



COMMENTARY ON THE GUIDE FOR

BUCKLING AND ULTIMATE STRENGTH ASSESSMENT FOR OFFSHORE STRUCTURES

MARCH 2005 (Updated February 2014 – see next page)

**American Bureau of Shipping
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Updates

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Foreword

This Commentary provides the fundamental principles and technical background, including sources and additional details, for the *ABS Guide for Buckling and Ultimate Strength Assessment for Offshore Structures*, April 2004, which is referred to herein as “the *ABS Buckling Guide*”. The Commentary presents supplementary information to better explain the basis and intent of the criteria that are used in the *ABS Buckling Guide*. The accuracy for determining buckling and the ultimate strength predictions obtained from the application of the *ABS Buckling Guide* is established by the comparison of its results against a very extensive database of test results assembled by ABS and also from the results of nonlinear finite element analysis. Results obtained using the criteria in the *ABS Buckling Guide* are also compared against existing recognized offshore standards, such as the *ABS MODU Rules*, *API RP 2A WSD*, *API Bulletins 2U and 2V*, *DnV CN30.1*, and *AISC LRFD* [references 1, 3, 5, 6, 7, 30].

It should be understood that the Commentary is applicable only to the indicated version of the *ABS Buckling Guide*. The order of presentation of the material in this Commentary generally follows that of the *ABS Buckling Guide*. The major topic headings of the Sections in both, the *ABS Buckling Guide* and the Commentary are the same, but the detailed contents of the individual Subsections will not typically have a one-to-one correspondence between the *ABS Buckling Guide* and the Commentary.

In case of a conflict between anything presented herein and the ABS Rules or Guides, precedence is given to the ABS Rules or Guides.

ABS welcomes comments and suggestions for improvement of this Commentary. Comments or suggestions can be sent electronically to rdd@eagle.org.

ABS COMMENTARY ON THE GUIDE FOR BUCKLING & ULTIMATE STRENGTH ASSESSMENT FOR OFFSHORE STRUCTURES 2005

COMMENTARY ON THE GUIDE FOR BUCKLING AND ULTIMATE STRENGTH ASSESSMENT FOR OFFSHORE STRUCTURES

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SECTION **C1 Introduction**

C1 General

An important aspect in the design of an offshore steel structure is the buckling and ultimate strength behavior of its fundamental structural components. To ensure high technical quality, ABS consulted a number of internationally recognized experts in the theoretical and experimental study of buckling and ultimate strength behaviors for offshore structures. This Commentary has been compiled to give background information on the formulations and design guidance presented in the ABS *Buckling Guide* to help engineers better understand some of the fundamental principles that form the basis of that Guide.

The Commentary follows the same format and the section numbering of the ABS *Buckling Guide*. The same nomenclature is adopted in most cases, and new symbols are defined where they are used in the Commentary. Some sections that are deemed to be sufficiently clear in the ABS *Buckling Guide* will be intentionally left blank in this Commentary.

The design criteria adopted in the ABS *Buckling Guide* use a working stress format, where the acting stresses are to be less than or equal to the allowable stresses. The working stress format is deterministic; therefore, uncertainties in loads and resistances are not specially addressed, but are inherently incorporated into the maximum strength allowable utilization factors.

The formulations proposed are generally based on the premises that:

- They should not depart significantly from the formulations presented in ABS existing Rules and Guides and be consistent throughout the whole Guide;
- Where departures from existing ABS formulations are recommended, they should tend towards a formulation presented in other widely used design standards, such as API RP 2A-WSD;
- Where appropriate, improvement in formulation accuracy, whether the starting point is ABS *MODU Rules*^[1], ABS *Steel Vessel Rules*^[2] or API RP 2A-WSD^[3], should be included in the proposed formulations.

In order to validate the two- or three-dimensional interaction equations of buckling and ultimate strength proposed in the Guide, a modeling uncertainty is introduced, which was suggested by Hoadley and Yura(1985)^[4]. The modeling uncertainty is the ratio of the distance from the origin to the test data point in question, L_1 , over the distance from the origin to the interaction curve, L_2 , and is written by:

$$\text{Modeling Uncertainty} = L_1/L_2$$

An example of the modeling uncertainty is shown in Section C1, Figure 1. From this definition, the buckling and ultimate strength prediction is conservative if modeling uncertainty is greater than 1.0. The modeling uncertainty is especially useful because it can be used in one, two and three dimensions, and it is not a function of the exponent of each term in the interaction equation. In addition, it can be used to determine the amount of conservatism in a state limit when the experimental points are outside the range of the interaction equation when excluding factors of safety.

This concept is also extended to determine the amount of conservatism of a design when design loads are inside the range of the interaction equation including factors of safety. In spreadsheets developed by the ABS Offshore Technology Department, the so-called ‘unity check’ method is used. In this method, the unity check is done by calculating the ratio of the distance, Q_1 , from the origin to the design load point A , over the distance, Q_2 from the origin to the point B on the interaction curve, as shown schematically in Section C1, Figure 2 and written by:

$$\text{Unity ratio} = Q_1/Q_2$$

The design is acceptable if the unity ratio is less than 1.0.

FIGURE 1
Definition of Modeling Uncertainty

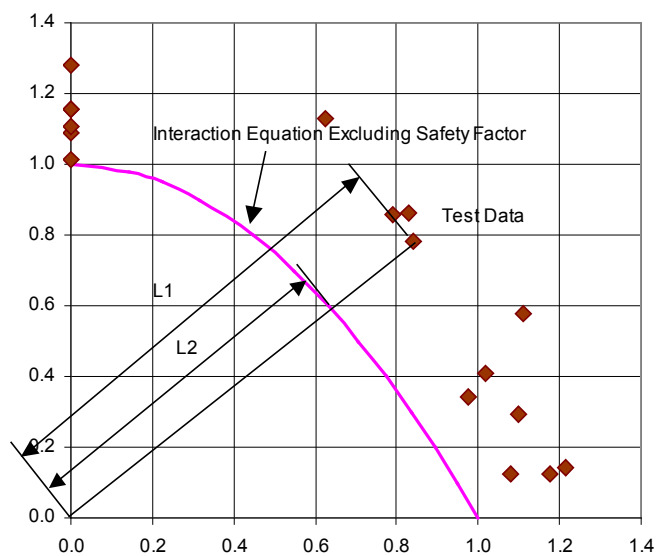
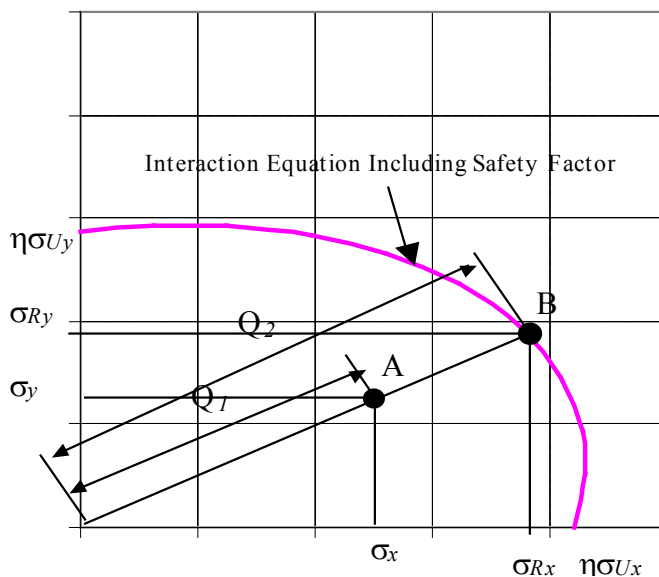


FIGURE 2
Definition of Unity Ratio



C3 Scope of This Commentary

The information in this Commentary is presented so that it can be helpful to engineers and designers in several ways. First, the comparison between the Guide's predictions and experimental and/or nonlinear FEM analysis results provides confidence about the application of the *ABS Buckling Guide*. Also the information can be useful in cases where the design parameters fall outside the range of validating test data. In such circumstances, engineers should seek to develop properly validated information. However, it is also possible to extrapolate the trends of the present criteria in initial design. It is expected that the background data presented here will be valuable in the extrapolation process.

The *ABS Buckling Guide* includes criteria for commonly used structural components in offshore structures such as:

- i) Individual Structural Members
- ii) Plates, Stiffened Panels and Corrugated Panels
- iii) Stiffened Circular Cylindrical Shells
- iv) Tubular Joints

In order to verify and validate the criteria, the ABS Offshore Technology Department gathered information and constructed a comprehensive test database in conjunction with a large amount of nonlinear buckling analysis results using the ANSYS program. The proposed criteria are also compared with results obtained from existing offshore codes, including API RP 2A WSD^[3], API Bulletin 2U^[5] and 2V^[6], and DnV CN30.1^[7], DnV RP C201^[8] and DnV RP C202^[142], etc.

The *ABS Buckling Guide* also provides guidelines on the use of an alternative Buckling Analysis by FEM, when adequate documentation is presented. It is important that new analysis methods be compared to recognized test results and/or service experience before they are declared fit for use.

C5 Tolerances and Imperfections

Determining the critical load of structural components subjected to compressive loads generally requires that major imperfections along with loading eccentricities be taken into account. However, there is a general lack of information about all of the aspects of initial geometric imperfections and residual stresses which could exist in actual structure. Because of their effect on strength, it is important that imperfections be monitored and repaired, as necessary, not only during construction, but also in the completed structure to ensure that the structural components satisfy tolerance limits. The tolerances on imperfections to which the strength criteria are considered valid are listed, for example, in the [ABS IACS Recommendation No. 47 "Shipbuilding and Repair Quality Standard"](#)^[9]. Imperfections exceeding such published tolerances are not acceptable unless it is shown using a recognized method that the strength capacity and the utilization factor of the imperfect structural component are within proper target safety levels.

C7 Corrosion Wastage

Offshore structures are exposed to the marine environment requiring the use of counteractive corrosion measures such as: a cathodic protection system, protective coatings or both to prevent corrosion damage. Despite these measures, there are numerous cases where offshore structural components have suffered from unexpected corrosion damage. Testing of a corroded member indicates that even nominal values of corrosion can result in lost capacity up to 35% to 50% (Ricles et al^[10]). Therefore, special care should be taken in the buckling and ultimate strength assessment of corroded components. It is recommended that actual as-gauged minimum thickness be used in the buckling and ultimate strength assessment in order to keep the corroded components within the acceptable safety level.

C9 Loadings

Offshore structures should be designed for the appropriate loading conditions, which produce the most severe, probable effects on the structure. For more detail information on loadings and loading conditions, refer to relevant ABS offshore Rules/Guides and API Recommended Practices.

