# Report of Committee V.1
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## APPLIED DESIGN – STRENGTH LIMIT STATES FORMULATIONS

Concern for the applicability of formulations related to the ultimate, buckling and the fatigue strength of structural components of ships, offshore platforms, and other marine structures. Emphasis shall be given to comparisons with available experimental data compiled in terms of probabilistic measures to evaluate the accuracy of these formulations for use in reliability-based design procedures.


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## 1. INTRODUCTION

Marine structures are designed to fulfill serviceability (SLS) and ultimate limit state (ULS) criteria. The latter category in general comprises criteria for structural integrity, and may be further classified in terms of /1/:

- ultimate limit state of components
- ultimate limit state of systems
- fatigue limit state of components

The ultimate limit may be characterized by loss of equilibrium, fracture or excessive deformation. The fatigue limit state is defined by a crack size that represents an ultimate limit /1, 2/.

Marine metal structures are made of beams, stiffened panels, shells, and joints. Ultimate limit states may be expressed by parametric formula, numerical calculation procedures (e.g., nonlinear finite elements), or by testing. Existing component ultimate limit states are generally given by parametric formulae in terms of stresses, forces or moments; and exceptionally by strains, which have been justified by test results. Fatigue criteria in design codes are typically expressed by SN curves. Numerical approaches usually need to be used in connection with systems capacity. Design by testing (alone) may be used in special cases of innovative design.

There are significant differences between formulae used in different codes for the same failure mode. Hence, there is a need to compare and harmonize strength expressions, to identify the "best" one. Efforts to harmonize requirements for ships are made through IACS; and for offshore structures through ISO. There is a need to harmonize ship and offshore strength requirements, especially for steel-plated components.

The purpose of this report is to review ultimate and fatigue limit states for steel components in marine structures. The focus is on formulations of beam-columns, stiffened plates and shells and planar and tubular joints given in terms of parametric equations. Approaches specified as a numerical procedure will be briefly commented upon, in cases where they seem to offer significant improvement compared to parametric formulae.

The mathematical expressions for the limit states will be used as a basis for:

- failure function for reliability analysis. In this case the parameters in the limit state are considered to be random variables.
- design equations. In this case the parameters in the limit state - according to the partial factor method - are given by their characteristic values, each multiplied by a partial safety factor.

However, for a given phenomenon, the same limit s formulation may not necessarily be applied for the de equation and the failure function; but a more optimal calibration is achieved by choosing the same limit state.

A particular issue of concern in connection with buo marine structures is the definition of ultimate limit state view of conditions where large deformations/strains cause l failure/fracture and, hence, possible leakage. "W tightness" may in principle be considered as a separate l state, which may imply a lower limit on deformations dictated by the pure load carrying capacity.

Generally, several formulations are applied for diffe limit states. Significant efforts would be required to com existing formulations to establish the "best" approach. committee has attempted to collect available information selected failure modes that could be considered as a mo contribution to future efforts of improving the correspon limit states.

Clearly, existing codes for marine and civil enginee structures should form the basis for the assessment of l states. Recent systematic experimental or numerical analy as reviewed by other ISSC committees, are useful as a basis code modification. Due to space limitation, only sele references can be quoted by this committee.

Ship rules have traditionally represented prescrip criteria for scantlings, which reflect strength and fatigue other considerations. So-called direct calculation methods ships are usually based on explicit loads and stre formulae. However, these formulations are often based on f yielding and elastic buckling with a simple correction plasticity, i.e., not truly ultimate limit states. For lac space, such methods will only briefly be touched upon her On the other hand, explicit fatigue criteria that have rece been introduced for ships, are considered.

The current availability of accurate numerical met allows determination of the strength on a case basis as wel to accomplish systematic studies of complex structures. particular advantage of numerical methods is that the effec each factor can be studied separately. However, numer methods to trace nonlinear behavior up to collapse susceptible to various sources of uncertainty, including h errors of different kinds. It is, therefore, necessar standardize the relevant numerical analysis procedures. It even be necessary to "certify users" and validate res before such analyses can be used by designers at larg determine resistances.

Up to now, ultimate strength formulations in codes been established primarily on the basis of simple struct mechanics models, with appropriate "knock down" fac determined experimentally. Relatively simple and accu formulations are achievable when failure mode follow simple mechanism (buckling, yield hinge mechanism, etc.).

achieve such simple failure modes, e.g., for complex stiffened, thin walled plates and shells, the stiffeners are considered to meet certain assumptions, which commonly are conservative.

This review refers to "strength of materials" type design formulations supported by experiments or systematic numerical studies. The main source of information for the limit states of concern herein are codes issued by the industry, governmental regulatory bodies and classification societies. Review of recent developments of analytical and numerical techniques as well as experimental methods as such, is outside the scope of this report. The aim is rather to address some possible directions of further code developments. To come up with a new code, is certainly beyond the scope of this ISSC committee.

Chapter 2 deals with various formats for limit states and a consistent assessment of the inherent uncertainties.

Chapters 3-6 present comparisons between limit states used for the different types of components.

A summary and recommendations for future work are given in Chapter 7.

## 2 LIMIT STATES AND UNCERTAINTY MODELLING

### 2.1 Limit state formats

Limit states should /6,7/:

- preferably be based on structural mechanics theoret formulations (not pure regression to data)
- explicitly contain the parameters of influence
- have as small (random) model uncertainty as possible
- be as simple as possible

Limit states should, as far as possible, be basec strength of materials, elastic buckling formulae, plastic l loads, etc., appropriately modified to account for resi welding stresses and geometrical imperfections, and should be obtained by a pure regression analysis of test results. would also be advantageous that a formulation converges say, elastic buckling formulae and yield stress for slender and stocky components, respectively. Besides nominal dimensions of the components, the stress-strain cu geometrical imperfections and residual stresses are parame of influence. Only the yield stress, and not the stress-st curve, is commonly referred to in resistance formulati Maximum geometrical deviations are usually assumed developing the strength limit states; and toler requirements are imposed in design to ensure that they complied with during fabrication. The residual stress pat depends upon the fabrication, and is generally not explic accounted for.

Limit states are subject to a systematic and ra uncertainty. As discussed below, the systematic componen accounted for directly, while the random component impli partial safety (material) factor.

Simplicity in the design formulation implies a rec chance of gross errors.

### 2.2 Model uncertainty

For a structural component subject to one load (i.e., plane compression, or lateral pressure), $s_i$ at a time, predicted resistance, $r_{pred.,i}(\underline{z})$ is a function of a set, variables, which may include yield strength, plate thickn geometrical imperfection, etc. Usually these parameter vary over the component, and each type of parameter represented by its value at a few locations, assuming that variation follows some pattern over the structure. Typica out of plane imperfections may be described by Fou components /3/. The yield stress is different in diffe directions, and in compression versus tension, and depend strain rate. It is important that the model uncertaint assessed using the same definition of yield stress as use design analyses. The resistance model is uncertain. By characterizing this model uncertainty by $\Delta_i$, the actual resistance, $r_{true,i}$ corresponding to a load $s_i$, may be written as /4/:

$$r_{true,i} = \Delta_i r_{pred.,i}(\underline{z}) \tag{1}$$

The model uncertainty, $\Delta_i$ is assessed by comparing predictions with results obtained by experiments or advanced numerical analyses, most often by assuming that the error in experimental or numerical calculations is negligible. The predictions are made by using a representative value of the parameters in the set, $\underline{z}_j$ corresponding to the specimen no. j. The corresponding sample of $\Delta_i$ is:

$$\Delta_i^{(j)} = r_{(test)}^{(j)} / r_{(pred)}^{(j)}\left(\underline{z}_j\right) \tag{2}$$

The statistics of $\Delta_i$ may then be calculated based on a sample $\left\{\Delta_i^{(j)}\right\}$. Usually limited information is available, and $\Delta_i$ is simply modelled by a normal or lognormal distribution.

In some cases, it may be interesting to determine the model uncertainty $\Delta_i$ as a function of some (geometrical or other) parameters. Hence, the uncertainty for beam-columns may firstly be determined for each type of member cross-section. Secondly, for a given type of component under compressive stresses, it may be convenient to determine $\Delta_i$, as a function of slenderness, because the uncertainty will depend upon whether it is buckling or plastic collapse that leads the component to reach its ultimate limit state. Such information may be useful in the first place to assess whether a trend in the bias may warrant a modified strength formulation, and secondly whether the partial safety factor should depend upon this parameter (e.g., slenderness).

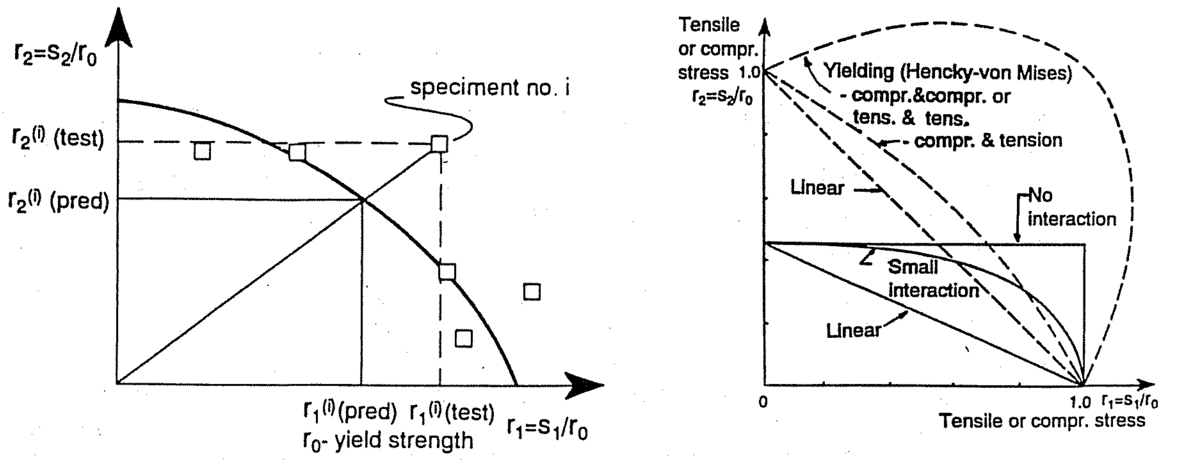
It follows from the above assumptions, that before applying experimental or numerical data to assess the uncertainty, they should be critically reviewed to eliminate data influenced by "gross" errors, or experimental data which involve incompletely defined tests. This is a major task in an uncertainty assessment.

Model uncertainties for the ultimate strength of components which are subject to two or more load components also need to be described. In general, the interaction equation may be written as:

$$g\left(\left\{\frac{s_i}{r_i}\right\}, \{c_k\}\right) = 0 \tag{3}$$

where $s_i$ and $r_i$ are load effects and resistances corresponding to load component no. (i), $\{c_k\}$ is a set of coefficients used

in the model. The interaction for the case of two loads
illustrated in Fig. 1.




