculated as:

$$S_r(\omega, \beta) = S(\omega) \cdot r_{Amp}^2(\omega, \beta) \qquad (7.15)$$

$$S_v(\omega, \beta) = S(\omega) \cdot \omega^2 r_{Amp}^2(\omega, \beta) \qquad (7.16)$$

The standard deviation of WF relative wave elevation (σ_r), standard deviation of WF relative vertical velocity (σ_v), and number of cycles of relative wave elevation in 3 hours (N_r) can be calculated as:

$$\sigma_r = \sqrt{\int_0^\infty S_r(\omega, \beta)\ \mathrm{d}\ \omega} \qquad (7.17)$$

$$\sigma_v = \sqrt{\int_0^\infty S_v(\omega, \beta)\ \mathrm{d}\ \omega} \qquad (7.18)$$

$$N_r = \frac{3 \cdot 3600}{2\pi} \sqrt{\frac{\int_0^\infty \omega^2 S_r(\omega, \beta)\ \mathrm{d}\ \omega}{\int_0^\infty S_r(\omega, \beta)\ \mathrm{d}\ \omega}} \qquad (7.19)$$

where

$S_r(\omega, \beta)$ = Response spectrum of relative wave elevation in a given sea state

$S_v(\omega, \beta)$ = Response spectrum of relative vertical velocity in a given sea state

3.5.2 Long-term Extreme Value Analysis

A long-term analysis can be performed by applying the joint probability in Equation (7.12) for all sea states in the scatter diagram.

Alternatively, the contour line approach may be used. For a given return period, the environmental contour line can be derived for an offshore site. Several sea states along the lines should be selected for the analysis.

3.5.3 Consideration of Mean and Low Frequency Vertical Motions

It can be assumed that the relative vertical velocity due to low frequency (LF) heel motion is insignificant and thus can be neglected. The total maximum relative vertical velocity, v_{max} can be calculated by:

$$v_{max} = v_{WF} \qquad (7.20)$$

3.7 Consideration of Wave Run-up

At locations near the column, the wave run-up effects should be considered in the vertical wave impact analysis.

The evaluation of air gap at locations very close to vertical surfaces is challenging because of local run-up in the form of water jets and sprays. Radiation-diffraction analysis based on a perturbation approach cannot give reliable results for locations closer than 0.15-0.20 times column diameter or width.

When the air gap analysis is performed based on the traditional radiation-diffraction analysis, the local wave run-up should be additionally considered at locations very close to the column wall, or to the vertical surface of the deck box.

The wave run-up height, the volume of the jet and its kinematics is a function of the wave steepness and the wave height to column diameter ratio. There is no simple method available to estimate the height and velocity in local run-up jets very close to the wall. Typical thickness of wave run-up jets is around 1 m with velocities up to 15-20 m/s. Run-up induced impact loads should be investigated by carefully instrumented model tests.

When the wave run-up factor is determined through model tests, the measured relative wave elevation can be used to account for local wave run-up or alternatively, wave run-up factors can be calibrated based on direct measurements of run-up induced force.

When there are no model test results available, empirical formula for wave run-up calculation from Stansberg, et al. (2010) may be applied

3.9 Consideration of Local Wave Crest Effects

Local wave crest effects as described in APR RP 2FPS should be considered in calculation of the vertical wave impact loads. A 15% increase in local crest elevation should be considered for local structural design.

At near column locations where wave run-up factors are applied, the 15% increase in local crest elevation may be omitted. However, the combined wave run-up factor with the local crest effects should not be less than 1.15.

3.11 Consideration of Effects of Forward Speed

When there is forward speed, such as the transit condition, the effects of forward speed should be considered.

Hydrodynamic analysis with forward speed should be performed to derive the relative wave elevation and relative vertical velocity.

5 Vertical Wave Impact Loads

5.1 General

In this section, the analysis procedure for vertical wave impact load prediction for local structural analysis is provided. The following aspects are considered for the determination of the vertical wave impact loads:

- Intensity and time duration of impact pressures
- Applied area of the vertical impact loads
- Geometry of the structural members subject to vertical wave impact loads

5.3 Peak Impact Pressure

The bottom impact pressure is impulsive loading. The magnitude of the peak impact pressure during a bottom impact is calculated by

$$p_{v\_peak} = \rho \cdot f_v \cdot f_a \cdot f_{s\_v} \cdot v_{WF}^2 \qquad (7.21)$$

where

p_{v_peak} = peak vertical wave impact pressure, kN/m^2 (T/m^2, lbf/ft^2)

ρ = water density, 1.025 (0.1045, 1.99)

f_v = 3.14

f_a = design factor

= 1.25

f_{s_v} = shape factor for vertical impact pressure

= 0.5, small tubular member (braces and truss members)