C11 Maximum Allowable Strength Utilization Factors

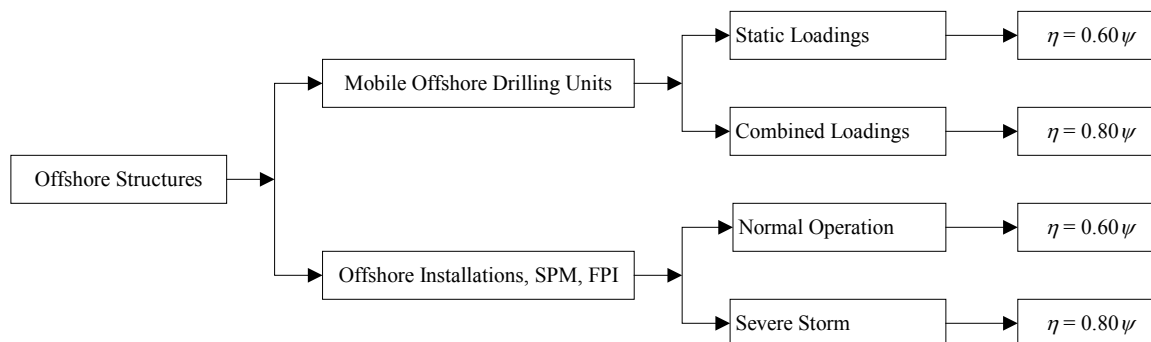
Major industry codes and the ABS criteria are used as references to establish the allowable utilization factors. In these codes and the ABS *Buckling Guide*, the design basis is working stress design, which implies that the acting stresses cannot exceed specified allowable values. The permissible stresses depend on the allowable utilization factor, η , or factor of safety, and structural critical strength. The relation between factor of safety and allowable utilization factor is as follows:

$$\eta = \frac{1}{\text{Factor of Safety}}$$

The establishment of the allowable utilization factors is not a trivial matter. Allowable utilization factors are a function of the loading conditions, the type of the structural components and the possible consequences of failure. They must take into account such matters as the following: the accuracy of the determined loads, inaccuracies in construction/quality of workmanship, variations in the properties of the material, deterioration due to corrosion or other environmental effects, accuracy of the method of analysis, consequences of failure (minor damage or major catastrophe) and so on.

The allowable utilization factors vary among different design codes. One design code can permit more or less risk than another. Besides, the design codes use different formulations for the prediction of the buckling and ultimate strength of structural components. Depending on the formulation of the strength used the utilization factor might be higher or lower, but typically not by much.

The allowable utilization factors may be divided into two parts in the current offshore codes. The first part corresponds to the basic utilization factors, which are very consistent among the codes and depend merely on the load conditions (See Section C1, Tables 1 to 4). The second part is related to the adjustment factors. These factors are different among the current offshore codes and will be discussed in subsequent sections. The allowable utilization factors are the product of basic utilization factors and adjustment factors. In the ABS *Buckling Guide*, the maximum allowable strength utilization factors have the following values.



where

ψ = adjustment factor

TABLE 1
Basic Utilization Factors in ABS MODU Rules^[1]

Load Conditions	Environmental Events	Basic Utilization Factors
Static Loadings	Operational gravity loads and the weight of the unit	0.60
Combined Loadings	Static loads combined with relevant environmental loads	0.80

TABLE 2
Basic Utilization Factors in ABS FPI Rules^[11]

<i>Load Conditions</i>	<i>Environmental Events</i>	<i>Basic Utilization Factors</i>
Loadout	Calm	0.60
Ocean Transit	10-year-return storm for the selected route condition (Owner specified)	0.80
Field Transit	1-year-return storm for the selected route condition (Owner specified)	0.80
Deck Installation	Calm	0.60
In-place Design Operating	1-year-return storm (minimum)	0.60
In-place Design Environmental	100-year-return storm at specific site	0.80
In-place Damaged	1-year-return storm	0.80

TABLE 3
Basic Utilization Factors in API 2A WSD^[3], Bulletin 2U^[4] and 2V^[5]

<i>Load Conditions</i>	<i>Environmental Events</i>	<i>Basic Utilization Factors</i>
Operations	Operating environmental conditions combined with dead loads and maximum live loads appropriate to normal operations of the platform.	0.60
Design Environments	Design environmental conditions with dead loads and maximum live loads appropriate for combining with extreme conditions.	0.80
Earthquake	Earthquake induced loading combined with gravity, hydrostatic pressure and buoyancy	1.0

TABLE 4
Basic Utilization Factors in DnV MOU Rules^[13]

<i>Load Conditions</i>	<i>Basic Utilization Factors</i>
Functional loads	0.60
Maximum environmental loads and associated functional loads	0.80
Accidental loads and associated functional loads	0.80
Environmental loads corresponding to a return period of 1 year and associated functional loads after credible failure, or accidental events	1.00
Environmental loads corresponding to a return period of 1 year and associated functional loads in a heeled condition corresponding to accidental flooding.	1.00

SECTION **C2 Individual Structural Members**

C1 General

Many assemblies used in offshore structures are made up of a variety of sectional shapes joined together by bolts, rivets or welds. For example in a jack-up (self-elevating) unit, the bracing members of a latticed leg are typically hollow tubular members while the leg chords have a variety of shapes such as tubulars, split tubulars around solid rack plates, and even hollow prismatic chord members.

A WSD methodology is widely used in the design check process as embodied in the ABS existing Rules/Guides. The WSD approach is also widely used in the design of tubulars in jacket structures via API RP 2A-WSD^[3].

C1.1 Geometries and Properties of Structural Members

The ABS *Buckling Guide* covers structural members that have at least a single axis symmetry.

It is noted that the formulations for the calculations of geometrical properties listed in Section 2, Table 1 of the ABS *Buckling Guide* are derived based on the assumption that the wall thickness is relatively small. For sections with relatively thick wall or sections not listed in this table, the key geometrical properties are to be calculated based on acceptable formulations.

C1.3 Load Application

<No Commentary>

C1.5 Failure Modes

The failure modes specifically addressed in the existing ABS *MODU Rules* are:

- Axial tension
- Axial compression including overall buckling
- Bending
- Shear
- Combined axial tension and bending moment
- Combined axial compression and bending moment
- Combined axial tension, bending moment and hydrostatic pressure
- Combined axial compression, bending moment and hydrostatic pressure

Other possible failure modes might include local buckling under axial compression or bending and any loading condition involving external or hydrostatic pressure. However, for a jack-up unit, these failure modes are not necessarily of critical concern for two reasons. Firstly, the cross-sectional slenderness of tubular sections used in jack-up construction is usually so low as to preclude any local buckling considerations. Secondly, for the water depths in which present-day jack-ups operate and again because of the low tubular slenderness involved, any pressure effects in respect to affecting tubular strength under axial, bending and shear loading conditions are minimal.

In relation to the appropriateness of the provisions specifically addressed in the present ABS *MODU Rules*, most of these are soundly based: although some aspects do need consideration as discussed below. However, in relation to bending, the major omission is the lack of provisions that allow the development of any plastic hinge capacity for tubular members. As will be seen later, this represents an extremely conservative approach when designing particularly low slenderness tubular for bending.

In identifying the requirements for revised design criteria for tubular members of ABS *MODU Rules*^[1], the aim is to overcome the more relevant of the weaknesses listed above. Local buckling under axial compression and bending is of importance in this respect. Also is the need to deal with hydrostatic pressure.

For the structural members with rolled and fabricated plate section, the failure modes of torsional buckling and lateral-torsional buckling also need to be taken into account (Timenshenko and Gere^[15]).

C1.7 Cross Section Classification

Cross-sections should be classified based on whether local buckling limits the maximum attainable strength. The assessment of each element of a cross-section subjected to compression due to a bending moment or an axial force should be based on its outer diameter-to-thickness ratio for tubular members or width-to-thickness ratio for rolled or fabricated-plate sections. The dimensions of these compression elements should be taken from Section 2, Table 1 of the *ABS Buckling Guide*. Each element of a cross-section is generally of constant thickness. A distinction should be made between the following types of element:

C1.7.1 Tubular Members

The compact limit for tubular section is defined by:

$$\frac{D}{t} \leq \frac{E}{9\sigma_0}$$

where

D	=	outer diameter
t	=	wall thickness
E	=	Young's modulus
σ_0	=	yield strength

C1.7.2 Rolled or Fabricated-Plate Sections

C1.7.2(a) Outstand elements attached to an adjacent element at one edge only, with the other edge free. The compact limit is defined by:

$$\frac{b_2}{t_f} \leq 0.4 \sqrt{\frac{E}{\sigma_0}}$$

where

b_2	=	width of outstand.
t_f	=	thickness of outstand.

C1.7.2(b) Internal elements attached to other elements on both longitudinal edges and including:

- Webs consisting of internal elements perpendicular to the axis of bending
- Flanges consisting of internal elements parallel to the axis of bending

The compact limit is defined by:

$$\frac{a}{t_f}, \frac{d}{t_w} \leq 1.5 \sqrt{\frac{E}{\sigma_0}}$$

where

a	=	width of an internal flange
t_f	=	thickness of an internal flange
d	=	height of webs
t_w	=	thickness of webs

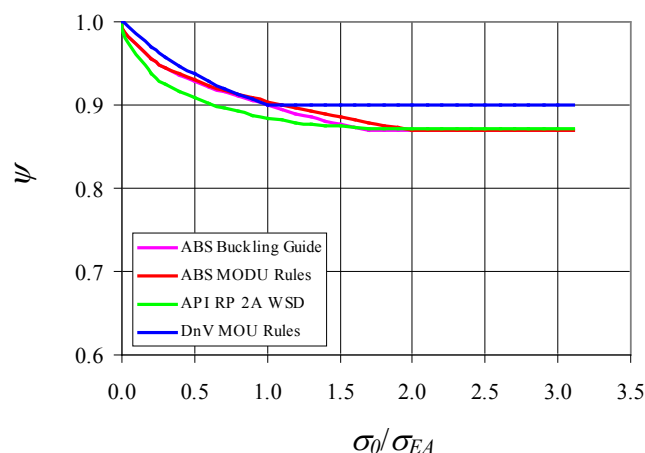
C1.9 Adjustment Factors

The adjustment factors for the allowable basic utilization factors in the existing offshore codes are provided in Section C2, Table 1 and Section C2, Figure 1. The adjustment factors for beam-column buckling and tension/bending from various codes are remarkably identical and the difference is less than 3.5%.