(a) Experimental vs. predicted values

(b) Typical interaction equati

Fig. 1 Strength interaction for two action effects

The model uncertainty of the interaction equation sho
be expressed so that it represents the model uncertainty
each basic load case as a special case when only one load,
is acting. This means that each $r_i$ should be modified with
which is determined for each $r_i$ as explained above.
addition, uncertainties, $\Delta_k$ need to be associated with
other parameters, $\{c_k\}$ in the interaction formula, Eq. (
e.g.:

$$g\left(\{s_i/\Delta_i r_i\}, \{\Delta_k c_k\}\right) = 0$$

If, for instance, the basic interaction equation consi
of terms like $(s_i/r_i)^{\alpha_i}$, terms of the type:

$$\Delta_k c_k \left(s_i/r_i\right)^{\alpha_i} \left(s_j/r_j\right)^{\alpha_j}$$

may be introduced to account for uncertainties of
interaction. A convenient way to characterize the uncertai
for the case of two loads (Fig. 1a) is by:

$$\Delta_{12}^{(j)} = r_{1(test)}^{(j)} / r_{1(pred)}^{(j)} = r_{2(test)}^{(j)} / r_{2(pred)}^{(j)} \qquad ($$

where $r_{i(test)}^{(j)}$ is the experimental resistance corresponding
the load i. Then, it needs to be demonstrated by a cert
choice of $\Delta_k$, that Eq. (4) implies the same uncertainty
expressed by $\Delta_{12}$ /7/. If the discrepancy is unacceptable,
improved interaction equation should be introduced.

If, however, interaction between, e.g., two loads $s_1$

limited to the range 0 to $\bar{s}_2/r_2$, the interaction equation may be expressed in terms of $s_1/r_1$, and the model uncertainty, $\Delta_1$ may be calculated by:

$$\Delta_1 = r^{(j)}_{1(test)}/r^{(j)}_{1(pred)} \tag{5b}$$

In this case (all) the uncertainty $\Delta_1$ should be assigned to the variable $r_1$, also for the case when $s_2 > 0$. Clearly, this approach will not be particularly accurate when $s_2$ varies from 0 to $r_2$, which implies a variation between a pure $s_1$ and a pure $s_2$ load condition.

An analogous interaction problem exists for fatigue, both in relation to crack initiation and crack growth, under multiaxial stress conditions. While the equivalent (plastic) strain would be a representative measure for crack initiation, tensile stresses normal to crack and shear stresses are the relevant stresses for crack growth. However, the normal stress usually dominates and the shear contribution is rarely considered for marine structures.

## 2.3 Model, parameter and total uncertainty

The resistance is influenced, for instance, by overall scantlings (a,b,...), plate thickness (t), initial imperfection ($\delta$), modulus of elasticity (E), yield stress ($\sigma_o$) and residual stress ($\sigma_r$). Representative values of the parameters a, t, and $\sigma_o$ are commonly explicitly accounted for in ultimate resistance formulae. Values of initial imperfections and residual stresses are often only implicitly assumed when establishing the resistance formula. Imperfections ($\delta$) are, however, controlled by tolerance requirements. Residual stresses, $\sigma_r$ are treated explicitly, e.g., in connection with stiffened cylinders.

When, for instance, initial imperfections are not explicitly considered in $r_{pred}(\underline{z})$, and the variation of imperfections in real structural components is not the same as in the test specimens used to estimate $\Delta_i$, the model uncertainty $\Delta_i$ needs to be modified to be considered representative for real components, /7/.

If the sample of specimens used to determine $\Delta_i$, is small, the statistical uncertainty should be combined with the model uncertainty using Bayesian methods /5/.

However, in some cases, the experimental/numerical data basis is applied in the first place to fit the theoretical model to the data. In that case, the model uncertainty $\Delta_i$ determined by the above procedure, obviously does not reflect the true model uncertainty, and $\Delta_i$ needs to be (subjectively) adjusted.

Finally, the relevance of the material, geometry and boundary as well as load conditions used in the experimental/numerical investigation to determine the model

necessary to modify $\Delta_i$ for a particular application of
prediction method, $r_{pred}$.

## 2.4 Characteristic and design values of resistance

Uncertainties should be handled by probabilistic metho
If a simple design criterion in terms of a single resistan
$R_i$ and load effect, $S_i$ is considered, and, e.g., a lognor
probability distribution is assumed for $R_i$ and $S_i$, the des
values of $R_i$ and $S_i$ can be separated, and the design value
$R_i$ becomes /1/:

$$r_{id} \cong \mu_{r_i} \exp\left[-\alpha_{r_i} \cdot \beta_T \cdot V_{r_i}\right]$$

where $\mu_{r_i}$ and $V_{r_i}$ are the mean and COV of $R_i$, respectively.
and $\beta_T$ are the so-called sensitivity factors and the tar
reliability index, respectively. $\alpha_{r_i}$ depends upon the relat
magnitude of the uncertainties in $R_i$ and $S_i$, and, var
between 0.2 and 1.0. $\alpha_{r_i}$ will typically be smaller for slen
members in bottom supported platforms subjected to wave lo
than for panel and shell components in large volume mar
structures, or structures subjected predominantly
hydrostatic pressure, because of the relative magnitude of l
and resistance uncertainties. In the design value method
which yields a simplified code calibration, $\alpha_{r_i}$ is taken to
0.8.

The characteristic value for a lognormal variable, $R_i$
be defined by means of $\mu_{r_i}$ and $V_{r_i}$ as:

$$r_{ik} \cong \mu_{r_i} \exp\left[-k_{r_i} V_{r_i}\right] \quad \text{for } V_{r_i} \leq 0.2$$

where $k_{r_i}$ is a constant which is selected so that $r_{ik}$ is defi
according to a certain fractile value, and is typically of
order 0.5 to 1.5.

The partial resistance factor, $\gamma_{Ri}$ may be defined by:

$$\gamma_{Ri} = \frac{r_{ik}}{r_{id}} \cong \exp\left[\left(\alpha_{r_i}\beta_T - k_{r_i}\right)V_{r_i}\right]$$

Hence, with $k_{r_i} = 1.5$, $\alpha_{r_i} = 0.5$ and a target safety le
corresponding to $\beta_T = 4.2$, $\gamma_{Ri} \sim 1 + 0.6V_{r_i}$.

Eq. (8) applies if the design resistance $r_{id}$ is base
Eq. (1), e.g., corrected due to $\Delta_i$. If the design is base
$r_{pred.}$, the safety factor to achieve a target reliability
would be $\gamma_{Ri}/\mu_\Delta$, where $\mu_\Delta$ is the mean $\Delta_i$. This shows
important influence of the bias in the resistance.

# 3. ULTIMATE LIMIT STATES FOR STEEL-PLATED COMPONENTS

## 3.1 Introduction

Basically two main types of steel-plated components are applied in marine structures (Fig. 2):

- stiffened panels subjected to predominantly in-plane axial stresses, but with some in-plane transverse and shear stresses and possibly lateral pressure
- stiffened girders subjected primarily to in-plane shear and longitudinal axial and bending stresses

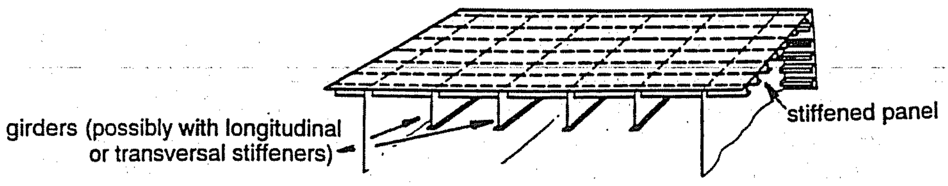



Fig. 2 Stiffened panels/girders

The collapse behavior of stiffened panels and girders received considerable attention in the 1970's and 1980's, especially in connection with several box girder bridge failures, culminating in the codes such as BS 5400 /9/ and EC3/ECCS /13,14/. Much of this work has been focused on modern limit state methods.

While the civil engineering developments of ultimate limit states for stiffened panels for bridges had some influence on marine practice, the significant efforts on stiffened girders in bridges have so far had less effect on the design practice for similar components in marine practice.

The disadvantage with EC3/EECS, BS5400 and other codes primarily developed for civil engineering structures is that they do not cover geometries and load conditions (e.g., in-plane and lateral pressure) typical of marine structures. Ship rules of classification societies are based on elastic/plastic buckling/first yield criteria, but since the formulations are implicit in scantling requirements, they are not pursued herein. Especially, the involvement of some classification societies in structures for offshore oil exploitation, which initially was heavily influenced by civil engineering practice, has made them introduce limit state formulations. Det norske Veritas has introduced a comprehensive guidance /11/, while Bureau Veritas /10/, Germanischer Lloyd (GL) /15/ and other societies follow. GL, for instance, base their recent (offshore) codes on Eurocode 3 (ECCS) and DIN 18800. Besides classification societies, governmental regulatory bodies such as HSE /16/ and NPD /17/ in the UK and Norway, respectively; and API (in the USA) issue codes for offshore structures.

Recent state-of-the-art reports on steel-plated structu /18-21/ refer primarily to civil engineering components, are partly relevant to marine structures.

However, the research goes on in this area, as reportec ISSC Committee III.1, generally by using experimental numerical methods. In this report, the focus will be on us data obtained by systematic numerical or experimental stuc to validate strength codes based on parametric formulae. publications deal with the uncertainties in the strength. this reason, this committee launched a benchmark study; /22/ and the brief summary in Section 3.5. The benchmark st was also done to assess the robustness of the formulations calculations, with respect to human errors.

Stiffened panels may exhibit the following failure moc depending upon the geometry and material property:

- local buckling or collapse of plate between stiffeners
- local buckling or collapse of stiffeners. The basic des philosophy is to avoid such failure modes by slenderr requirements
- overall buckling of stiffened panel between frames bulkheads

Limit states for the first failure mode are discussec Section 3.2. Overall collapse of stiffened panels under ax and possible lateral loads (bending moments) is treated Sections 3.3 and 3.4, while the benchmark test for such par is described in Section 3.5. The overall collapse of stiffe girders subjected to in-plane bending and shear is discussec Sections 3.6 and 3.7.

## 3.2 Plate elements

### 3.2.1 Formulations

The basic component is a plate element subjected to plane or lateral loads.

In modern limit state approaches the design principle to assume that the plate element reaches its collapse l: before overall stiffener failure. Estimates of the ultin limit capacity of plate elements are therefore crucial.

Significant efforts have been devoted to improving original Von Karman estimate of the ultimate strength axially loaded plate. Various in-plane and flexural bounc conditions, and imperfections and residual stresses need to considered. Recent reviews or the various formulations been made in /18,23-29/. Some of the discrepancies bet different specifications may be explained by diffe assumptions regarding initial imperfection, boundary condi and residual stress levels, which are all factors of influe

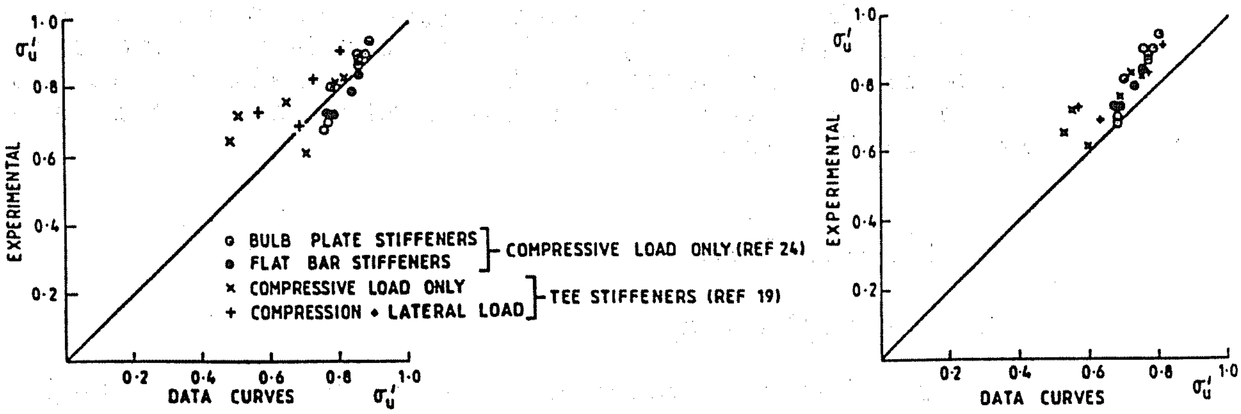
In particular, it is noted that the different codes specify different level of acceptable imperfection tolerance!