1.0, girder, plating and stiffener

v_{WF} = maximum relative vertical velocity calculated based on 7/3.5, m/s (m/s, ft/s)

5.5 Vertical Wave Impact Loads for Local Structural Analysis

The vertical wave impact loads are impulse loads that can cause dynamic structural responses. To account for these dynamic effects due to wave impact on local structures, two alternative methods may be applied to the structural finite element analysis:

i) Time-domain dynamic structural analysis of the local structure subject to the dynamic wave impact pressure

ii) Static local structural analysis using the equivalent static vertical wave impact pressure

5.5.1 Dynamic Local Structural Analysis

A dynamic local structural analysis should be performed in the time domain. The same procedure as given in 6/3.5.1 can be applied for dynamic local structural analysis under vertical wave impact loads.

A time history of dynamic vertical wave impact pressure can be determined by:

i) Using the peak pressure given in 7/5.3 and a series of combinations of the peak impact pressure rise time and the total wave impact time duration. The time history of impact pressure can be approximated by a triangular shape as shown in Section 6, Figure 2. The typical rise time of the peak impact pressure is in the range from 50 ms to 150 ms. The total impact time duration can be taken from 100 ms to 500 ms. Different combinations of the rise time and the total wave impact time duration should be evaluated to identify the one that leads to the most unfavorable structural responses.

ii) Numerical simulations (see 7/5.7) and/or model tests (see 7/5.9)

The pressure application area can be determined from model test. When model test results are not available, the pressure can be applied to the area as defined in 7/5.5.4.

5.5.2 Static Local Structural Analysis

The equivalent static vertical wave impact pressure is calculated as the peak impact pressure multiplied by a dynamic amplification factor, i.e.:

$$p_{v\_s} = k_d \cdot p_{v\_peak} \qquad (7.22)$$

where

p_{v_s} = equivalent static vertical wave impact pressure

k_d = dynamic amplification factor (not to be smaller than 1.0)

p_{v_peak} = peak vertical wave impact pressure, as defined in 7/5.3

The dynamic amplification factor k_d can be derived by performing structural analyses using the dynamic vertical wave impact pressure time history (see 7/5.5.1) and the peak vertical wave impact pressure load, respectively. If such analyses are not available, the dynamic amplification factor may be taken the same as given in 6/3.5.2.

The pressure applied area can be determined from model test. When model test results are not available, the pressure can be applied to the area as given in 7/5.5.4 within the boundary of deck bottom.

5.5.3 Structural Scantling Calculation

For structural scantling calculations, the peak vertical wave impact pressure as defined in 7/5.3 along with the design factor $f_a = 1$ can be used.

5.5.4 Area of Load Application

For both static and dynamic structural analysis, the vertical wave impact pressure should be applied within the boundary of bottom of the deck box or topside structures and as described below:

i) *For calculating responses in shell plating and stiffeners*: The bottom structure can be divided into different zones with plating and stiffeners divided by longitudinal and transverse girders, and vertical bulkheads. At least one field point should be selected at the center of the zone to calculate vertical wave impact pressure. When the distance of the zone to the column is closer than one diameter of the column, additional field points close to the column should be selected. Vertical wave impact pressures from different field points can be averaged. Pressure can be assumed a uniform distribution on the zone.

ii) *For calculating responses in girders:* An area can be taken along the span of the girder and half girder space at each side of the girder span. At least one field point should be selected at the center of the girder span to calculate vertical wave impact pressure. When the distance of the girder to the column is closer than one diameter of the column, additional field points close to the column should be selected. Vertical wave impact pressures from different field points can be averaged. Pressure can be assumed a uniform distribution along the girder span.

5.7 Numerical Analysis of Wave Impact Loads

A direct calculation based on the nonlinear hydrodynamic analysis or CFD analysis can be used to predict the vertical wave impact load for the local structural strength assessment. Considerations for numerical simulations are given in 6/3.7.

5.9 Model Test for Wave Impact Loads

Model test results can be used to derive vertical wave impact loads. Considerations for model test are given in 6/3.9.

5.11 Effects of Hydroelasticity

Considerations for effects of hydroelasticity are given in 6/3.11.

7 Structural Strength Assessment and Design Considerations

The structural strength of the semi-submersibles upper structure subject to vertical wave impact can be assessed using the direct analysis approach or based on scantling calculation.

For structural scantling calculations, 5B-A1-1/7 of the *FPI Rules* can be used to determine the scantling of the semi-submersible deck bottom structure under the peak vertical wave impact pressure as defined in 7/5.3 along with the design factor $f_a = 1$.

For direct analysis, the design considerations as given in 6/5 are applicable.