TABLE 1
Adjustment Factors

	<i>Beam-Column Buckling</i>	<i>Tension/ Bending</i>	<i>Local Buckling</i>
ABS Buckling Guide	$\psi = 0.87$ if $\sigma_{EA} \leq P_r \sigma_0$ $= 1 - 0.13 \sqrt{P_r \sigma_0 / \sigma_{EA}}$ if $\sigma_{EA} > P_r \sigma_0$	$\psi = 1.0$	$\psi = 0.833$ if $\sigma_{Ci} \leq 0.55 \sigma_0$ $= 0.629 + 0.371 \sigma_{Ci} / \sigma_0$ if $\sigma_{Ci} > 0.55 \sigma_0$
ABS MODU Rules ^[11] / SPM Rules ^[14]	$\psi = 0.87$ if $\sigma_{EA} \leq 0.5 \sigma_0$ $= 1 / (1 + 0.15 \sqrt{0.5 \sigma_0 / \sigma_{EA}})$ if $\sigma_{EA} > 0.5 \sigma_0$	$\psi = 1.0$	N/A
API RP WSD 2A (AISC) ^[3]	$\psi = 0.87$ if $\sigma_{EA} \leq 0.5 \sigma_0$ $= \frac{1}{1 + 0.1588 \left(\frac{\sigma_0}{\sigma_{EA}} \right)^{0.5} - 0.0529 \left(\frac{\sigma_0}{\sigma_{EA}} \right)^{1.5}}$ if $\sigma_{EA} > 0.5 \sigma_0$	$\psi = 1.0$	$\psi = 0.8333 \sim 1.0$ for axial compression $\psi = 0.8333$ for external pressure
DnV MOU Rules ^[13]	$\psi = 1.0$ if $\sigma_{EA} \geq 25 \sigma_0$ $= 1.025 - 0.125 \sqrt{\sigma_0 / \sigma_{EA}}$ if $\sigma_0 < \sigma_{EA} \leq 25 \sigma_0$ $= 0.9$ if $\sigma_{EA} \leq \sigma_0$	$\psi = 1.0$	N/A

FIGURE 1
Comparison of Adjustment Factor for Beam-Column Buckling



C3 Members Subjected to Single Actions

C3.1 Axial Tension

<No Commentary>

C3.3 Axial Compression

The proposed formulation includes flexural buckling and torsional buckling. The state limit is defined by the following equation:

$$\sigma_A / \eta_1 \sigma_{CA} \leq 1$$

where σ_A is the axial compressive stress and σ_{CA} is the critical buckling stress:

$$\sigma_{CA} = \begin{cases} \sigma_{EA} & \text{if } \sigma_{EA} \leq P_r \sigma_F \\ \sigma_F \left[1 - P_r (1 - P_r) \frac{\sigma_F}{\sigma_{EA}} \right] & \text{if } \sigma_{EA} > P_r \sigma_F \end{cases}$$

where

P_r = proportional linear elastic limit of the structures

σ_F = specified minimum yield strength for the compact section or local buckling stress for the non-compact section, as specified in 2/3.3 of the *ABS Buckling Guide*

σ_{EA} = elastic buckling stress, which is the least of the solutions of the following equation for an arbitrary thin-walled cross section (Timenshenko and Gere, 1961^[15]):

$$\frac{I_o}{A} (\sigma_{EA} - \sigma_{Ey})(\sigma_{EA} - \sigma_{Ez})(\sigma_{EA} - \sigma_{ET}) - \sigma_{EA}^2 y_0^2 (\sigma_{EA} - \sigma_{Ez}) - \sigma_{EA}^2 z_0^2 (\sigma_{EA} - \sigma_{Ey}) = 0$$

where

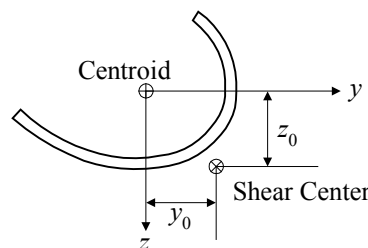
σ_{Ey}, σ_{Ez} = Euler buckling stresses

σ_{ET} = torsional buckling stress given by:

$$= \frac{EK}{2.6I_o} + \left(\frac{\pi}{kL} \right)^2 \frac{E\Gamma}{I_o}$$

y_0, z_0 = coordinates of shear center, as shown in Section C2, Figure 2

FIGURE 2
Geometry of Thin Walled Members



For a section with one plane of symmetry about a longitudinal axis, the elastic buckling stress can be solved from the following equation (Section 2/3.3 of the *ABS Buckling Guide*)

$$\frac{I_o}{A} (\sigma_{EA} - \sigma_{E\eta}) (\sigma_{EA} - \sigma_{ET}) - \sigma_{EA}^2 d_{es}^2 = 0$$

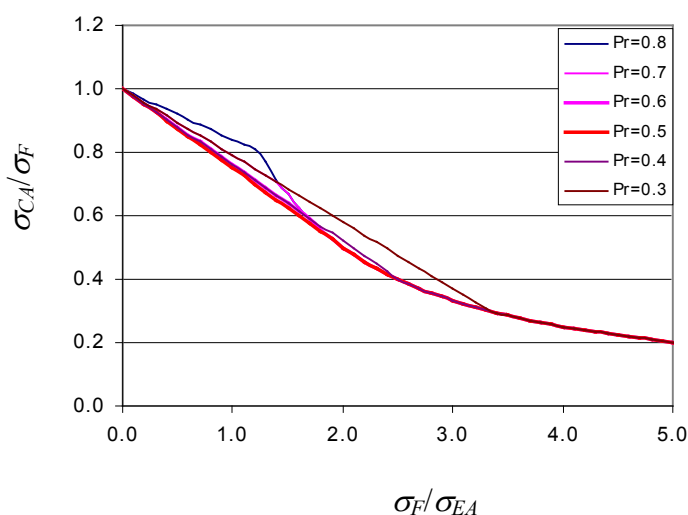
For an arbitrary section with one plane of symmetry, assuming it to be through the y -axis, we obtain the equation for calculating the elastic buckling stress

$$\frac{I_o}{A} (\sigma_{EA} - \sigma_{Ey}) (\sigma_{EA} - \sigma_{ET}) - \sigma_{EA}^2 y_0^2 = 0$$

The smaller of the roots and Euler buckling stress in the plane of symmetry represents the elastic buckling stress.

In the existing ABS Rules, API 2A WSD^[3] and AISC^[16], 0.5 is taken for P_r , but 0.6 is used in the Guide to be in agreement with ABS *Steel Vessel Rules*^[2]. Using 0.6 instead of 0.5 does not provide a big difference in the predicted strength (See two thick lines in Section C2, Figure 3).

FIGURE 3
The Effect of P_r on the Critical Buckling Stress



For tubular members, Section C2, Figure 4 presents a comparison of the ABS *MODU Rules*^[1], API RP 2A-WSD^[3] and the ABS *Buckling Guide* with test results. The column test database consists of 12 tests on fabricated tubular members, Chen and Ross^[17] and Smith et al^[18]; 2 on seamless pipe, Smith et al^[19]; and 70 on ERW pipe, Steinmann and Vojta^[20] and Yeomans^[21]. This is considerably larger than that previously used to validate offshore tubular strength formulations. The increase is primarily due to the inclusion of relevant results from a large CIDECT test program (Yeomans^[21]). The figure confirms that the ABS *MODU Rules*^[1] and API RP 2A-WSD^[3] formulations are identical. However, the statistics of the comparisons between the formulations and the test data indicate that differences do arise. For example, the means for the two formulations are 1.0736 and 1.0743 respectively. An examination of the calculation details reveals that differences arise because of an API RP 2A-WSD local strength requirement. This applies for $D/t \geq 60$; whereas the ABS *MODU Rules* local buckling limit is in excess of 60 (or using ABS *MODU Rules*^[1] definitions, $D/t > 59$) for yield stresses up to 386 N/mm². The mean and COV of modeling uncertainty of various codes are given in Section C2, Table 2.

Section C2, Table 3 provides comparison between the ABS *Buckling Guide* and AISC Code^[16] for two rolled-plate sections.

The allowable buckling stress from the ABS *Buckling Guide* is remarkably close to that from the AISC Code when local buckling is ignored, as is the case for compact sections. Differences arise for non-compact sections, in which the allowable buckling stress from the ABS *Buckling Guide* is considerably smaller than that from the AISC Code. This is reasonable because the ABS *Buckling Guide* includes the local buckling effect for non-compact sections.

TABLE 3
Column Buckling for Rolled-plate Sections

		<i>W-shape</i>	<i>Square Hollow Section</i>
Geometry and Material			
Length	ℓ	144.00	144.00
Section shape type		2	4
Specified minimum yield point	σ_o	36.00	36.00
Modulus of elasticity	E	2.90E+04	2.90E+04
Poisson's ratio	ν	0.30	0.30
Flange width	b	3.75	7.96
Flange thickness	t_f	0.25	0.43
Web depth	d	3.75	9.75
Web thickness	t_w	0.25	0.29
Section classification		Non-compact	Compact
The ABS Buckling Guide			
Allowable buckling stress considering local buckling		13.73	—
Allowable buckling stress ignoring local buckling		16.14	19.90
AISC Code			
Allowable buckling stress ignoring local buckling		15.99	19.46



C3.5 Bending Moment

The ABS *Buckling Guide* includes two failure modes that take proper account of plastic moment capacity and lateral torsional buckling capacity for the members.

The proposed buckling state limit is defined by the following equation:

$$\sigma_b / \eta_2 \sigma_{CB} \leq 1$$

where

σ_b = bending stress due to bending moment

σ_{CB} = characteristic bending strength given as follows:

- i) For tubular members, the critical bending strength is obtained from the equation in Section 2/9.3 of the ABS *Buckling Guide*, in which the fully plastic capacity of the section could be developed.
- ii) For members with rolled or fabricated sections, the critical bending strength is determined by the critical lateral-torsional buckling stress.