For instance, the tolerance requirement for initial imperfection of plating varies, and the imperfection amplitude to plate width is, e.g., $w_o/b < 0.01$ (API Bul. 2V /8/, DnV /11/, 1987), $w_o/b \leq 0.002$ (ECCS /13/). According to BS 5400 /9/, it varies with a/b and $\sigma_o$, but is approximately $w_o/b \lesssim 0.01$. The allowable initial deflection of stiffeners is in the range 1/750 of the span (BS 5400) to 1/500 (ECCS). Considering moderate imperfections ($w/t = 0.1\beta^2$, where $\beta$ is normalized plate slenderness) and compressive residual stresses ($0.15\sigma_o$), the following ultimate to yield strength, $\sigma_u/\sigma_o$ was proposed in /23,28/:

$$\sigma_u/\sigma_o = 0.23 + 1.16/\beta - 0.48/\beta^2 + 0.09/\beta^3 \qquad (9)$$

which slightly deviates from previously proposed formulae.

An assessment of the sensitivity of $\sigma_u$ to imperfections is given in /26/. Experimental results are compared with numerical results using moderate imperfections ($w_o/t = 0.1\beta^2$) in real plates as well as numerical results using actual imperfections of the specimens; see Fig. 3. The effect of residual stresses and imperfections was also studied in /29,30/.




(a) Average imperfection (b) Actual imperfection

Fig. 3 Comparison of numerical and experimental results /20/. Numerical predictions using average or actual imperfection.

An interaction formula for biaxial compression was proposed in /24,27/, which converges to the von Mises-Hencky and linear interaction for stocky and very slender plates, respectively. Also, an interaction curve for biaxial compression and shear is given in /24,27/. Based on ultimate limit state formulation, it is concluded for the shear strength in /24/ that the DnV curves are rational, but generally

conservative and the LR rules are too simplistic and lea
varying levels if the same safety factor is applied.

A special phenomenon is observed for slender p
elements (e.g., $\beta \gtrsim 3$) supported by strong stiffeners
subjected to shear loading, in that a tension fiel
developed and, hence, leads to significant capacity be
linear shear buckling (see Section 3.7).

Concerning lateral pressure combined with bia
compression, interaction curves are provided in /25/
particular imperfections and a range of aspect rat
slenderness and pressures. These curves are translated
parametric formulations to give the variation of the stre
with the lateral pressure. This model predicts that mode
pressure has a much greater effect on compressive resist
than suggested by current design rules.

### 3.2.2 Uncertainty measures

The uncertainty of predicted ultimate strength (effe
width) of plating can be assessed based on the re
displayed in Fig. 3 /26/. It is seen that the smallest
defined according to Eq. (2) (of 1.05) is achieved
predictions are made on the basis of average imperfect
The scatter then corresponds to a COV of 13%. If the a
imperfection is used, the bias surprisingly increases to
but the COV is reduced to 7%.

## 3.3 Stiffened panels subject to axial compression

### 3.3.1 Formulations

The strength of stiffened panels under axial compre
may be treated as

- a series of disconnected beams
- an orthotropic plate
- a discretely stiffened plate

For most (wide) stiffened panels, the "column" appr
considering flexural collapse of a single stiffener wit
adjacent plate, is adopted. Local buckling of the stiff
is avoided by assuming that their geometry fulfills ce
local slenderness constraints. Generally, two diff
"column" approaches are applied. The first approach is
on reduced slenderness concept with a strength express
means of the Euler buckling load, modified for elasto-pl
behavior by the Johnson-Ostenfeld parabola (API Bul. 2V,
ship rules). The second method is the Perry approach (e.g
BS 5400, DnV, ECCS). Recently, various methods used
predicting the ultimate strength of stiffened panels (

axial compression) have been assessed and new methods proposed /31,32/.

The beam model is sometimes established by taking the plate flange to have a given effective width, while in others, the relative effective width is to taken to be the ultimate strength $\sigma_u/\sigma_o$, according to, e.g., Eq. (9) for axial compression. For tension flanges, the relative width is 1.0. The basic Perry equation for a column subjected to an axial compressive force P and a bending moment, M at a cross section with an effective area $A_{eff}$ and an effective moment of inertia, $I_{eff}$ is:

$$\frac{P}{A_{eff}} + \omega \frac{M}{I_{eff}} y = \frac{P_o}{A_{eff}} \tag{10}$$

where y is the distance from the neutral axis to the outer fiber, $\omega$ is $\pm 1$ and $P_o$ the yield load. For an imperfect column under axial compression, the maximum bending moment is approximately taken as:

$$M = \Delta_o \cdot P/(1 - P/P_E) \tag{11a}$$

where the $\Delta_o$ is the initial imperfection and $P_E$ the Euler load.

In evaluating the collapse stress using the Perry equation, it is necessary to examine two cases as the overall column initial geometrical imperfection ($\Delta_o$) may occur in a direction towards the plate or towards the stiffener tip. For each of these two cases, the check point (i.e., the value of y) may be different, depending on whether the stress check for compressive yield is applied to the plating (PC) or to the stiffener (SC).

To account for transverse or shear stresses in the plating, an efficient yield stress may be applied; see, e.g. DnV-CNB /11/.

In Eq. (10), $\omega = 1$ when checking against compressive failure either in the plate panel (PC) or at the stiffener tip (SC) while, on the other hand, $\omega = -1$ when checking against tensile failure at the stiffener tip (ST).

Eqs. (10-11a) may be readily solved and $\sigma_u = P_u/A_{eff}$ expressed in terms of $\Delta_o$, $\omega$, $\sigma_o$ and $\sigma_E = P_E/A_{eff}$. Different codes specify different values of the "effective" imperfection parameter $\Delta_o$. It is noted that the ECCS (column) approach is a modified "Perry approach" by referring to the stress at the cross-section centroid rather than the stiffener tip stress. DnV introduces an imperfection for the collapse mode associated with the column deflection towards the stiffener tip. However, it is pointed out in /30,31/ that in the analysis of continuous panels, the line of action of the load is allowed to follow this shift in neutral axis, and the correction is unnecessary.

The column approach does not normally cater for tripping of stiffeners. At present, it is usual to specify slenderness

criteria to prevent this mode from occurring prior to ove
collapse. Some relaxation on the slenderness limitation
possible if the stiffener is not fully stressed or if
stress distribution across the depth has a high ber
component. In most cases, these criteria are based on
buckling response of a single stiffener outstand, modeled
plate panel under uniform axial compression with three e
simply supported and one edge free, but this model ca
properly take into account the interaction between the p
and the stiffener /34,35/. Plate buckling, which preceeds
ultimate limit state, may initiate deformation and prema
stiffener failure /35/. While this problem can be analyze
numerical methods, appropriate "strength of materials
design code formulations", are still lacking.

The strength of narrow stiffened flanges with only a
longitudinal stiffeners can be underestimated by the co
approach, since transverse action may be significant /
ECCS provides a strength formulation for orthotropic
stiffened panels.

### 3.3.2 Uncertainty measures

The uncertainty of existing code formulations (DnV, I
BS 5400, and API) and a method proposed by the research
at Imperial College has been assessed by using the result
23 tests for axially compressed panels /31,32/.
imperfections and residual stresses in the prediction m
are taken as fixed, not varied according to those in
specimens. The results are summarized in Table 1. The
results are not included because this method does not see
have been correctly used in /31/. It is seen that the bia
COV are almost the same (1.10 and 15%) for all methods, e
that the proposed IC-method yields less scatter. This ma
due to the fact that the actual imperfection for each spe
was used in the theoretical predictions by the IC-method
closer look at the results /33/ reveals that the uncert
depends upon plate and stiffener slenderness:

- for stocky plates (normalized plate slenderness, $\beta$ = 0.9
  - the ECCS orthotropic approach seems over-conservativ the normalized stiffener slenderness, $\lambda < 0.8$
  - the ECCS column approach could over-estimate the str for a very slender column ($\lambda < 0.2$)
- for slender plates ($\beta > 2.9$) all codes are conservative

When applying the model uncertainty in Table 3.1
should be recognized that it somewhat accounts for variab
of $e(x,y)$ and $\sigma_r(x,y)$. The uncertainty measures, hence,
to be appropriately modified to reflect the real variab
and the tolerance level set.

Table 1 Comparison of Model Uncertainty Based on 23 Tests of Axially Compressed Panels /31/

| Code | Minimum Value | Maximum Value | Mean Value | COV (%) |
|---|---|---|---|---|
| BS5400 (1982) | 0.89 | 1.46 | 1.10 | 13 |
| DnV, Buckl. Notes 30.6 (1987) | 0.94 | 1.52 | 1.10 | 16 |
| ECCS (Column Appr.) (1990) | 0.86 | 1.44 | 1.08 | 14 |
| ECCS (Orth. Appr.) (1990) | 0.94 | 1.68 | 1.14 | 15 |
| Method proposed by Imp. College | 0.94 | 1.34 | 1.10 | 08 |

The model uncertainty of the proposed IC method was also estimated by comparing predictions with this method with accurate numerical results for 63 panels. The resulting bias and COV of 1.05 and 5% are, as expected, less than 1.10 and 8%, previously obtained for the IC method. This is partly because imperfections etc. are more well defined in the "numerical test" panels. However, since the latter uncertainty measures do not contain any effect of parameter uncertainties, the bias and random uncertainty must be appropriately modified when applied to actual panels.

It seems that several prediction methods yield a reasonable accuracy for panels under axial compression. Moreover, the uncertainty varies with the slenderness of the plate and panel, and it could be further reduced by expressing it as a function of slenderness.

## 3.4 Stiffened panels subject to axial compression, bending or lateral pressure

### 3.4.1 Formulations

Some codes specify ultimate strength for panels under combined axial load (P) and bending, due to end moments or lateral load by ultimate strength interaction equations, such as Eq. (17). The DnV approach for combined axial lateral loading, q, in multispan stiffened plates is essentially such an approach using an equivalent moment equal to $qba^2/16$ (where a and b are plate length and width, respectively) and an assumed effective length of 60% of the span. This involves carrying out a stress check for each possible failure mode (that is PC, SC and ST; see Section 3.3.1) at the most critical section within the span under consideration. Considering both the normal and bending stresses in a simply supported strut under bending, the Perry equation (10), should be applied with /31,32/:

$$M = \left(M_{eq.} + P\Delta_o\right)/\left(1 - P/P_{cr}\right) \qquad (11b)$$

$$M \cong M_q + \left(P\left(\Delta_o + \Delta_q\right)\right)/\left(1 - P/P_{cr}\right) \qquad (11c)$$

for the case of end bending moments and uniform lat pressure, q, respectively. $M_{eq.}$ is the equivalent ben moment for the case of variable moment. $\Delta_o$ is the ini column deflection, and $M_q$ and $\Delta_q$ are the bending moment deflection at the midspan due to lateral load, respectiv When calculating the effective width for the plate element cases with lateral pressure, the effect of lateral pressur $b_e/b$ needs to be considered.

All the proposed equations (Eqs. 10-11) are related simply supported single-span model /31,32/, and they ca used to study axially compressed stiffened panels between c frames. However, a simply-supported single-span model is appropriate for the design of multi-bay longitudina stiffened panels subjected to axial and lateral loads, du continuity effects and interaction in between adjacent s having different initial imperfections.