APPENDIX 1 Example of Horizontal Wave Impact Load Calculation *(1 May 2020)*

1 Example Semi-submersible and Upper Structure Particulars *(1 May 2020)*

A semi-submersible has a still water air gap of 13 m. The maximum significant wave height is 16 m. The calculated negative air gap is 4 m at all locations along peripheral sides of the upper structure. The calculated horizontal wave impact loads in the example is provided for local structural analysis.

The lower deck is 1.5 m above the deck bottom. The plate between the deck bottom and the lower deck is strengthened with vertical stiffeners.

The tween deck height is 2.5 m. The plate between the lower deck and the tween deck is strengthened with vertical girders and horizontal stiffeners with a 500 mm spacing.

The upper deck is 8 m above the deck bottom and 4 m above the tween deck. The plate between the tween deck and the upper deck is strengthened with vertical girders and horizontal stiffeners with a 1000 mm spacing.

3 Impact Loads for Plating and Stiffener Strength Assessment

Assume the calculated negative air gap is 4 m. The maximum relative crest height $r_{max} = 4 + 13 = 17$m. The size and shape factor $f_s = 1.0$. The design factor $f_a = 1.25$.

i) For plating and stiffeners between the lower deck and the deck bottom, the pressure is calculated at the elevation of the middle of the stiffener span.

The elevation $z = 13 + 1.5/2 = 13.75$ m. The ratio of the elevation to the maximum relative crest height is $z/r_{max} = 13.75/17 = 0.809$. From 6/3.3 TABLE 1 and 6/3.3 FIGURE 1, the elevation factor $f_e = 1.0$. The peak pressure is:

$$p_{peak} = kf_af_ef_sH_s = 65.5 \times 1.25 \times 1.0 \times 1.0 \times 16.0 = 1310 \quad \text{kN/m}^2$$

ii) For plating and stiffeners between the lower deck and the tween deck, the pressure is calculated at the elevation of the center of the lowest panel above the lower deck (i.e., 1.5 + 0.5/2 = 1.75 m above the deck bottom).

The elevation $z = 13 + 1.75 = 14.75$ m. The ratio of the elevation to the maximum relative crest height is $z/r_{max} = 14.75/17 = 0.868$. From 6/3.3 TABLE 1 and 6/3.3 FIGURE 1, the elevation factor $f_e = 0.928$. The peak pressure is:

$$p_{peak} = kf_af_ef_sH_s = 65.5 \times 1.25 \times 0.928 \times 1.0 \times 16.0 = 1216 \quad \text{kN/m}^2$$

iii) For plating and stiffeners between the tween deck and the upper deck, the pressure is calculated at the elevation of the center of the lowest panel above the tween deck (i.e., 4 + 1/2 = 4.5 m above the deck bottom).

The elevation $z = 13 + 4.5 = 17.5$ m. The ratio of the elevation to the maximum relative crest height is $z/r_{max} = 17.5/17 = 1.029$. From 6/3.3 TABLE 1 and 6/3.3 FIGURE 1, the elevation factor $f_e = 0.363$. The peak pressure is:

$$p_{peak} = kf_a f_e f_s H_s = 65.5 \times 1.25 \times 0.363 \times 1.0 \times 16.0 = 476 \quad \text{kN/m}^2$$

5 Impact Loads for Girder Strength Assessment

Assume the calculated negative air gap is 4 m. The maximum relative crest height $r_{max} = 4 + 13 = 17$ m. The size and shape factor for girder of stiffed panel $f_s = 0.656$. The design factor $f_a = 1.25$.

i) For the vertical girder between the lower deck and the tween deck, the pressure is calculated at the elevation of the mid of the girder span (i.e., 1.5 + 2.5/2 = 2.75 m above the deck bottom).

The elevation $z = 13 + 2.75 = 15.75$ m. The ratio of the elevation to the maximum relative crest height is $z/r_{max} = 15.75/17 = 0.926$. From 6/3.3 TABLE 1 and 6/3.3 FIGURE 1, the elevation factor $f_e = 0.696$. The peak pressure is:

$$p_{peak} = kf_a f_e f_s H_s = 65.5 \times 1.25 \times 0.696 \times 0.656 \times 16.0 = 598 \quad \text{kN/m}^2$$

ii) For the vertical girder between the tween deck and the upper deck, the pressure is calculated at the elevation of the mid of the girder span (i.e., 4 + 4/2 = 6 m above the deck bottom).

The elevation $z = 13 + 6 = 19$ m. The ratio of the elevation to the maximum relative crest height is $z/r_{max} = 19/17 = 1.118$. From 6/3.3 TABLE 1 and 6/3.3 FIGURE 1, the elevation factor $f_e = 0.250$. The peak pressure is:

$$p_{peak} = kf_a f_e f_s H_s = 65.5 \times 1.25 \times 0.250 \times 0.656 \times 16.0 = 215 \quad \text{kN/m}^2$$

APPENDIX 2 **References** *(1 October 2018)*

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