The critical lateral-torsional buckling stress is obtained by:

$$\sigma_{C(LT)} = \begin{cases} \sigma_{E(LT)} & \text{if } \sigma_{E(LT)} \leq P_r \sigma_F \\ \sigma_F \left[1 - P_r (1 - P_r) \frac{\sigma_F}{\sigma_{E(LT)}} \right] & \text{if } \sigma_{E(LT)} > P_r \sigma_F \end{cases}$$

where

$\sigma_{E(LT)}$ = elastic lateral-torsional buckling stress, which is given below (Timenshenko and Gere^[15])

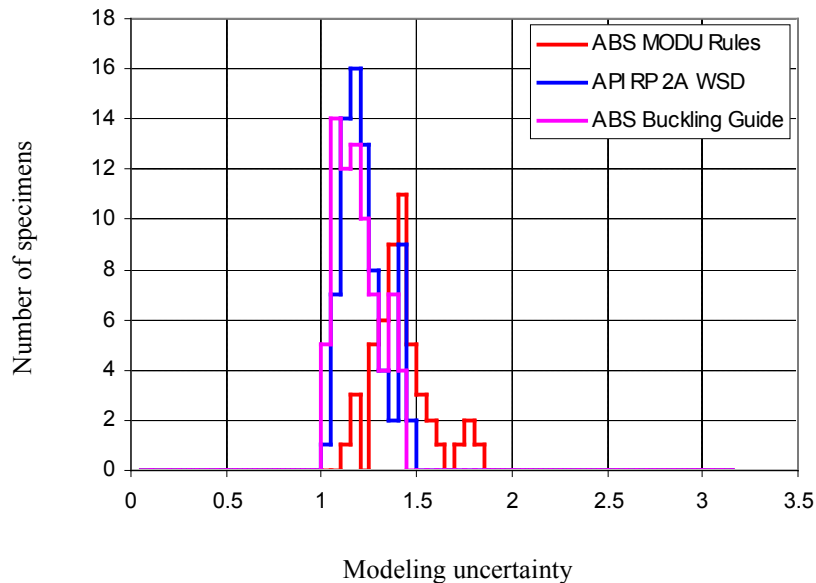
$$= \sqrt{C} \frac{\pi^2 EI_\xi}{SM_c (kL)^2}$$

Comparisons are presented for tubular members in Section C2, Figure 5 between the existing ABS *MODU Rules*, API RP 2A-WSD and the ABS *Buckling Guide* for bending and the test data. The bending database consists of 57 results published by Steinmann and Vojta^[20], Kiziltug et al^[22], Sherman^[23,24], Korol and Hudoba^[25] and Korol^[26]. In the ABS *MODU Rules*^[1], bending strength is limited to the range where $D/t \leq E/9\sigma_0$ or $\sigma_0 D/Et \approx \leq 0.11$ and local buckling effect is ignored. Over this valid range, the ABS *MODU Rules*^[1] underestimate the bending strength. Section C2, Table 4 presents the Mean/COV of modeling uncertainty of bending strength for tubular members.

TABLE 4
Mean/COV of Modeling Uncertainty of Bending Strength
for Tubular Members

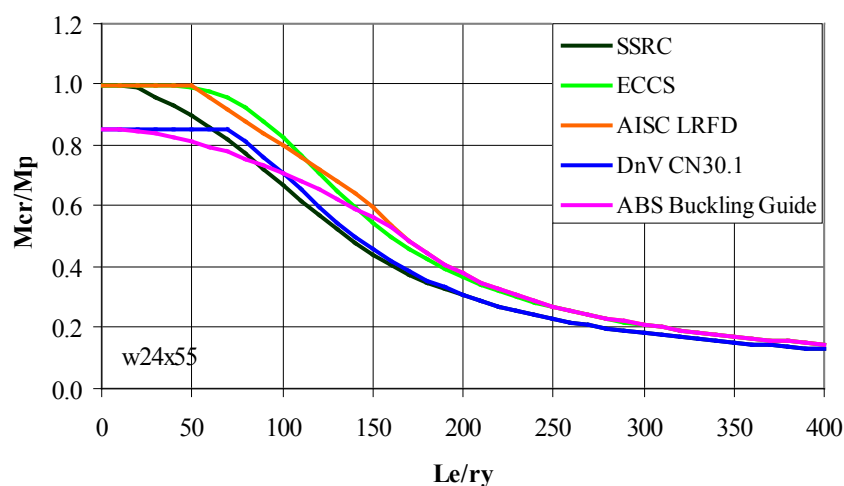
	<i>ABS MODU Rules</i>	<i>API RP 2A WSD</i>	<i>ABS Buckling Guide</i>
Mean	1.3678	1.1741	1.1463
COV	10.83%	9.40%	9.79%

FIGURE 5
Bending Strength for Tubular Members



The critical bending strength for the beams with rolled or fabricated compact sections obtained from SSR^C^[28], ECCS^[29], AISC LRFD^[30], DnV CN30.1^[7] and the ABS *Buckling Guide* is shown in Section C2, Figure 6. The criterion proposed in the ABS *Buckling Guide* is conservative for the short beam. In this case, the critical bending strength is governed by the development of full plasticity. The criterion proposed in the ABS *Buckling Guide* is acceptable for the beams with rolled or fabricated compact sections in the practical range of slenderness ratio.

FIGURE 6
Comparison of Lateral-Torsional Buckling Strength



C5 Members Subjected to Combined Loads

C5.1 Axial Tension and Bending Moment

<No Commentary>

C5.3 Axial Compression and Bending Moment

The criteria for combined column buckling and bending moment in the *ABS Buckling Guide* is based on the individual formulation for column buckling and bending combined via the interaction equation involving Euler amplification of deflections by axial loading. This applies when the ratio of axial stress σ_a to the column strength σ_{CA} is greater than 0.15, i.e., the buckling failure is dominant. Otherwise, a relationship that does not involve the amplification is adopted, in which a yield failure governs. The equations in the *ABS Buckling Guide* are:

For tubular members:

$$\frac{\sigma_a}{\eta_1 \sigma_{CA}} + \frac{1}{\eta_2} \left\{ \left[\frac{1}{\sigma_{CBY}} \frac{C_{my} \sigma_{by}}{1 - \sigma_a / (\eta_1 \sigma_{Ey})} \right]^2 + \left[\frac{1}{\sigma_{CBZ}} \frac{C_{mz} \sigma_{bz}}{1 - \sigma_a / (\eta_1 \sigma_{Ez})} \right]^2 \right\}^{0.5} \leq 1 \quad \text{when } \sigma_a / \sigma_{CA} > 0.15$$

$$\frac{\sigma_a}{\eta_1 \sigma_{CA}} + \frac{1}{\eta_2} \left[\left(\frac{\sigma_{by}}{\sigma_{CBY}} \right)^2 + \left(\frac{\sigma_{bz}}{\sigma_{CBZ}} \right)^2 \right]^{0.5} \leq 1 \quad \text{when } \sigma_a / \sigma_{CA} \leq 0.15$$

For rolled or fabricated plate sections:

$$\frac{\sigma_a}{\eta_1 \sigma_{CA}} + \frac{1}{\eta_2 \sigma_{CBY}} \frac{C_{my} \sigma_{by}}{1 - \sigma_a / (\eta_1 \sigma_{Ey})} + \frac{1}{\eta_2 \sigma_{CBZ}} \frac{C_{mz} \sigma_{bz}}{1 - \sigma_a / (\eta_1 \sigma_{Ez})} \leq 1 \quad \text{when } \sigma_a / \sigma_{CA} > 0.15$$

$$\frac{\sigma_a}{\eta_1 \sigma_{CA}} + \frac{\sigma_{by}}{\eta_2 \sigma_{CBY}} + \frac{\sigma_{bz}}{\eta_2 \sigma_{CBZ}} \leq 1 \quad \text{when } \sigma_a / \sigma_{CA} \leq 0.15$$

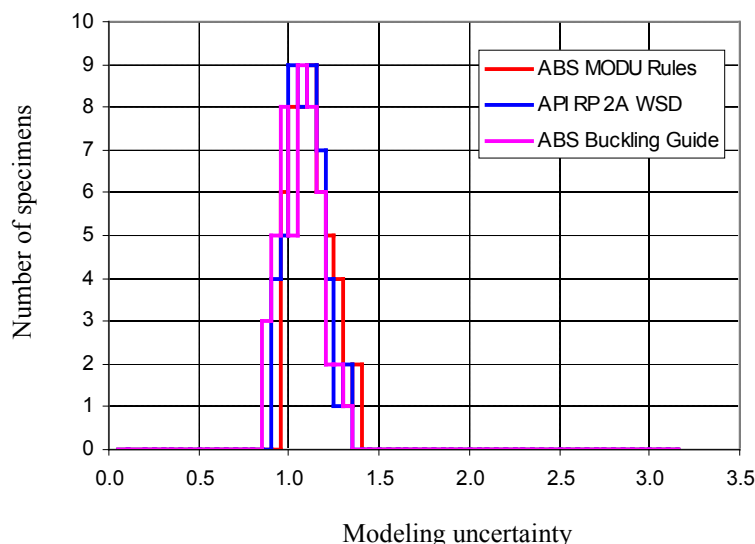
The comparisons between the *ABS Buckling Guide* for combined column buckling and bending and test data for tubular members are presented in Section C2, Figure 7. For overall buckling, 49 test results exist extracted from Prion and Birkemoe^[31], Kiziltug et al^[22], Ellis^[32], Wagner et al^[33], Kato and Akiyama^[27] and Smith et al^[18]: 34 results were rejected on the grounds of being too thin and inadequately documented. For local buckling, 19 data have been extracted from Kiziltug et al^[22] and Prion and Birkemoe^[31]: no data were rejected.

The mean and COV of modeling uncertainty of various codes are presented in Section C2, Table 5.

TABLE 5
Mean/COV of Modeling Uncertainty for Beam-Column Buckling

	<i>ABS MODU Rules</i>	<i>API RP 2A WSD</i>	<i>ABS Buckling Guide</i>
Mean	1.0811	1.0439	1.0180
COV	10.03%	9.52%	10.84%

FIGURE 7
Beam-Column Buckling for Tubular Members



C7 Tubular Members Subjected to Combined Loads with Hydrostatic Pressure

Unstiffened tubular members under external hydrostatic pressure are subjected to elastic or inelastic local buckling of the shell wall between restraints. Once initiated, the collapse will tend to flatten the member from one end to the other. Similarly, ring-stiffened members are subject to local buckling of the shell wall between rings. The shell buckles between the rings, while the rings remain essentially circular. Consequently, it is desirable to provide rings with sufficient reserve strength to prevent general instability.

Strength design interaction equations for the cases in which a tubular member is subjected to axial tension or compression, and/or bending combined with external hydrostatic pressure have been proposed in API RP 2A WSD. The hoop compression is not explicitly included in the analysis, but its effect on member design is considered within the design interaction equations. The hoop collapse design check must be satisfied first. The method described is based on the explicit application of the capped-end axial compression, which allows for a more precise redistribution of the capped-end load based on the relative stiffness of the braces at a node. A collection of the test data and additional comparisons of the design equations to test data can be found in Miller and Salikis^[34].