The Perry model for the case with axial and lateral l may be extended to clamped ends. While Eqs. (10,11a) are realistic for multispan panels when axial loads predomin the clamped end case is expected to be a better approxima when lateral loads on multispan panels predominate.

The IC method /31-33/ deals with multi-span by introdu an additional deflection and an additional central span mo due to the lateral pressure acting on a fixed ended s Then, each span (column) with its additional deflection moment is calculated as an axially loaded simply-suppo span.

Interaction equations such as Eq. (17), are conservative than methods based on a stress check at crit sections (Eqs. 9-11), but are still more convenient implementation in design rules since various forms and le of bending can be catered for.

The Perry equations, however, are essentially a st check at one point, and their use in complex load cases be involved. Reduced uncertainty usually may require a complicated checking procedure. Hence, this refinement mus balanced against the consequences of potential errors mad using the method.

Interaction curves of stiffened plates under combined plane compression and shearing stresses, was also investig in /37/. Buckling, ultimate and fully plastic strength states are considered for flat plates and unidirection stiffened plates. Minimum stiffness ratio of stiffene plate for buckling ($\gamma^B_{min}$) and for ultimate strength ($\gamma^U_{min}$) determined. For $\gamma < \gamma^B_{min}$, the buckling interaction curv valid, for $\gamma^B_{min} < \gamma < \gamma^U_{min}$ the ultimate strength curve is and, if the stiffened plate is stiff enough, a fully pl strength interaction curve is proposed. Ultimate str determination is based on stress coefficients that ma analytically evaluated using parametric formulae.

### 3.4.2 Uncertainty measures

There is very limited information about the uncertainties in the strength models for panels with axial and lateral loads. Comparisons have been made in /31/ between four experimental tests of the axial capacity for given lateral load levels and values predicted by the DnV and IC-methods. The ratio of test/predicted values ranged between 1.1 to 1.35 for the DnV approach and between 1.05 to 1.25 for the Perry formula (10, 11c). Interaction Eq. (17) yields even larger (conservative) bias factors.

## 3.5 Benchmark test of stiffened panels

Several classification societies and other organizations that have engaged in developing strength formulations for stiffened panels, were asked to determine the ultimate capacity of 10 multi-span stiffened panels which were selected by this ISSC committee. The focus was on the ultimate strength of (multi-span) stiffened panels located between transverse frames, failure modes and effective width of flange plating. Within the 10 panels, there are 8 longitudinally compressed panels and 2 subjected to longitudinal compression and lateral pressure. There are 6 Smith's panels /39/ (same geometries and actual imperfections) and 4 typical panels in the VLCC "Energy Concentration" ship that broke its back during discharge of oil in 1980 /40/.

The contibutors to the benchmark study were first asked to analyze the 10 panels. Then all participants were informed about the outcome of all analyses and, if they wished, allowed to rerun their analysis /22/. The resulting model uncertainties are shown in Table 2. The names of contributors and codes/methods used, are made anonymous. The contributors are referred by one capital letter (A, B, C, etc.) and the codes by the symbols (M1, M2, etc.). The codes are in alphabetic order, BS5400 /9/, Bureau Veritas /10/, DnV /11,12/, ECCS column and orthotropic approaches /13/, Imperial College method /31/, Lloyds Register /41/ and Ueda-Yao's method /30/. Some contributors used codes as well as finite elements and other numerical methods. The latter types of results are not included in this summary.

It should be noted that the initial imperfections of the specimens have a large variation, and especially that the initial deflection of the stiffener for case 3 is two times larger than the tolerance. Initial plate deflections exceed the particularly restrictive tolerance requirement of the ECCS (column) code for all cases. Moreover, some methods had to be used outside their specified range of validity in this study. It was found that the effective width used in the various methods, differed by 0-15%, except for the method M6, which gave up to 45% lower value than other methods.

Some codes have been used by several contributors. It is

Table 2 Ultimate Strength to squash load for stiffened panels predicted by various organizations using different methods, obtained by the participants after being informed about the outcome of all first analyses.

| CONTRIBUTORS | | A | A | A | A | B | B | B | B | B | C | C | D | D | E | F | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| METHODS/CODES / Boundary conditions | | M1 With edge Stiff. | M1 No edge Stiff. | M2 With edge Stiff. | M2 No edge Stiff. | M1 No edge Stiff. | M2 No edge Stiff. | M3 No edge Stiff. | M4 No edge Stiff. | M5 No edge Stiff. | M5 Single beam column | M6 Single beam column | M6 With edge Stiff. | M7 No edge Stiff. | M8 Single beam column | M9 Single beam column | | |
| Panel Reference No | Pu/Po Experim. C.SMITH | Pu/Po | Pu/Po | Pu/Po | Pu/Po | Pu/Po | Pu/Po | Pu/Po | Pu/Po | Pu/Po | Pu/Po | Pu/Po | Pu/Po | Pu/Po | Pu/Po | Pu/Po | Mean | COV |
| Smith 1.a | 0.76 | 0.708 | 0.672 | 0.680 | 0.631 | 0.672 | 0.572 | 0.659 | 0.686 | 0.564 | 0.580 | 0.930 | 0.708 | 0.672 | 0.751 | 0.650 | 0.676 | 0.126 |
| Smith 2.b | 0.83 | 0.805 | 0.800 | 0.832 | 0.812 | 0.803 | 0.764 | 0.811 | 0.797 | 0.833 | 0.790 | 0.930 | 0.812 | 0.803 | 0.543 | 0.870 | 0.800 | 0.098 |
| Smith 3.b | 0.61 | 0.593 | 0.585 | 0.588 | 0.544 | 0.570 | 0.559 | 0.556 | 0.418 | 0.683 | 0.660 | 0.910 | 0.634 | 0.625 | 0.118 | 0.783 | 0.588 | 0.282 |
| Smith 4.a | 0.82 (1) | 0.709 | 0.703 | 0.703 | 0.689 | 0.693 | 0.652 | 0.664 | 0.560 | 0.784 | 0.770 | 0.940 | 0.775 | 0.771 | 0.508 | 0.870 | 0.719 | 0.145 |
| Smith 5 | 0.72 | 0.527 | 0.495 | 0.484 | 0.432 | 0.493 | 0.411 | 0.482 | 0.542 | 0.461 | 0.480 | 0.920 | 0.495 | 0.455 | 0.378 | 0.467 | 0.501 | 0.237 |
| Energ. Conc. Bottom Shell | - | 0.899 | 0.871 | 0.845 | 0.810 | 0.880 | 0.771 | 0.839 | 0.858 | 0.824 | 0.580 | 0.700 | 0.892 | 0.877 | 0.731 | 0.907 | 0.818 | 0.106 |
| Energ. Conc. Upper Deck | - | 0.798 | 0.783 | 0.819 | 0.794 | 0.791 | 0.735 | 0.752 | 0.804 | 0.802 | 0.750 | 0.850 | 0.870 | 0.858 | 0.874 | 0.689 | 0.798 | 0.063 |
| Energ. Conc. Side Shell | - | 0.928 | 0.917 | 0.863 | 0.843 | 0.923 | 0.813 | 0.887 | 0.892 | 0.893 | 0.840 | 0.880 | 0.924 | 0.915 | 0.923 | 0.881 | 0.888 | 0.039 |
| .a Smith 1.b without lateral pressure | - | 0.702 | 0.664 | 0.662 | 0.613 | - | - | - | - | - | - | - | 0.684 | 0.645 | - | - | 0.662 | 0.043 |
| with lateral pressure P=103.4kN/m2 | 0.73 | - | - | - | - | - | - | - | 0.512 | 0.529 | 0.540 | 0.930 | - | - | 0.670 | - | 0.636 | 0.247 |
| 0 Energ. Conc., Bottom Shell without lateral pressure | - | 0.889 | 0.871 | 0.845 | 0.810 | 0.880 | 0.771 | 0.839 | 0.858 | 0.824 | 0.580 | 0.770 | 0.892 | 0.877 | 0.731 | 0.907 | 0.818 | 0.106 |
| with lateral pressure P=200kN/m2 | - | - | - | - | - | - | - | - | 0.660 | 0.774 | 0.580 | 0.700 | - | - | 0.820 | - | 0.707 | 0.119 |

(1) C. Smith panel 4.a has longitudinals interspersed with two tee-bars of 40 inches that are not considered by the contributors.

different contributors are different. Hence, this benchmark test shows two kinds of uncertainties: model (and parameter) uncertainties associated with the method for strength prediction, and uncertainties/"errors" associated with transforming the physical problem into the mathematical model handled by the method (i.e., modeling the geometry, boundary conditions, etc.), possible errors in the respective computer codes, use of the computer code, etc.

The benchmark study is described in more detail in /22/.

## 3.6 Stiffened girders subject to in-plane axial and bending stresses

### 3.6.1 General

Girders may be part of monocoque steel-plated structures as shown in Fig. 2. The flanges may be wide and stiffened; or narrow, unstiffened plates. The ultimate capacity of flanges with uniform in-plane loads is reviewed in Sections 3.2-3.3. However, it is noted that the stress in wide box girder flanges will be non-uniform due to shear-lag. Except for cases with very slender stiffeners, the effect of shear-lag on ultimate strength, however, can be ignored, as demonstrated, e.g., in the review /43/ and the recent study /36/.

The focus here is on the stiffened web subjected to in-plane axial bending or shear forces, and the interaction between the web and flanges. The international research effort in response to the box-girder bridge failures in the early 70's has led to the publication of numerous ultimate shear strength analytical models (based on the tension field approach), as well as some numerical models, which attempt to provide the most efficient and effective solution to the design of structures. Ultimate strength formulations for plate and box girders of civil engineering structures have recently been reviewed in /18,43,44/ and quantitatively assessed in /45,46/. As codes for marine structures have not been based on truly ultimate strength formulations, the review provided in Sections 3.6-3.7, is partly done as a tutorial presentation.

Currently, two approaches are applied in estimating the contribution to the ultimate capacity from the web subjected to in-plane direct and bending stresses:

- for webs with no stiffeners or transverse stiffeners connected to wide flanges (box girders) the contribution from the web is ignored (and the web capacity is utilized to carry shear forces); see, e.g., BS 5400 (Pratt truss approach)
- for webs with transverse or especially longitudinal stiffeners connected to narrow webs, the web is accounted for by a complete analysis of the load shedding from webs to flanges by means of an effective width approach.

Overall collapse is, by the current design pract governed by failure of longitudinal load carrying stiffe and flange failure. By assuming that the plate buckles be the stiffeners and flange, the effect of the plate car accounted for by the effective width concept.

### 3.6.2 <u>Plate failure modes</u>

For unstiffened webs, the "efficiency" ($\eta$) of the compression zone, may be calculated by /18/:

$$\eta = \left(1 - 0.05(3 + \psi)/\lambda_p\right)/\lambda_p$$

This expression is a simple, generalized form of Eq. (9) plates under uniform axial stress. The efficiency depend the normalized web slenderness ($\lambda_p$) and the ratio ($\psi$) of axial stress in the upper and lower flange. Alterna expressions are also provided in /18/. The effective cr section method may also be employed for the case longitudinally stiffened webs with the difference that eac the forming subpanels must be considered separately /18/. addition, Cooper's formula /47/ is commonly used for do symmetric I sections (e.g., North America) as well as in Cardiff approach (paragraph 3.7.1) /48,49/.

The information about the effect of perforations/cut- in girder webs on the capacity under direct stresses bending, is limited, but the effect is also, especially girders with longitudinal stiffeners, relatively small.

Local failure of narrow compression flange(s) stiffeners are prevented by requiring that slenderness crit are fulfilled. For instance, the width to thickness of girder flange should satisfy the following criterion:

$$c/t_f \leq 18.5\beta\sqrt{235/\sigma_{of}} \qquad ($$

where $\sigma_{of}$ (in N/mm$^2$) is the yield stress of the flange mater The semi-empirical Winter formula requires $\beta \cong 0.72$ and value has been adopted in Eurocode 3 and in the Recommendations /18/. AISC /50/ specifies a value of $\beta \equiv$ ( while BS5950 /51/ requires a value of $\beta \equiv 0.76$. When the w to thickness ratio does not comply with the limiting value "effective" compression flange should be defined /44/. Sin local buckling criteria apply for stiffeners aligned in direction of loading.

To avoid buckling of the flanges into the web, the depth, d to thickness, $t_w$ should satisfy /18/:

$$d/t_w \leq 0.55\sqrt{A_w/A_f}\left(E/\sigma_{of}\right) \qquad ($$

where $A_f$ is the cross-sectional area of the larger fl (The web slenderness must anyway be restricted to reduce of-plane "breathing" which can cause fatigue cracks under repeated loading conditions.)

### 3.6.3 Overall failure of narrow girder flanges and stiffeners

For girders with narrow flanges, the collapse of the flange out of the girder plane needs to be considered.

For plate girders with high web slenderness ratios, the pure torsion contribution is neglected and the critical torsional buckling stress is expressed only by the elastic warping resistance, $\sigma_{LT,cr}$. There is no general agreement on the formula to be used for the ultimate lateral torsional buckling stress. In Europe the following has been recommended /53/:

$$\sigma_{LT,u} = \sigma_o\left(1 + \lambda_{LT}^{2n}\right)^{-1/n} \tag{14}$$

with n = 2.5 and $\lambda_{LT} = \sqrt{\sigma_o/\sigma_{LT,cr}}$. Eurocode 3 indicates a value of n = 1.5 and suggests another approach based on a column buckling type curve. Formulae similar to the above are suggested in /19/.

Transverse stiffeners have a limited effect on the capacity under longitudinal direct stresses. Such stiffeners are more important in the case of predominantly shear loads and are, hence, discussed in Section 3.7.

Longitudinal stiffeners must be able to support the load shedding due to plate buckling in the web subpanels, up to the ultimate limit state, and must maintain nodal lines in a buckled web subject to shear. Hence, they must be appropriately sized. Experiments and numerical simulations indicate /44/ that the relative flexural rigidity ($\gamma_L = 10.92 I_L/dt_w^3$) of the longitudinal stiffeners need to be increased beyond the optimum relative stiffness $\left(\gamma_L^*\right)$, i.e., $\gamma_L \geq m_L\gamma_L^*$ where $m_L$ is a semi-empirical amplification factor. The required rigidity depends, e.g., on the location of the longitudinal stiffeners, the aspect ratio of the longitudinally stiffened web panel, the web slenderness, the relative cross-sectional area of the stiffener and the web, the relative torsional rigidity of the stiffener (the effect of which is usually disregarded) and the type of loading /54/. In calculating the second moment of area about the neutral axis parallel to the web plate, an effective width of plate given by $b_e = t_w\sqrt{E/\sigma_{ow}}$ /18,44/ should be assumed.

Longitudinal stiffeners are load-carrying and a direct strength check may be more reasonble (BS 5400), e.g., considering a strut approach. However, in general, for girders subjected to both shear and bending, a stiffness or strength approach is recommended /44/.

### 3.6.4 Uncertainty measures

The uncertainty for direct and bending in-plane load
expected to be as for the stiffened panels treated in Sect
3.2-3.3. The uncertainty associated with the BS 5400 appr
for bending loading, was estimated to correspond to a bias
COV of 0.988 and 4%, respectively /55/. Similarly, based
sample of 18 test specimens, the bias and COV for the Co
model was estimated to be 0.99 and 4-5%, respectively /56/.

## 3.7 Girders under predominantly shear stresses

### 3.7.1 General

Several theories to calculate the ultimate capacit
girders under shear, have been reviewed in detail in /44-
The general conclusion /46/ is that the Cardiff model /48
represents the state-of-art. This view is also supporte
the results of uncertainty analysis referred in Section 3.7

The Cardiff model has been implemented, e.g., in the c
/9,14,51,52/, however, with slight modifications. BS5400
BS5950 do not allow the use of tension field theory
longitudinally stiffened girders. EC3 does not c
longitudinally stiffened girders as yet.

The ultimate strength of girders in shear consists of
sum of three terms /48,49/ as shown in Fig. 4:

(1) The elastic/inelastic critical buckling load $V_1$ of the
plate (beam action).

(2) The load $V_2$ carried by the post-critical tension f
(tension field action). The tension field contribu
will be reduced by the presence of web perforati
especially if they are located at the geometrical cent
of the web plate.

(3) The load $V_3$ carried by the flanges at the instan
plastic hinge formation in the flanges (Vierendeel f
mechanism).

From vertical equilibrium, the ultimate strength in shear
plate girder is determined by (Fig. 4):

$$V_{ult} = V_1 + V_2 + V_3 = V_{cr} + 0.5\sigma_t^o\, t_w \sin 2\theta(d - b\tan\theta) + \sigma_t^o\, t_w(c_c + c_t$$

where $\sigma_t^o$ is the tension field stress to cause yielding o
web material in the tension band (based on the von Mises
equation); d and $t_w$ are the depth and thickness of the we
is the distance between transverse stiffeners; $\theta$ is the in
nation of the tension band to the horizontal; and $(c_c, c_t)$
the anchorage lengths of the tension band on the compre

and tension flanges, respectively. $c_c$ and $c_t$ identify the positions of the plastic hinges. The latter are dependent on the flange rigidities which are themselves influenced by the presence of any direct and bending in-plane stresses.

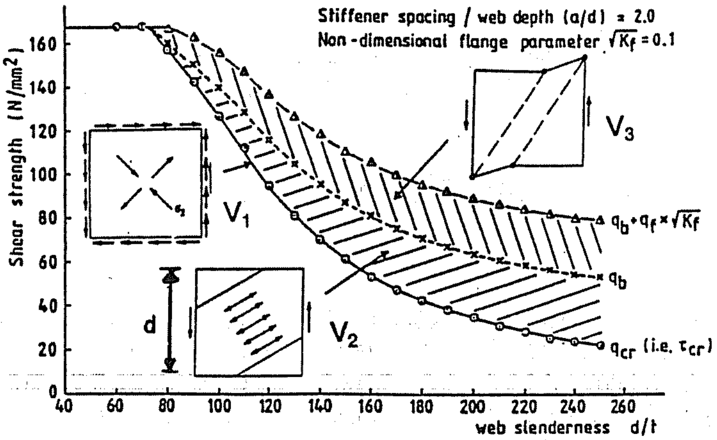



Fig. 4 Contributions to the ultimate shear strength of a plate girder /48/.

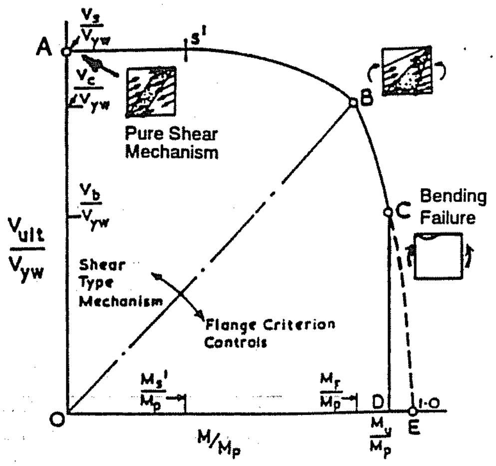



Fig. 5 Interaction curve between shear and bending effects /45/.

The Cardiff model has been validated for plate girders with narrow, fairly thick flanges (see Section 3.7.5). The third term ($V_3$) may not be fully developed in box girders with wide, thin flanges, by using the "conventional" effective width of the flanges. Hence, limited use of the plastic frame mechanism action is recommended for box girders in, e.g., BS5400; see /43/. If $V_3$ is neglected, the Cardiff model effectively reduces to the Basler model /44,48/.

Transverse stiffeners affect all terms in the expression, and are crucial to the shear capacity. Longitudinal stiffners are only accounted for by their influence on the critical buckling stress of the web, and hence the term $V_1$. Although the Cardiff model does not represent the most accurate approach /18/, it is recommended in /18,57/ because of its simplicity.

### 3.7.2 Effect of direct and bending in-plane stresses – interaction between shear and bending

Any direct and bending in-plane stresses affect the membrane stress tension field $\sigma_t^o$ and the plastic moment capacity $M_{pf}$ of the flanges, in addition to the critical buckling stress of the web plate.

The stress resultant of the diagonal tension band gives rise to a horizontal component in the flanges. The Cardiff model does not account for the effect of this horizontal component on the bending capacity of the flanges, while /46,58/ propose to do so. However, the Cardiff model makes allowance of axial stresses in the flanges caused by in-plane girder bending and axial forces.

Interaction between shear and bending and direct ax stresses is accounted for by the Cardiff model using a diag as shown in Fig. 5. Parts AC and CD are based on the sh capacity (Section 3.7.1) and bending (direct stress) capac (Section 3.6), respectively. A transition in failure mo occurs at point B, where the applied bending moment is assu to be solely carried by the flanges. Between points S' (wh $M_s \sim 0.5$ Mp) and B; and B and C, the curve may be represen by parabolas, or, without much loss of accuracy, strai lines. Points D and E refer approximately to first yield plastic moment, respectively.

### 3.7.3 Stiffener requirements

The shear capacity can be achieved only if the stiffen fulfill certain stiffness and strength requirements.

Transverse stiffeners (including an effective width of web) should fulfill stiffness requirements analogous to th mentioned for longitudinal stiffeners in Section 3.6. Howev transverse stiffeners carry compression forces equal to difference in predicted ultimate strength ($V_u$) of the gir and the critical buckling strength of the web panel in she acting in the plane of the web. The eccentricity to the sti ener's neutral axis of the loads introduces an additio bending moment in the stiffener. In addition, the out- straightness of possible longitudinal stiffeners (in the c pression zone only) causes a destabilizing moment in transverse stiffener. The intensity of each transverse fo is recommended to be taken as 1% of the compression load in longitudinal stiffener (without any web effective width) /4 The strength of the transverse stiffeners is then verified the beam-column approach, similar to those outlined in Sect 3.3, using a buckling length of 70% of the stiffener de /18,44/. Some transverse stiffeners may have to carry p loads from other major components, e.g., in frame corners. additional stiffeners requirement for transverse stiffeners briefly reviewed in /44/.

Longitudinal stiffeners should fulfill certain stiffn and strength criteria, considering the destabilizing effect shear stress.

### 3.7.4 Effect of web cut-outs

Significant information only exists for shear loading w very low axial bending stresses and for vertically stiffe plate girders containing one cut-out per panel. Accur formulations for special cases with (reinforced) cut-outs presented, e.g., in /59,60/.

The ultimate capacity can be assessed approximately linear interpolation between the unperforated ultimate stren ($V_u$) and the Vierendeel (frame mechanism) load ($V_v$) /6 Hence:

$$V_{ult} = V_v + (V_u - V_v)(1 - D/d) \quad (16)$$

where D/d is the ratio of the opening diameter to girder depth. As mentioned before, this approach has been validated for plate girders where the relatively heavy flanges provide bending capacity in the frame action.

### 3.7.5 Uncertainty measures

Uncertainty measures for plate girders under shear loading are given in Table 3.