C7.1 Axial Tension, Bending Moment and Hydrostatic Pressure

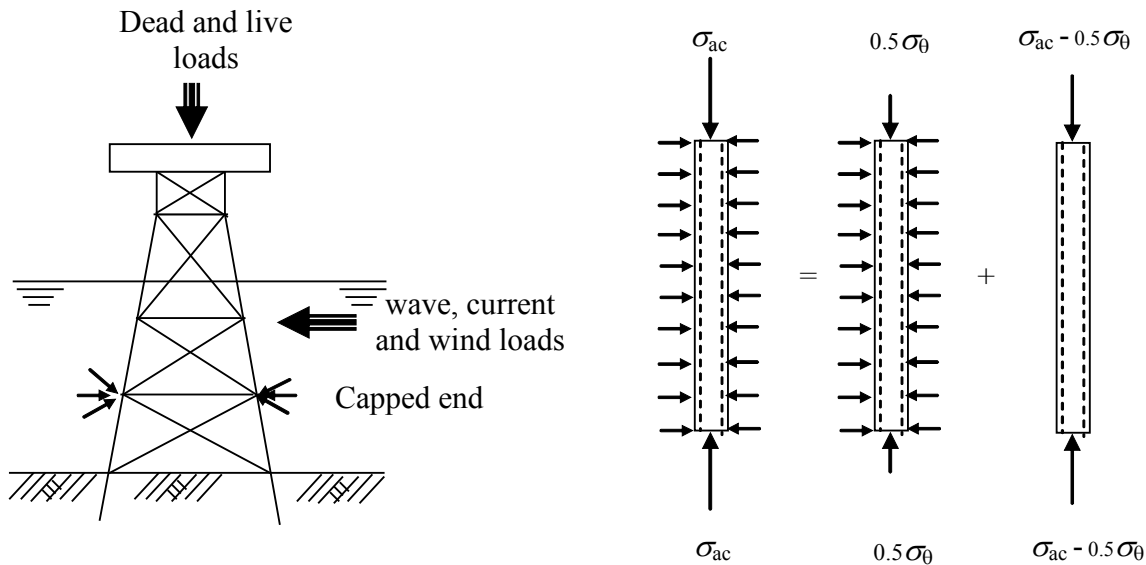
The member net axial stress is the calculated value, σ_{lc} , since the effect of the capped-end axial compression is explicitly included in the design analysis. Therefore, the calculated axial tensile stress, σ_{lc} , can be used directly in the cross-sectional strength check.

Test data for tubular members subjected to combined axial tension, bending, and hydrostatic pressure can be found in Miller et al^[35].

C7.3 Axial Compression, Bending Moment and Hydrostatic Pressure

In this method, the calculated axial stress, σ_{ac} , is the net axial compressive stress of the member since the capped-end axial compression is included in the design analysis. For the stability check, the axial compression to be used with the equation is the component that is in addition to the pure hydrostatic pressure condition (see Section C2, Figure 8). Therefore, the capped-end axial compression is subtracted from the net axial compressive stress. For the strength check, the net axial compressive stress is used. In addition, the cross-section elastic buckling criterion needs to be satisfied. The comparison between the ABS *Buckling Guide* and test data for tubular members subjected to combined axial compression, bending, and hydrostatic pressure can be found in Loh^[36,37].

FIGURE 8
Capped-end Action Arising from Hydrostatic Pressure



Tubular members subjected to combined compression, bending moment and hydrostatic pressure are to satisfy the following proposed equations at all cross-sections along their length.

When $\sigma_{ac}/\sigma_{CA\theta} > 0.15$ and $\sigma_{ac} > 0.5\sigma_{\theta}$:

$$\frac{\sigma_{ac} - 0.5\sigma_{\theta}}{\eta_1\sigma_{CA\theta}} + \frac{1}{\eta_2\sigma_{CB\theta}} \frac{C_m\sigma_b}{1 - (\sigma_{ac} - 0.5\sigma_{\theta})/(\eta_1\sigma_E)} \leq 1$$

When $\sigma_{ac}/\sigma_{CA\theta} \leq 0.15$:

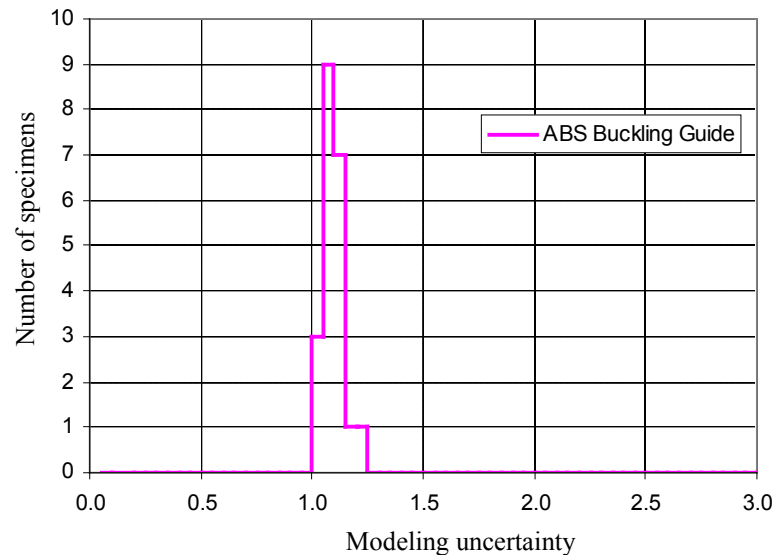
$$\frac{\sigma_{ac}}{\eta_1\sigma_{CA\theta}} + \frac{\sqrt{\sigma_{by}^2 + \sigma_{bz}^2}}{\eta_2\sigma_{CB\theta}} \leq 1$$

When $\sigma_x > 0.5\eta_1\sigma_{C\theta}$ and $\sigma_{Cx} > 0.5\sigma_{C\theta}$ the following local buckling state limit should also be satisfied:

$$\frac{\sigma_x - 0.5\eta_1\sigma_{C\theta}}{\eta_1(\sigma_{Cx} - 0.5\sigma_{C\theta})} + \left[\frac{\sigma_{\theta}}{\eta_1\sigma_{C\theta}} \right]^2 \leq 1$$

The comparison of the characteristic buckling strength from the above interaction equation to test data is shown in Section C2, Figure 9. This is based on test data for $D/t \leq 120$ from Das (2000)^[38] and includes both stiffened and unstiffened members under hydrostatic pressure or combined loadings. The mean and COV of the modeling uncertainty are 1.0299 and 13.28% respectively.

FIGURE 9
Local Buckling under Hydrostatic Pressure and Combined Loadings



C9 Local Buckling

For a member with a non-compact section, the local buckling may occur before the member as a whole becomes unstable or before the yield point of the material is reached. Such behavior is characterized by local distortions of the cross section of the member. When a detailed analysis is not available, the equations given as below may be used to evaluate the local buckling stress for the member with a non-compact section.

C9.1 Tubular Members in Axial Compression

Because of the absence of any ABS *MODU Rule* local buckling requirement, and the absence of an appropriate form of non-dimensionalisation for API RP 2A-WSD^[3] local buckling formulation, the formulation recommended for tubular members is,

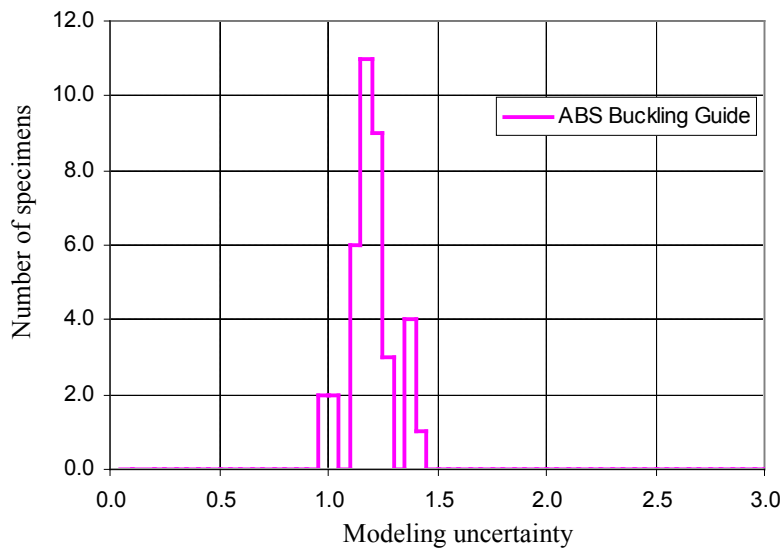
$$\sigma_{Cx} = \begin{cases} \sigma_{Ex} & \text{if } \sigma_{Ex} \leq P_r \sigma_0 \\ \sigma_0 \left[1 - P_r (1 - P_r) \frac{\sigma_0}{\sigma_{Ex}} \right] & \text{if } \sigma_{Ex} > P_r \sigma_0 \end{cases}$$

where

- P_r = proportional linear elastic limit of the member, which may be taken as 0.6 for steel
- σ_0 = specified minimum yield point
- σ_{Ex} = elastic buckling stress, given by $0.6Et/D$ for tubular members, to account for the inevitable effects of initial imperfections on perfect cylinder compressive buckling strength.

Comparisons for local buckling of tubular members under axial compression. are presented in Section C2, Figure 10 The local buckling database consists of 38 results performed by several investigators Chen and Ross^[17], Marzullo and Ostapenko^[39], Ostapenko and Grimm^[40], Prion and Birkemoe^[31], Eder et al^[41] and Kiziltug et al^[22]. Twenty five test results were rejected as being too thin and/or not adequately documented. The mean and COV were found to be 1.1449 and 8.89%, respectively.

FIGURE 10
Local Buckling for Tubular Members under Axial Compression



C9.3 Tubular Members Subjected to Bending Moment

<No Commentary>

C9.5 Tubular Members Subjected to Hydrostatic Pressure

<No Commentary>

C9.7 Plate Elements Subjected to Compression and Bending Moment

The critical local buckling of a member with W-shape, channel, T-section, rectangular hollow section or built-up sections may be taken as the smallest of local buckling stress of the plate elements. The local buckling stress of a plate element is obtained from the following equation with respect to uniaxial compression and in-plane bending moment:

$$\sigma_{Cx} = \begin{cases} \sigma_{Ex} & \text{if } \sigma_{Ex} \leq P_r \sigma_0 \\ \sigma_0 \left[1 - P_r (1 - P_r) \frac{\sigma_0}{\sigma_{Ex}} \right] & \text{if } \sigma_{Ex} > P_r \sigma_0 \end{cases}$$

where

σ_0 = specified minimum yield point

σ_{Ex} = elastic buckling stress, as given by:

$$= k_s \frac{\pi^2 E}{12(1 - \nu^2)} \left(\frac{t}{s} \right)^2$$

The buckling coefficient k_s for a plate element are specified based on the *ABS Steel Vessel Rules*^[2], Timoshenko and Gere^[15] and Galambos^[28].