Table 3 Model uncertainty for stiffened girders (without cut-outs) subject to shear loading

| | Cardiff | EC 3[2] | BS 5400[1] | EC 3[2] |
|---|---|---|---|---|
| Ref.<br>Sample | /46/<br>164/67 | /46/<br>164/67 | /62-63/<br>102 | /62-63/<br>109 |
| Bias<br>COV (%) | 0.98/1.00<br>6/7 | 1.08/1.09[3]<br>16/13[3] | 1.14/1.6<br>16/36 | 1.10<br>14.5 |

1) BS 5400 for longitudinally stiffened panels is not based on ultimate limit state and tension field approach.
2) EC 3 do not provide criteria for longitudinally stiffened girders.
3) Refers to evaluation of model developed in Germany for longitudinally stiffened girders, for inclusion in EC 3.

While the uncertainties for the girder models presented in Table 3 are determined for "all available" data, a closer examination of the results shows that the uncertainty depends, for instance, upon the web slenderness $b_w/t_w$. The COV for $b_w/t_w$ less than 250 is about twice that for larger slendernesses (up to 400) /46/. By comparing the model uncertainties with and without accounting for the effect of the horizontal component of the tension field force in the flanges, the effect of these different formulations is found to be neglibible. Also, it is observed that the slightly different versions of the Cardiff model implemented for transversely stiffened girders in EC3 and British standards do not yield very different model uncertainties.

The BS 5400 model for longitudinally stiffened girders is based on empirical considerations /63'/ conducted before the accuracy of the Cardiff model for such girders had been proven.

The simplified approach, Eq. (16) mentioned above for accounting for cut-outs implies a bias and COV equal to 1.60 and 0.35, respectively. The more refined formulations for circular and rectangular cut-outs with/without reinforcement typically yield bias' in the range 0.83 to 1.07, and COV's between 6 and 17% /56,59-61/.

# 4. ULTIMATE STRENGTH OF CYLINDRICAL SHELLS

## 4.1 Introduction

Cylinders are frequently used in offshore platfor Significant research on the ultimate strength of cylindri columns and beam-columns was accomplished for civil engineer structures in the 1960-70's, see, e.g., /20/, and for lo failure modes of more thin-walled cylinders in aerosp structures in the 1960's; see, e.g., /64/. However, the m "stocky" geometry and the radial pressure loads acting offshore structures were not considered until the late 197 and 1980's, partial reliance being placed on long-term resea relating to submarine shells /65/.

Cylinders in offshore platforms may suffer both local overall failure. The relatively long, unstiffened thick-wal tubulars used in bottom supported platforms exhibit over failure; however, for deep water platforms, external press affects the strength. Thin-walled, stiffened tubulars floating platforms primarily suffer local failure modes, wh are particularly sensitive to imperfections.

## 4.2 Overall column and beam-column collapse

### 4.2.1 Failure modes and formulations

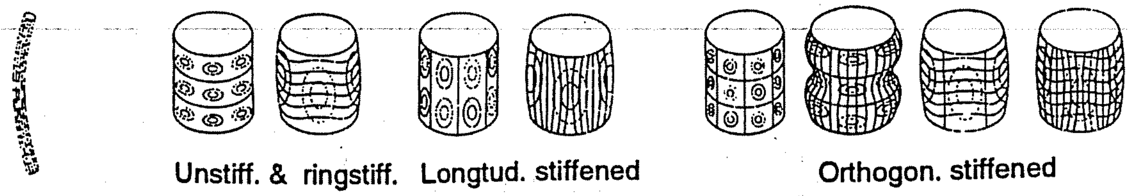
With the relatively stocky cross-section of members bottom supported platforms, overall collapse is of prim concern. For the design for local buckling, cross-sections normally classified into four groups as follows:

- plastic — full plastic action can develop and maintained through significant rotations
- compact — plastic hinge capacity can be achieved but limited rotations only
- semi-compact — yield can develop at the extreme fibers
- slender — local buckling occurs limiting flexu capacity.

The slenderness limits applicable to each group can v depending on the structural action although this subtlety frequently ignored.

### 4.2.2 Column capacity

The capacity of compression members is determined by interaction between buckling and yielding, and depe primarily upon yield strength, geometrical imperfections, well as residual stresses (and, hence, the fabrication metho as represented by column curves in Fig. 7 /74/. The effect residual stresses vanishes at high slenderness, $\lambda$, since total (including residual) stress is less than the proportional limit. For really high slendernesses, both effects vanish and the capacity approaches the ideal elastic buckling strength.




a) Column mode b) Local modes

Fig. 6. Buckling forms of cylindrical shells, see e.g. /11/.

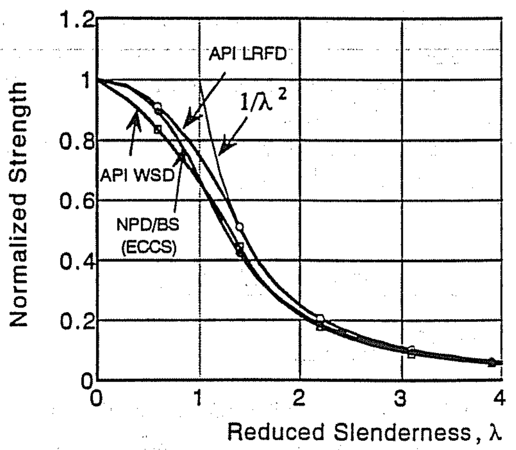



Fig. 7 Column curves

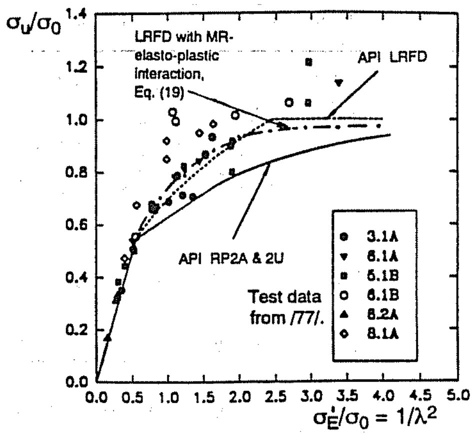



Fig. 8 Local buckling capacity under hoop compression.

Typically, differences in cross-section shape and plate thickness will influence the residual stress (and the ratio of I and A), and multiple column curves are applied. For tubular beam columns in offshore structures, there is no distinction between hot- and cold-formed tubulars /75/. Some codes (e.g., API LRFD, and some class society rules) are based on the SSRC Curve 1, while others (BS5950, DnV, NPD) apply the ECCS curve. The SSRC curve is similar to a Johnson-Ostenfeld correction to the elastic buckling stress. This curve follows the Euler curve for $\lambda > \sqrt{2}$. It may be interpreted as a correction for residual stresses (tangent stiffness formulation). The ECCS curve more explicitly accounts for both residual stresses and geometrical imperfection. In principle, a reduced yield stress should be used when applying the European column curve to fabricated columns. However, since offshore tubulars are constructed by cold rolling (which enhances the strength in the first place) and welding, this reduction is dispensed with. It is noted that API WSD is based on the AISC WSD, and is considered to be somewhat conservative for tubulars in offshore structures. This is because it is based on WF-shapes that possess high residual stresses compared with, e.g., cold-formed tubulars.

### 4.2.3 Bending

If D/t satisfies the requirement to fully plastic cr sections, the plastic moment is applicable (API codes, BS59 For larger D/t the appropriate reduction due to local buck is applied; see Section 4.3. NPD/DnV refer to the first y for the whole D/t range.

### 4.2.4 Axial compression and bending

Interaction between axial compression ($\sigma_a$) and ben ($\sigma_b$) is established in some codes (API, DnV tubular memb by:

$$\frac{\sigma_a}{\sigma_{au}} + \frac{C_M \cdot \sigma_b}{\sigma_{bu}\left(1 - \frac{\sigma_a}{\sigma_E}\right)} = 1.0$$

where $\sigma_{au}$ is determined by the appropriate column curve, σ the ultimate stress under pure bending and $\sigma_E$ is the E buckling stress. $C_M$ is a factor to account for the varia of (first order) bending stress along the member.

In addition to the overall "stability" limit state, (17) there are local yielding and buckling criteria. For b columns experiencing plastic strains before collapse, interaction between local and global behavior is complex /

The ECCS interaction equation is based on a Perry approach. The axial capacity, $\sigma_{au}$ is represented by two te $\sigma_a + e^* \sigma_a / (1 - \sigma_a / \sigma_E) = \sigma_{au}$ , such that it corresponds to the co curve by suitably choosing a fictitious eccentricity e account for the reduction in strength due to imperfection residual stresses. The interaction equation for a compression and bending is then [ECCS; e.g., NPD/DnV]:

$$\frac{\sigma_a}{\sigma_0} + \frac{e^* \sigma_a + C_M \sigma_b}{(1 - \sigma_a/\sigma_E)\sigma_0} = 1.0$$

If the plastic bending capacity is utilized in bending, (18) should be modified by increasing e* and $\sigma_0$ in the se term by the ratio of the plastic to elastic section modu Often a conservative value of $C_M$ is used in many c especially when the first order moments cause the beam-c to bend in an S-shape.

### 4.2.5 Axial compression, bending and hoop stress

Current codes do not account for the combined effe external pressure, axial compression and bending loads on columns, or they represent this interaction poorly, e.g. API 2U. Such effects are important for deep-water platf

Loh /76/ modified the (elasto-plastic part of the) RP2A column curve and the moment capacity to account for external pressure.

#### 4.2.6 Uncertainty measures

Uncertainty in overall column curves is given in Table 4.

Table 4 Uncertainty in the ultimate capacity of columns under compression

| Formulation | Data | No. of specimens | Bias | COV (%) |
|---|---|---|---|---|
| API RP2A | /77/ | 56 | 1.14 | 7 |
| API LRFD | /77/ | 56 | 1.02 | 8 |
| ECCS (DnV/NPD) | /74/ | 280 | see 1) | 5-16[1)] |

[1)]The bias may be represented by $B = 1.0 + 0.1\,\lambda$ (for $\lambda \leq 2$) and $B = 1.0$ (for $2 < \lambda < 3$) and the COV also varies depending upon the reduced slenderness, $\lambda$.

It is seen that the LRFD approach corresponds better to the mean of experimental results than the RP2A (WSD) approach. The random uncertainty (COV) for the relevant formulations is seen to be relatively small.

Ref. /78/ provides an estimate of the model uncertainty for tubular beam-columns which have so small cross-section slenderness that local buckling effects have an effect. By using Eq. (17) with $\sigma_{au}$ and $\sigma_{bu}$ that account for local buckling, the linear interaction between axial and bending stresses that include second order effects, is found to have a bias of the order 1.1.

The Loh correction of the LRFD-column curve for hoop compression yields a bias of 1.07 and a COV of 6%, while the corresponding formulation for bending capacity implies a bias and COV of 1.24 and 7%, respectively. The latter uncertainty is reflected in a bias of 1.23 and a COV of 14% for beam-columns subjected to axial compression and bending under hoop compression.

Code formulations for ultimate bending capacity and axial-bending under hydrostatic pressure, should be improved.

A crucial factor in applying the column curves is the effect of boundary conditions, as taken into account by the K-factor. Often this is done for offshore structures by using a constant value for braces and legs of jackets. This implies a significant conservative bias and some variability /79/.

## 4.3 Local buckling of unstiffened and ring stiffened shell

### 4.3.1 Failure modes

Frequently, codes for ring stiffened shells spe stiffness requirements for the stiffeners to prevent stiff collapse until the shell has failed. Hence, interframe fai is generally of concern. Nevertheless, explicit ulti strength criteria are given for densely stiffened cylind see Section 4.4. Generally, the API codes LRFD, WSD and 2U differ although it should be observed that the former codes are applicable for D/t < 300 while Bul. 2U is also v for more slender shells.