SECTION C2 Appendix 1 – Examples of Buckling Analysis of Individual Structural Members

An MS EXCEL application, entitled, *ABS- Individual Structural Members*, has been developed to facilitate the use of the *ABS Buckling Guide*. There are three worksheets, namely “Input Data”, “Output Data” and “Intermediate Results”. In the worksheet “Input data”, the input data including the Member ID, Load case, Type of sectional shape, Geometries, Material parameters, Effective length factors, Moment reduction factors, Loadings and Basic utilization factor corresponding to the specified load case are provided by the user. Once the input data are entered, a macro represented by a large button “Structural Members” at the left-hand side corner is run. Buckling check results and intermediate results can be seen in the worksheets “Output” and “Intermediate Results”. All symbols used in the spreadsheet are consistent with those in the *ABS Buckling Guide*. Appendix C2A1, Table 1 shows several examples using tested tubular members.

TABLE 1
Examples Containing Detail Information for Tubular Members

Member ID			TEST-1	TEST-A3	TEST-2
Load Case			1	1	1
	Total length	L	5.49E+03	1.81E+03	1.57E+03
	Length between Transverse Frame or Diaphragms	ℓ	5486.40	1811.02	1567.00
	Section shape type		1	1	1
	Specified minimum yield stress	σ_0	271.03	337.93	216.20
	Modulus of elasticity	E	2.14E+05	2.00E+05	2.05E+05
	Poisson's ratio	ν	0.30	0.3	0.3
	Effective length factor along y-axis	k_y	1.00	1.00	1.00
	Effective length factor along z-axis	k_z	1.00	1.00	1.00
	Moment reduction factor around y-axis	C_{my}	1.00	1.00	1.00
	Moment reduction factor around z-axis	C_{mz}	1.00	1.00	1.00
Circular Tube (Shape type=1)					
	Outer diameter	D	381.00	458.47	114.30
	Thickness	t	8.03	16.54	3.51
Loading					
	Axial force	F_x	-2.47E+06	0.00E+00	-1.41E+05
	Bending moment about major axis	M_y	0.00E+00	1.20E+09	4.52E+06
	Bending moment about minor axis	M_z	0.00E+00	0.00E+00	0.00E+00
	External pressure	p	0.00	0.00	0.00
	Shear Force along major axis	F_y	0.00	0.00	0.00
	Shear Force along minor axis	F_z	0.00	0.00	0.00
	Torque	T	0	0.00	0.00
Maximum Allowable Strength Utilization Factor					
	Basic	η	1.00	1.00	1.00
Is the Section Compact?			Yes	Yes	Yes
Unity Check					
	Overall buckling		1.03	1.11	1.08
	Local buckling		N/A	N/A	N/A

Intermediate Results					
Geometry	Sectional Area	A	9404.78	22957.39	1221.68
	Moment of Inertia about y-axis	I_y	1.64E+08	5.61E+08	1.88E+06
	Sectional modulus about y-axis	SM_y	8.59E+05	2.45E+06	3.28E+04
	Moment of Inertia about z-axis	I_z	1.64E+08	5.61E+08	1.88E+06
	Sectional modulus about z-axis	SM_z	8.59E+05	2.45E+06	3.28E+04
	Radius of gyration about y-axis	r_y	131.90	156.36	39.19
	Radius of gyration about z-axis	r_z	131.90	156.36	39.19
	St. Venant torsional constant	K	3.27E+08	1.12E+09	3.75E+06
	Polar moment of inertia	I_o	3.27E+08	1.12E+09	3.75E+06
	Warping constant	G	0.00E+00	0.00E+00	0.00E+00
Stress	Normal stress due to axial force	σ_a	263.04	0.00	115.38
	Bending stress about y-axis	σ_{by}	0.00	489.60	137.60
	Bending stress about z-axis	σ_{bz}	0.00	0.00	0.00
	Shear stress along y-axis	τ_y	0.00	0.00	0.00
	Shear stress along z-axis	τ_z	0.00	0.00	0.00
	Torsional stress	t	0.00	0.00	0.00
Resistance					
	Euler buckling stress about y-axis	$\sigma_{(EC)y}$	1219.51	14713.53	1265.50
	Euler buckling stress about z-axis	$\sigma_{(EC)z}$	1219.51	14713.53	1265.50
	Elastic torsional buckling stress	$\sigma_{(ET)}$	8.22E+04	7.68E+04	7.88E+04
	Elastic local buckling stress	σ_{Ex}	2702.34	4327.98	3777.17
	Local axial buckling stress	σ_{Cx}	271.03	337.93	216.20
	Critical buckling stress about y-axis	σ_{CFy}	256.58	336.07	207.34
	Critical buckling stress about z-axis	σ_{CFz}	256.58	336.07	207.34
	Critical buckling stress about minor-axis	σ_{CF}	256.58	336.07	207.34
	Critical torsional buckling stress	σ_{CT}	270.82	337.57	216.06
	Critical buckling stress	σ_{CA}	256.58	336.07	207.34
	Critical bending stress about y-axis	σ_{CB_y}	341.45	442.07	287.30
	Critical bending stress about z-axis	$\sigma_{CB_{bz}}$	341.45	442.07	287.30
Unity Check for each individual load					
	Axial stress		1.03	0.00	0.56
	Bending stress about y-axis		0.00	1.11	0.48
	Bending stress about z-axis		0.00	0.00	0.00

Units: Length – [mm], Area – [mm²], Sectional Modulus – [mm³], Sectional Moment of Inertia – [mm⁴],
St. Venant torsional constant – [mm⁶], force – [N], Bending Moment – [N-mm], Stress and Pressure – [N/mm²]

SECTION **C3 Plates, Stiffened Panels and Corrugated Panels**

C1 General

Plates, stiffened panels and corrugated panels are used extensively in offshore structures. The panels form basic members for the deck structures, some pontoon structures, habitation units and watertight bulkheads and shear diaphragms. There is an extensive history of the application of plates, stiffened panels and corrugated panels as major and secondary components of thin-walled structures due to their easy construction and good structural efficiency measured by the ratio of strength to weight.

The various aspects of stiffened panels have been widely investigated, and the results of the design formulas have been compared to available test results. In general, the design formulas are based on a most comprehensive set of test results and other data for panels subjected to a variety of loading conditions. The predictions for strength resulting from these recommendations should be sufficiently accurate.

The process of buckling and ultimate strength assessment of stiffened panels is shown in the flow chart of Section C3, Figure 1. It consists of three parts, which are plate panels, stiffened panels, and girders and webs. The objective of the check performed in each step is explained in the flow chart. The procedure has been incorporated into an MS Excel application developed by OTD, ABS Technology.

C1.1 Geometry of Plates, Stiffened Panels and Corrugated Panels

A rectangular plate is a flat structure characterized by having its length and breadth dimensions very much greater than its thickness. It is usually supported in-plane around all four edges and may also be flexurally restrained.

The stiffeners themselves can act as single or multiple plates, depending on their geometry. They may buckle independently or in conjunction with the plate sections between the stiffeners. The precise analysis of a stiffened panel should reflect the capacity of the panel to act as a member in overall buckling between its outer boundaries, in addition to the possibility of the plates buckling locally between stiffeners.

Knowledge of the general behavior of plate panels subjected to the common forms of in-plane loadings is essential to understanding their response. The plate aspect ratio and the geometrical slenderness ratio are the two governing geometrical parameters in the buckling and ultimate strength assessment. These two parameters of plate panels in offshore structures may be different from those in other metal structures. In order to achieve an appropriate balance between accuracy and simplicity, a statistical description for the aspect ratio and slenderness ratio for the four types of floating production installations (FPIs) is carried out^[42]. The recorded values are the design values or so-called nominal values. The histograms of plate aspect ratio, ℓ/s , and the geometrical slenderness ratio, s/t , are shown in Section C3, Figures 2 and 3. The size of database for TLP, Spar and Semi-Submersible (Column Stabilized) is 8856, 7859 and 78639, respectively. The statistical data of FPSO is converted from Tankers, and the data source provided by Guedes Soares^[43] is used. The statistical characteristics are given in Section C3, Table 1.

FIGURE 1
Flow Chart of Buckling and Ultimate Strength Checks
for Plates, Stiffened Panels and Girders and Webs

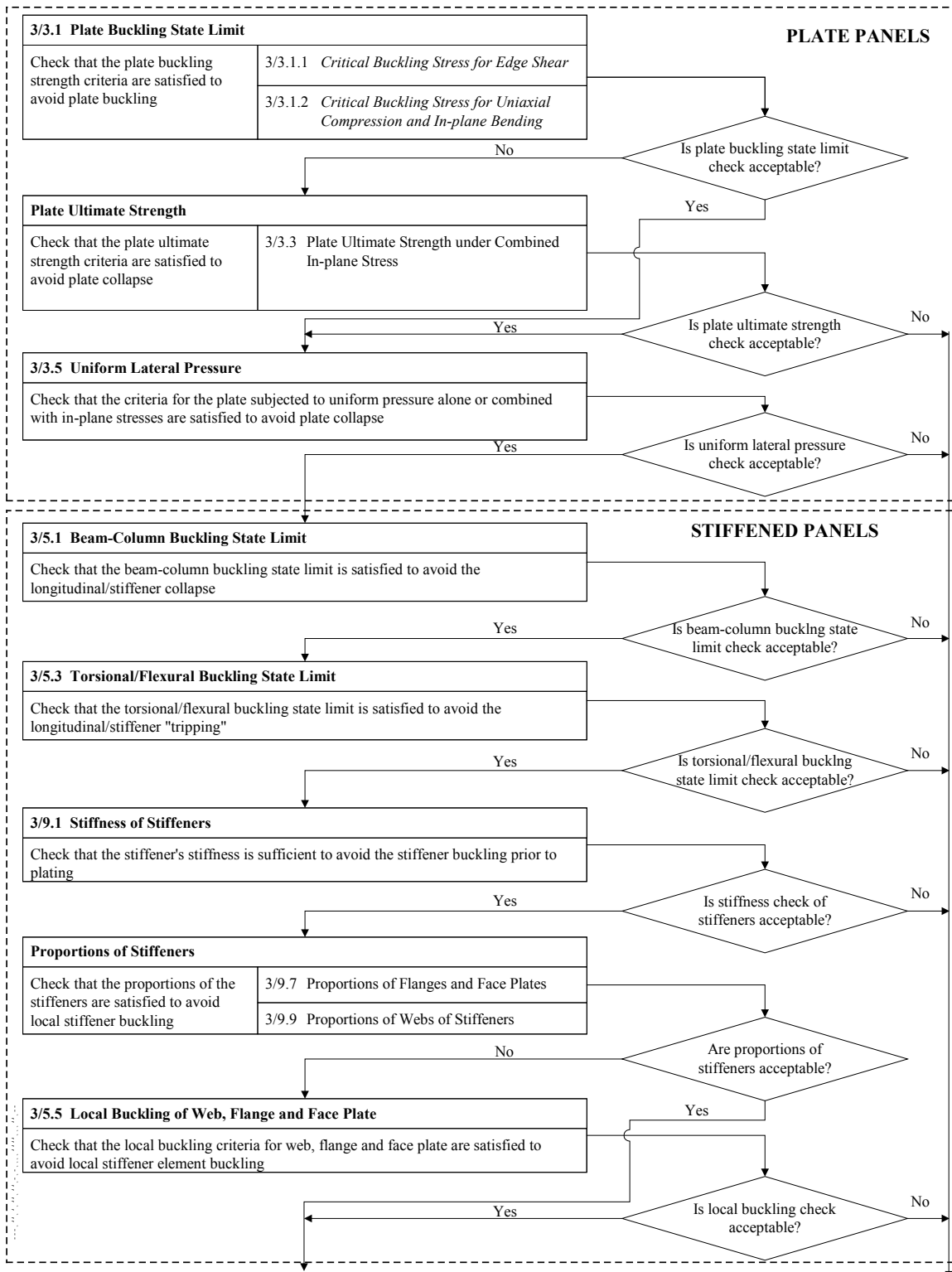


FIGURE 1 (continued)
Flow Chart of Buckling and Ultimate Strength Checks
for Plates, Stiffened Panels and Girders and Webs

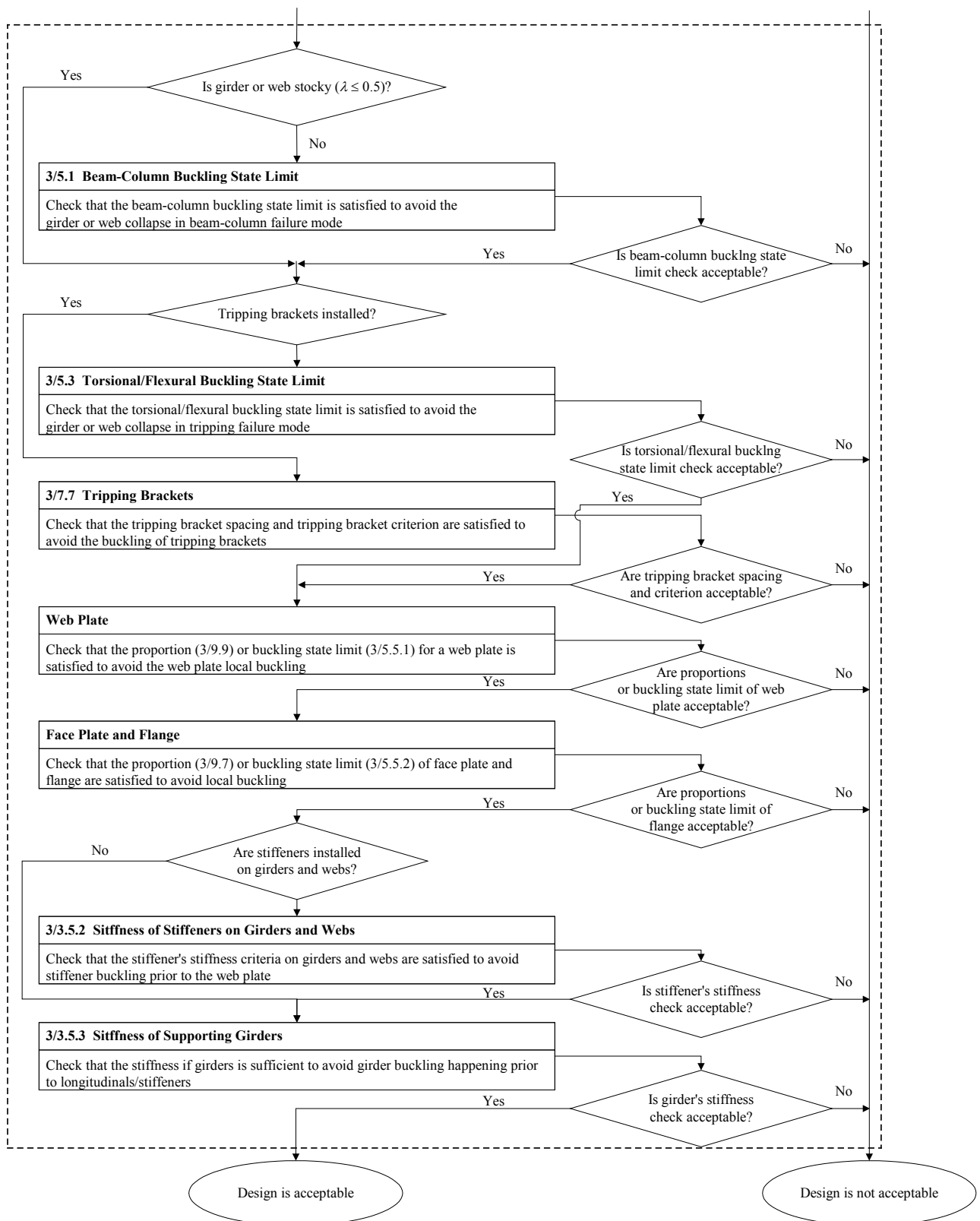


TABLE 1
Statistical Characteristics of Floating Production Installations (FPIs)

	<i>Aspect Ratio(α)</i>		<i>Slenderness Ratio(s/t)</i>	
	<i>Mean</i>	<i>COV</i>	<i>Mean</i>	<i>COV</i>
TLP	2.5	0.28	41	0.22
Spar	2.8	0.12	39	0.24
Semi	4.0	0.20	44	0.30
FPSO	4.7	0.16	46	0.25

According to design practice, the plate width (stiffener spacing) has relatively little variation. From Section C3, Table 1, it can be seen that the means of plate aspect ratio of TLP and Spar are very close, which implies that the plate length (transverse girder spacing) for these two types of structures is consistent. The COV of plate aspect ratio of TLP is greater than that of Spar, demonstrating the variation of girder spacing in a TLP is typically greater.

Semi-Submersible and FPSO have similar statistical characteristics on plate aspect ratio, implying that these two types of structures have the same design principles in the selection of a stiffening system.

All types of FPIs have identical statistical characteristics on plate slenderness ratio, implying that the longitudinal buckling and ultimate strength are in agreement with each other if the same steel grade is selected.

Corrugated panels in a watertight bulkhead are usually corrugated vertically without horizontal girders and supported by stools at upper and lower ends. Corrugated shear diaphragms typically omit the stool support structure. Based on experimental results, each corrugation of a corrugated panel deforms similarly under lateral pressure. This implies that the behavior of a unit corrugation can represent that of the entire corrugated panel when subjected to lateral pressure.

FIGURE 2
Statistical Depiction of Plate Aspect Ratio of FPIs

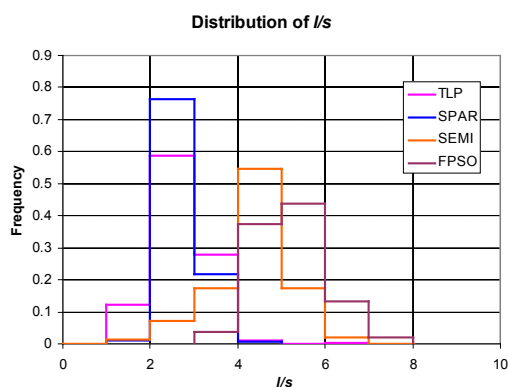
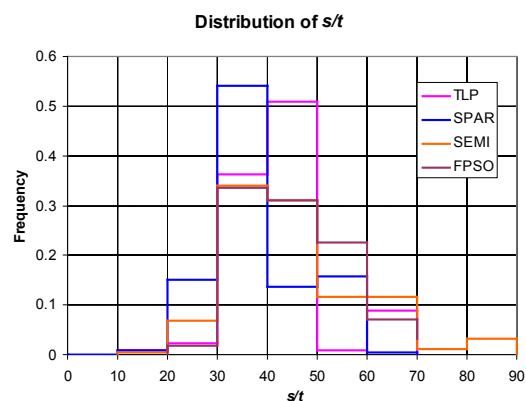


FIGURE 3
Statistical Depiction of Plate Slenderness Ratio of FPIs



C1.3 Load Application

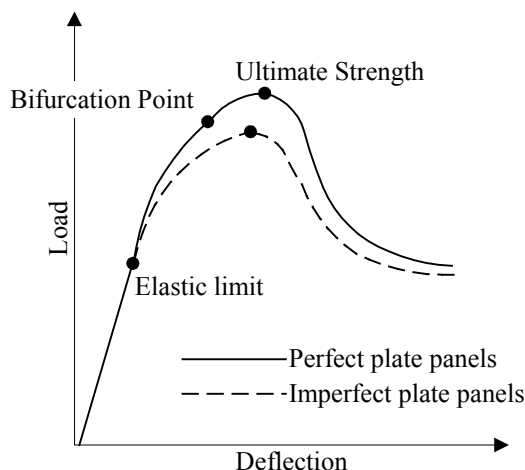
<No Commentary>

C1.5 Buckling Control Concepts

In the design of stiffened panels, one should keep in mind that there are three levels of failure mode, namely the plate level, the stiffened panel level and the entire grillage level; the higher level of failure usually leads more severe consequence than the preceding level. Therefore suitable scantling proportions between plates, stiffeners and girders are necessary to guarantee the sufficient safety of the stiffened panels.

Theoretically, plate panels exhibit a continued increase of resistance after the bifurcation point, before they finally reach the ultimate load carrying capacity. In other words, the plate panels have stable postbuckling behavior. Therefore, it is acceptable that plate panels are designed to reach the buckling state but not the ultimate state. The nominal load-deflection relationship of plate panels is shown schematically in Section C3, Figure 4.

FIGURE 4
Load-Deflection Relationship of Plate Panels



There are three types of buckling mode for stiffeners and girders, i.e., beam-column buckling, torsional-flexural buckling and local flange/web plate buckling. The buckling of stiffeners and girders is restricted because their resistance decreases quickly if any one of these three types of buckling occurs. Their buckling strength is regarded as the ultimate strength. If the associated plating of a stiffener buckles, but is below its ultimate state, the plating's effective width acting with the stiffener is to be applied.