### 4.3.2 Axial compression or bending

Axial compression stresses cannot reach yield if D/t for $\sigma_a$ = 240 MPa, and the bending capacity will be less the plastic moment if D/t > 30-50 for $\sigma_0$ = 360-240 MPa, w the capacity under external pressure is essentially a l failure mode.

The three API formulations /WSD,LRFD,2U/ provided local buckling under axial load is based on the class elastic buckling formula in the elastic range. In the inela range the strength is expressed as a function of D/t, whic fitted as a lower bound to experimental data. The NPD/DnV code uses a Merchant-Rankine approach /80/ to represent interaction between elastic buckling and yielding. Cho Frieze /81/ propose to improve the correlation with test by using a knock-down factor, $\rho$ on the elastic buck strength:

$$1/\sigma_u^2 = 1/\sigma_0^2 + 1/\left(\sigma_E'\right)^2 = 1/\sigma_u^2 = 1/\sigma_0^2 + 1/\left(\rho\sigma_E\right)^2 = \left(1 + \lambda^4\right)/\sigma_0^2$$

where $\sigma_u$, $\sigma_0$ and $\sigma_E$ are the ultimate, yield and elastic buck strengths, respectively. $\lambda$ is the normalized slenderness g by $\lambda^2 = \sigma_0/\sigma_E'$.

### 4.3.3 Hoop compression

The hoop capacity is described by the elastic buck formulae for D/t > 60 for mild steel; the API-WSD an formulations are a lower bound and LRFD a mean fit to test in the elasto-plastic regime, described by several expressi The NPD approach is again based on the MR approach and below the API predictions, because the $\sigma_E$ used is conservat A modified MR method can be used to replace the capacity single expression over the whole D/t range /81/. Fig. 8 s $\sigma_u/\sigma_0$ vs.$1/\lambda^2$ using $\sigma_E'$ and $\sigma_u$ given by API and Eq. respectively. For D/t between 10 and 40 (a relevant range pipelines and tethers), a Timoshenko formulation, w essentially is a stress check of an imperfect ring, is foun yield a good estimate, e.g., better than consideration of a plastic mechanism /82/.

#### 4.3.4 Axial loading and hoop compression

API codes specify different interaction equations for all combinations of axial tension/compression and bending and hoop compression. For instance, the interaction between axial and hoop compression is described by a quadratic relationship in 2U, while WSD and LRFD take the smallest values implied by the interaction equation for elastic buckling and the elasto-plastic buckling capacities for individual loads.

NPD/DnV is based on a generalization of the Merchant-Rankine formula for interaction between elastic buckling and plastic yielding /80/. The advantage of this approach is that it allows also tensile loads to be easily handled and that it converges into linear interaction and the Von Mises-Hencky interaction for large and small D/t, respectively.

A further development of this method for combined axial and hoop compression was made in /81/ by calibrating knock-down factors on the elastic buckling stresses against test data.

For shells under axial compression and radial pressure, the effects of the various parameters controlling strength such as L/R, R/t, L/t, and $E/\sigma_0$ were examined with a view to identifying those important for inclusion in knockdown factors. The effects were found to differ markedly between the two actions, but bearing in mind a preference to use the same parameters within all of the knockdown factors in order not to complicate the equations, the parameter chosen was $(E/\sigma_0)\sqrt{Z}$, where the Batdorf slenderness parameter $Z = (L^2/Rt)\sqrt{1 - \nu^2}$; R = D/2 is the mean shell radius.

The determined equations in the presence of axial stress $\sigma_x$ and hoop stress $\sigma_\theta$ were:

$$\left(\frac{\sigma'_x}{\rho_x\sigma_{xc}} + \frac{\sigma'_\theta}{\rho_\theta\sigma_{\theta c}}\right)^2 + \left(\frac{\sqrt{\left(\sigma_x^2 - \sigma_x\sigma_\theta + \sigma_\theta^2\right)}}{\sigma_0}\right)^2 = 1.0 \qquad (20)$$

where $\sigma'_x = \sigma_x$ for $\sigma_x < 0$ and = 0 for $\sigma_x \geq 0$, $\sigma'_\theta = \sigma_\theta$ for $\sigma_\theta < 0$ and = 0 for $\sigma_\theta \geq 0$.

Clearly, this approach can be extended to other combinations of loads, but different knock-down factors will then have to be applied for a given load in different combinations to achieve a best fit.

#### 4.3.5 Uncertainty measures

The model uncertainty for current API codes (especially the LRFD approach) have been estimated by data reported in

/77/. Experimental results for ring stiffened cylinders compiled in /81/ and used to assess the uncertainty of modified Merchant-Rankine formulation, BS5500 and an version of the DnV formulation, where appropriate. While nominal geometrical data and yield stress are given for specimens, no information is given about initial imperfect and residual stresses.

Because a number of tests on single bay and st relieved shells also had been conducted, the effects of t particular factors were also examined in detail.

Among the API codes, 2U was found /79/ to give the fit to data for local buckling under axial compression, wi bias and COV of 1.07 and 8%, respectively. The NPD/DnV approach for this loading yields a bias and COV of 1.59 32%, respectively. However, as shown, e.g., in /81/, the m uncertainty increases with slenderness, suggesting applying correction factors on the elastic buckling st would improve the accuracy. By using such a modified appr a bias and COV of 1.05 and 14% was achieved /81/. different data bases applied in estimating these m uncertainties should be observed.

For bending the API LRFD formulation, which converge the nominal plastic capacity for small D/t, yields a best to data within 15 < D/t < 80, with a bias and COV of 1.16 9%, respectively. Hence, there is a significant res capacity beyond the plastic moment of an ideal pla material, essentially due to strain hardening. Howe another investigation /82/ for long cylinders with 10 < D 40 shows that the bias and COV are 0.87 and 7%, respectiv For this D/t range, it is demonstrated that other approac e.g., used for pipelines by Shell, yield a better fit.

The local hoop buckling stress in the elastic regio approximately the same for the API WSD, LRFD and 2U. The equation gives significantly higher stresses in the inela region than the "lower bound" estimates that WSD and provide. The bias and COV of the LRFD is 1.12 and respectively, by using data for D/t in the range 30 to The NPD/DnV approach is found to yield a bias of 1.15 and a of 8%. The corresponding numbers for the ECCS approach 1.21 and 11%. For more thick-walled tubulars (10 ≤ D/t ≤ the Timoshenko formula is found to yield a good fit /82/.

Comparison of the three API formulations for a tension-hoop compression shows that the LRFD formulation yi the smallest bias with 1.2, while the COV of 16% is slig larger than that for the 2U method, which has a bias and CO 1.3 and 14%. A closer look at the results shows that the predicts the inelastic capacity well, while the discrepancy long ("unstiffened") shells show a bias of the order 1.4- While the initial NPD/DnV-CNB formulation yields a bias and of 1.50 and 13%, a modified MR-formulation yields a bias COV of 1.16 and 9%, respectively.

The uncertainties in the three API formulations for interaction between axial and hoop compression are quite close; with, e.g., a bias and COV of 1.15 and 10% for API LRFD. The bias of 1.45 and COV of 20% in the NPD/DnV-CNB approach can be reduced to 1.03 and 8%, respectively, by introducing knockdown factors on the elastic buckling formulae.

For interaction between bending and hoop compression the API formulations, LRFD and 2U represent the extemities, with a bias and COV of 1.29 and 14%; and 1.44 and 8%, respectively.

An examination of stress relieved ring stiffened shell data demonstrated that, while residual stresses had little effect on axial compressive strength, under increasing levels of radial pressure, the shells exhibited increasing strength, up to 37%, in the case of pure radial pressure. It was assumed that stress relieving allowed substantial moments to develop at the frames in contrast to when residual stresses were present and yielding over the frames began early in the loading history thereby inhibiting the development of significant boundary moments.

## 4.4 Frames in ring stiffened cylinders

Among the API formulations, the 2U approach seems to be a refinement of the other formulations. This is also borne out by the model uncertainties determined /77/. The model uncertainty of 2U is clearly smaller, amounting to a bias and COV of 0.96 and 6.8%. Since this sample is small, a statistical uncertainty should be added. Since the LRFD and WSD approaches are based on buckling in two modes in the circumferential direction, they become very conservative for high buckling modes. The BS 5500 approach is also considered relevant for shells subjected to radial or hydrostatic pressure, while for axial loading the NPD/DnV-CNB may be suitable. However, no uncertainty measures are available.

## 4.5 Orthogonally stiffened shells

### 4.5.1 Failure modes and formulations

A wide range of failure modes can occur in orthogonally stiffened shells, involving the shell and stiffening elements in isolation and in combination. Thus the shell alone can buckle, the shell and longitudinal stringers combined can suffer interframe buckling, the stringers and frames alone can trip, and the shell, frames and stringers can suffer general instability - orthotropic buckling. The stiffened shell may itself not buckle, but it can suffer overall column buckling. As for ring stiffened shells, the frames are generally designed not to fail. Stringers can be proportioned not to fail in a local or overall lateral torsional mode.

The most relevant formulations for orthogonally stiffened shells in marine structures are API Bul. 2U /68/, DnV-CNB /11/, ECCS /73/ and RCC /83/. Approaches based on discr stiffeners and orthogonal shell theory are given.

This review is to a large extent based on comprehensive study /84,85/.

### 4.5.2 Axial loading

API Bul. 2U calculates the strength of the shell plat under axial compression based on the theoretical elas buckling stress, a knock-down factor which (implicit accounts for imperfections and inelastic behavior dur collapse. The axial capacity of the stiffened panel determined by first calculating the elastic buckling stres for the plate and stiffener separately. The elastic buckl stress of the stringer is determined by assuming it to simply supported, including an effective width of the she with a reduction factor for residual welding stresses. elastic buckling of the stiffened panel is then calculated the sum of the two contributions, and the collapse strength obtained by using the Ostenfeld-Bleich tangent mod approach. The resulting stress $\sigma_{ic}$ is used to calculat revised slenderness parameter and an effective width, $b_{em}$. ultimate axial capacity, $\sigma_u$ is then

$$\sigma_u = \sigma_{ic}(A_s + b_{em} t) / (A_s + bt)$$

when the ultimate force is averaged over the total cr section area.

The RCC approach, which preceded Bul. 2U, follows the procedure, however, with a modified elastic buckling stress knock-down factor for the plating, and introducing reduc factors for residual stresses depending on the weldment.

The ECCS approach is similar to that of the ab mentioned codes, but uses a less refined knock-down factor shell buckling. The DnV approach (formulation C) deviates the API/RCC methods by referring only to the stiffener wit effective shell, rather than to contributions from stiffener and shell, and like to ECCS approach, by using si "knock-down" factors.

### 4.5.3 Radial loading

The formulation for radial loading is analogous to for axial loading.

API Bul. 2U predicts the ultimate capacity by superpo the buckling pressure for the (unstiffened) shell and plastic collapse of the ring stiffener including an effec flange width, with a knock down factor.

The RCC approach is based on the plastic capacity of ring stiffener, using several empirical geometrical correc factors, and an Ostenfeld-Bleich tangent modulus to account for plasticity. However, the effective flange of the shell considered seems to be small.

The NPD/DnV-CNB and ECCS approaches for radial pressure are similar to those used for axial load, however, using the appropriate theoretical buckling coefficients.

### 4.5.4 Combined axial and radial loading

API Bul. 2U recommends the following ultimate strength interaction equation for combined axial load and radial load

$$R_x^2 + C R_x R_\theta + R_\theta^2 = 1 \qquad (22)$$

where $R_i$ is the utilization ratio for load no. i). The initial RCC approach used a different quadratic interaction function. However, in the recent study /84/, Eq. (22) was recommended, but with a different C than in the RCC formulation.