Corrugated panels are 'self-stiffened' panels. There are three levels of failure mode, i.e., flange/web plate buckling, unit corrugation buckling and entire corrugation buckling. Depending on the loading type, corrugated panels may collapse into the different failure modes. For examples, axial compression mainly induces the flange/web buckling, lateral pressure induces the buckling of unit corrugation, and edge shear force leads to the buckling of entire panels. The buckling strength is the least value obtained from those established for the three failure modes considering any load type and load combination.

C1.7 Adjustment Factors

The adjustment factors for the allowable basic utilization factor in the existing offshore codes are provided in Section C3, Table 2.

TABLE 2
Adjustment Factor

	<i>Plate Panels</i>	<i>Stiffeners</i>	<i>Girders and Webs</i>	<i>Corrugated Panels</i>
<i>ABS Buckling Guide</i>	1.0	1.0	1.0	1.0
<i>API RP WSD 2A (AISC)^[3]</i>	1.0	1.0	1.0	—
<i>DnV MOU Rules^[13]</i>	1.1	1.0	1.0	—

DnV MOU Rules^[13] suggests a higher adjustment factor for plate panels, as the ultimate strength in DnV CN30.1^[7] is generally underestimated. The allowable ultimate strength requirements for plate panels from those three codes are very comparable, which will be demonstrated in the comparison study for a Spar in Section C3, Appendix 2.

C3 Plate Panels

C3.1 Buckling State Limit

A large amount of effort has been carried out over the past three decades and different interaction equations have been suggested. In the ABS *Buckling Guide*, the buckling state limit for plate panels between stiffeners is defined by the following equation:

$$\left(\frac{\sigma_{x \max}}{\eta \sigma_{Cx}}\right)^2 + \left(\frac{\sigma_{y \max}}{\eta \sigma_{Cy}}\right)^2 + \left(\frac{\tau}{\eta \tau_C}\right)^2 \leq 1$$

where

σ_{Cx}, σ_{Cy} = critical buckling stresses for compressions

τ_C = critical buckling stress for edge shear

The critical buckling stress for edge shear, τ_C , may be taken as (Johnson-Ostenfeld formula):

$$\tau_C = \begin{cases} \tau_E & \text{for } \tau_E \leq P_r \tau_o \\ \tau_o \left[1 - P_r (1 - P_r) \frac{\tau_o}{\tau_E} \right] & \text{for } \tau_E > P_r \tau_o \end{cases}$$

In the above equations, the elastic buckling stress is:

$$\tau_E = k_s \frac{\pi^2 E}{12(1-\nu^2)} \left(\frac{t}{s}\right)^2$$

where

k_s = boundary dependent constant.

$$= \left[4 \left(\frac{s}{\ell} \right)^2 + 5.34 \right] C_1$$

The critical buckling stress in a given direction, σ_{Ci} ($i = x$ or y), for plates subjected to combined uniaxial compression and in-plane bending may be taken as (Johnson-Ostenfeld formula):

$$\sigma_{Ci} = \begin{cases} \sigma_{EI} & \text{for } \sigma_{EI} \leq P_r \sigma_0 \\ \sigma_0 \left[1 - P_r (1 - P_r) \frac{\sigma_0}{\sigma_{EI}} \right] & \text{for } \sigma_{EI} > P_r \sigma_0 \end{cases}$$

The elastic buckling stress is given by

$$\sigma_E = k_s \frac{\pi^2 E}{12(1-\nu^2)} \left(\frac{t}{s}\right)^2$$

where

k_s = buckling coefficient

For loading applied along the short edge of the plating (long plate):

$$k_s = C_1 \begin{cases} \frac{8.4}{\kappa + 1.1} & \text{for } 0 \leq \kappa \leq 1 \\ 7.6 - 6.4\kappa + 10\kappa^2 & \text{for } -1 \leq \kappa < 0 \end{cases}$$

For loading applied along the long edge of the plating (wide plate)

$$k_s = C_2 \begin{cases} \left[1.0875 \cdot \left(1 + \frac{1}{\alpha^2} \right)^2 - 18 \frac{1}{\alpha^2} \right] \cdot (1 + \kappa) + 24 \frac{1}{\alpha^2} & \text{for } \kappa < \frac{1}{3} \text{ and } 1 \leq \alpha \leq 2 \\ \left[1.0875 \cdot \left(1 + \frac{1}{\alpha^2} \right)^2 - 9 \frac{1}{\alpha} \right] \cdot (1 + \kappa) + 12 \frac{1}{\alpha} & \text{for } \kappa < \frac{1}{3} \text{ and } \alpha > 2 \\ \left(1 + \frac{1}{\alpha^2} \right)^2 (1.675 - 0.675\kappa) & \text{for } \kappa \geq \frac{1}{3} \end{cases}$$

It should be noted that the formulation for k_s for wide plates is basically similar to K_t in 5-1-A2/3 of the ABS *Steel Vessel Rules*^[2]. The difference is that ABS provides three points between which the designer is to use linear interpolation, while the formulation given above relies on a continuous curve.

Comparison Study – Buckling Strength

Test Database

The plate test database consists of 206 datasets for plates under uniaxial compression along short edges and 15 datasets for plates under uniaxial compression along long edges. The data were collected from numerous publications. No records related to plates under edge shear loading were included in the database.

Comparison Basis

Comparisons between the strength predictions of the existing formulations and test data are generally conducted on the basis of safety factors excluded (set to unity). In all cases, measured values of geometry and material properties were input to the strength equations. For single-acting loading conditions such as pure compression, the resultant strength was taken as the predicted value for the model in question. For assessing accuracy in terms of ‘Modeling Uncertainty’, as defined in Section 1, the test result is then divided by the value from the strength formulations. The average of all of these modeling uncertainty values is then calculated along with the standard deviation in order that the COV (coefficient of variation) can be determined as the ratio of (standard deviation) ÷ (average). The average (or mean or bias as it is more usually referred to) and COV provide the statistics by which the accuracy of the formulation can be quantified.

Comparison Results

- **Long Plates.** Section C3, Figure 5 shows the relation between $\sigma_{C\kappa}$ and β for the long plates. Section C3, Figure 6 shows the distribution of modeling uncertainty. Section C3, Table 3 gives the statistical characteristics of the modeling uncertainty.

TABLE 3
Mean/COV of Modeling Uncertainty of Long Plates

	ABS Buckling Guide	DnV CN30.1	API Bulletin 2V
Mean	1.1149	1.1731	1.0747
COV	0.2845	0.2723	0.3050

FIGURE 5
Buckling Stress of Long Plates

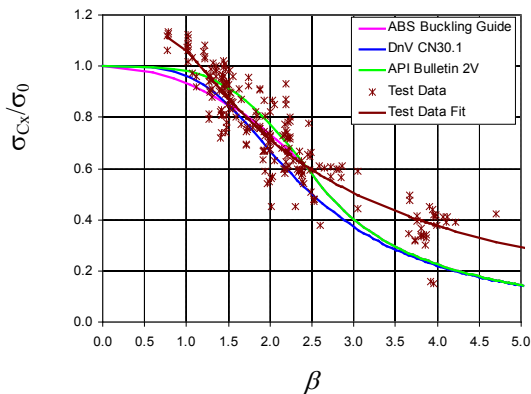
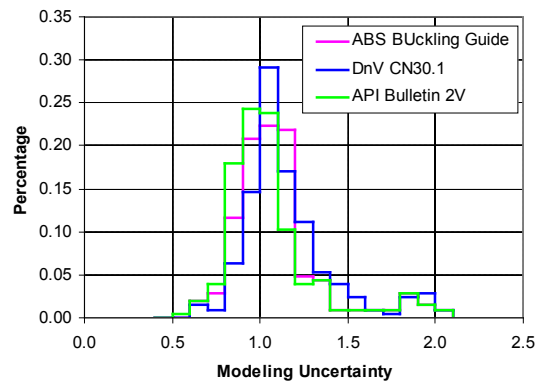


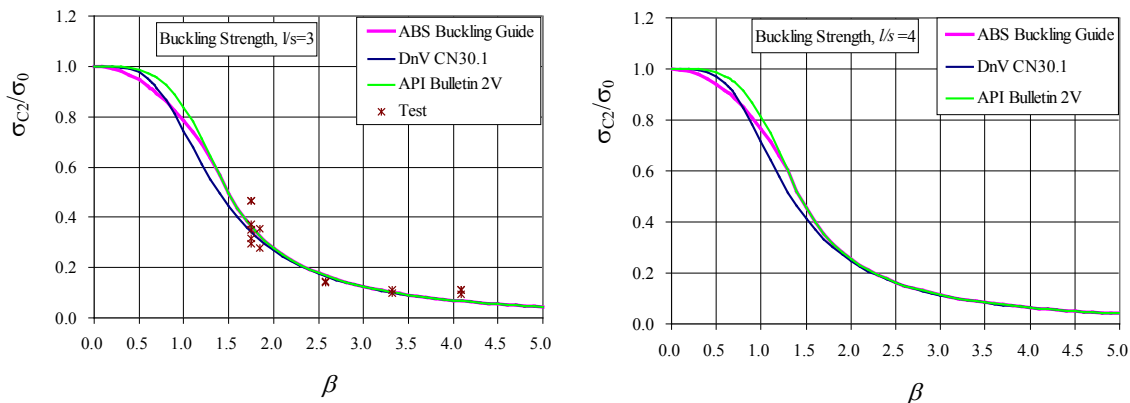
FIGURE 6
Modeling Uncertainty of Long Plates



The mean of the modeling uncertainty in the three codes is greater than 1 and its COV is around 30%. The relatively large COV is not surprising because the test data have been collected from many different published sources. In general, the buckling strength evaluated by the three codes is conservative compared to the test data.

- *Wide Plates.* Section C3, Figure 7 shows graphically the relation between σ_{Cy} and β for the wide plates for two different aspect ratios according to the formulae of the *ABS Buckling Guide*, DnV CN30.1, API Bulletin 2V and test data. The results obtained from the formulae of the three codes are in good agreement with test data.

FIGURE 7
Buckling Stress of Wide Plates



Section C3, Figure 8 shows the comparison of buckling coefficient for wide plates subjected to non-uniform compression on the long edges. The buckling coefficient value obtained from the *ABS Buckling Guide* is somewhat different from those from API Bulletin 2V and DnV CN 30.1 when the plate aspect ratio is less than 3 and the stress ratio is negative.