DnV-CNB uses a generalized Rankine-Merchant approach described above for unstiffened shells.

### 4.5.5 Uncertainty measures

Table 5 shows the model uncertainties estimated in /84/ using a selection of available test results for orthogonally stiffened cylinders. It is shown particularly that aluminium shells tested for aerospace structures failing by elastic buckling are not relevant for validating prediction methods for offshore structures. Also, a closer look at the model uncertainties, reveals that they vary depending upon which test sample is considered. For instance, the bias of the RCC formulation for axial load is 1.13 using 6 small scale specimens (IC1-6) and 0.93 when using 5 specimens B1-5. This fact may be related to differences in residual stresses and geometrical imperfections, even if imperfections were reported to be within the tolerance level.

Among the methods compared, the RCC method gave the best fit. Various possible improvements of the RCC formulation for axial capacity were investigated in /84/, but only resulted in minor reductions in the model uncertainty. The NPD/DnV-CNB formulation is not satisfactory for radial and combined loading. The large bias and COV are attributed to considering the shell unstiffened when the Batdorf width parameter $Z_s$ < 8.586. If the more applicable narrow panel DnV model for axial compression is used, the figures would probably improve. Even if the RCC model for radial load implies a small uncertainty, it is observed in /84/ that this is partly due to using adjustment factors to fit the limited test data available. Hence, further improvement of the basic formulation is necessary. It is also desireable to improve the interaction equation to reduce the random uncertainty (COV).

Table 5 Model uncertainties estimated by comparison with results for steel shells /84/

| Load: Test data[1]: | Axial load 48 specimens | | Radial load 11 specimens | | Interaction 38 specimens | |
|---|---|---|---|---|---|---|
| Code | Bias | COV (%) | Bias | COV (%) | Bias | COV (% |
| API 2U | | | | | | |
| - discrete | 1.02 | 14 | 1.21 | 15 | 1.13 | 16 |
| - orthotropic | 0.86 | 25 | 0.87 | 46 | – | – |
| RCC | 1.02 | 14 | 0.97 | 10 | 1.09 | 25 |
| DnV-CNB[2] (1984) | 0.97 | 21 | 1.40 | 39 | 1.68 | 25 |
| ECCS (1983) | 1.21 | 24 | – | – | – | – |

[1] $\sigma_0$: 310 to 510 MPa; R/t: 95 to 505; L/R: 0.64 to 4.8; s/t: 25 to 130.
[2] Only DnV-CNB formulation C (orthotropic theory) is applied.

It is noted that the bias of the API Bul. 2U and formulations for axial or radial loads is practically invar with respect to the predicted strength to squash load, the for the other formulations decreases towards 1.0 increasing strength (slenderness).

The advantage of the general interaction approach used in DnV-CNB is that it converges to the von Mises-He yield criterion and linear interaction for stocky and slender structures, respectively. A further advantage is it accounts for the effect of tensile stresses. However, fact that the bias/COV varies with the slenderness sugg that the formulation may be improved by introducing additi knockdown factor on the elastic buckling stress, demonstrated for unstiffened/ringstiffened shells in Sec 4.3. However, it is desireable to do this based on be information about imperfections and residual stresses, als view of the magnitude of such fabrication factors in ac structures.

Finally, it is noted that formulations for trip failure of stiffeners in general is based upon a hinge bet stiffener and shell plating. DnV-CNB allows using an ela restraint on the stiffener, but the fact that shell buck may initiate stiffener failure, is not yet covered.

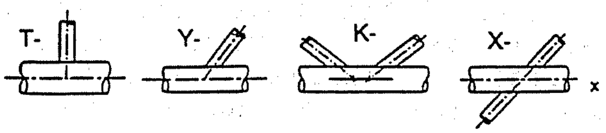
## 5. ULTIMATE STRENGTH OF TUBULAR JOINTS

### 5.1 General

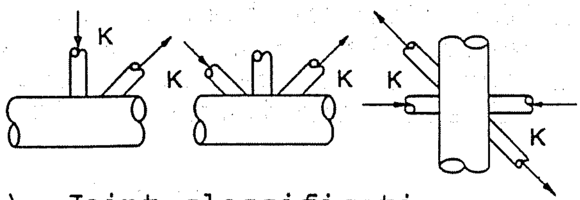
Joints between panels, and rectangular tubulars with the same width can be designed (with diaphragms) to transfer forces by membrane actions, and the ultimate strength may be fairly easily estimated. Joints between cylindrical members have a much more complex - shell - behavior. Such joints may be classified as follows:

- unstiffened, plane joints
- unstiffened, multiplane joints
- stiffened joints
- grouted joints

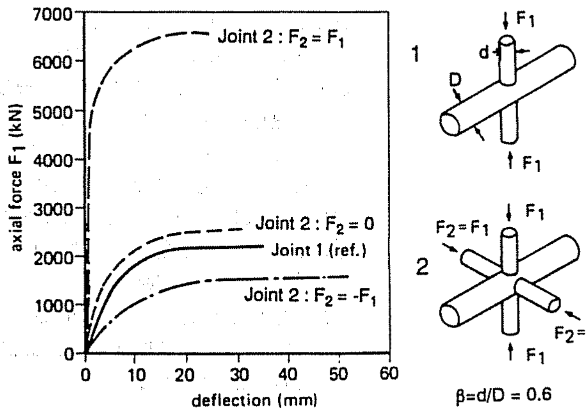
Current codes /16,17,66,67,86-88/ specify parametric formula for unstiffened, plane K, T, Y, X shaped joints, corroborated by tests. Other joints, e.g., plane KT and DT and multiplane joints, need to be considered on a case basis. Other unstiffened joints are usually classified based on the geometry and loading, see, e.g., Fig. 9b. General reviews of ultimate strength of tubular joints may be found in, e.g., /86,90/.




a) Basic joint configurations




b) Joint classification




c) Behavior of a double X-joint /99/

Fig. 9 Tubular joints

Depending upon material and geometrical properties, ultimate failure of a joint may occur due to buckling (due to compressive loading), excessive deformation, fracture or gross separation, shear failure of the chord-brace joint, lamellar tearing (of thick chords). For Y- and T-joints, it is, therefore, distinguished between compression and tensile loading. Different definitions of crack size that implies tensile failure have been used. While API, CSA, NPD and DnV use first crack, HSE /16,91/ refers to ultimate load capacity. The former approach is conservative for static loading, but is convenient to avoid consideration of interaction between fatigue and fracture. In case of tension and interaction between tension and in- or out-of-plane bending, the joint does not exhibit a distinct limit capacity in the load-deflection behavior and the limit state should be given by max deformation, and possibly maximum strain (to avoid fracture

By merging all failure modes in one ultimate limit st it is important to ensure ductility, especially for mate with yield stress, $\sigma_o$ exceeding 400 Mpa. Different c specify the ratio $\sigma_o/\sigma_u$ in the range 0.70 to 0.83.

Current formulations have been established by fit parametric formulae to experimental data. Recently, system studies using nonlinear finite element methods have also used to generate a data basis for fitting ultimate limit s equations. While API adopts a pragmatically determined l bound of test data, European codes are based more statistical analysis to determine mean and 5% fractile curv

## 5.2 Formulations for simple joints

Strength provisions in the API WSD/LRFD codes are widely used. They contain parametric formulae establishe the basis of a lower bound on experimental data selecte /92/. The formulae are generally of the form /66/:

$$P_u = Q_u\, Q_f\, \frac{\sigma_o T^2}{\sin\theta} \qquad (2$$

$$M_u = Q_u Q_f\, \frac{\sigma_o T^2}{\sin\theta}\,(0.8d) \qquad (2$$

in which $\sigma_o$ and T are the chord yield stress and thickness; $Q_u$ and $Q_f$ are nondimensional factors for brace load and c stress effects, respectively. These factors are function the ratios of the diameters of the tubulars, of diamete plate thickness etc.

The following interaction equation is recommended by A

$$1-\cos\left[\frac{\pi}{2}\frac{P}{P_u}\right] + \sqrt{\left(\frac{M}{M_u}\right)^2_{IPB} + \left(\frac{M}{M_u}\right)^2_{OPB}} = 1.0$$

where IPB and OPB refer to in- and out-of-plane bending.

Since the review /92/ was accomplished, new data become available and data used before, have been obsolete. New data bases have been established by /93/ /94/. Other code formulations have been proposed by, e.g., HSE, DnV, and NPD. In particular, improved formulations been introduced for X-joints and in-plane bending. Also, Canadian and European codes refer to a 5% (or 95%) fra instead of a lower bound.

The deviation of the NPD formulation from, e.g., AP partly caused by NPD introducing the constraint that the d values shall be obtained from the characteristic value using the same material (safety) factors.

Instead of Eq. (24) other codes apply an interaction which is linear in the axial force and out-of-plane moment and quadratic in terms of the in-plane moment.

It is also noted that the strength formulae given in various codes have different range of validity.

In view of the harmonization of offshore codes, a new data basis /95/ was established by a critical screening of existing data, and recommendations on selection of formula were given in /96/. See also Table 6.

In particular, API LRFD /66/ applies a $Q_u$ (in Eq. (23)) for in-plane bending, which is $Q_u = 3.4 + 19\beta$, where $\beta$ is the ratio of brace and chord diameters. Other codes (e.g., DnV, HSE, NPD) apply a formula of the type $Q_u = k\sqrt{\gamma}\,\beta$ (where $\gamma$ is the chord radius to thickness ratio), however, with different values of k. Recently, these formulations have been reviewed /97/ in view of a new experimental and numerical data basis for T and Y joints. While the latter form of $Q_u$ with $k = 4.75$, fits the mean numerical data generated by nonlinear shell analysis, it represents a lower bound on experimental data.

A closer look at the data for axially loaded T/Y joints reveals that the strength factor $Q_u$ is determined from tests with loads in the chord, which makes $Q_u$ smaller than it otherwise would be. However, when using Eq. (5.1) with this $Q_u$, also a $Q_f$ factor is applied. To avoid that the effect of load in the chord is accounted for twice the initial $Q_u$ should be adjusted to correspond to zero beam-bending /98/.

## 5.3 Model uncertainty for simple joints

Table 6 summarizes the uncertainty data of the formulations proposed by Yura /92/ and used in existing API guidelines. This table also shows the uncertainties implied by one slightly modified formulae, using a new data basis, which appears to be an improvement. The large scatter experienced for tensile loading is in part due to imprecise formulation of the ultimate limit state. For this reason, it may be necessary to establish two types of limit states for tensile loading, namely one referring to initial crack occurrence and one based on the ultimate load capacity.

Clearly, the uncertainties depend on how the joints are categorized in terms of geometry and load conditions. The wider variety of cases considered in one category, the larger is the scatter. It is also, therefore, necessary for instance, when all joints with in-plane bending are treated together in one category, to ensure that the data basis is representative. But for joints subject to bending there is very limited reliable experimental data for other joints than T/Y-joints, and the uncertainty measures need to be judgementally adjusted.

Table 6 Model Uncertainty of Design Formulae for Unstiff
Tubular Joints

| Joint type | Load type | Yura/API /92/ | | Study /96/[1] | | |
|---|---|---|---|---|---|---|
| | | B | COV (%) | B | COV (%) | Formu |
| T & Y | tension | 1.41 | 41 | 1.06 | 15 | CS |
| X & DT | tension | | | 1.15 | 30 | DEn/ |
| T & Y | compression | 1.07 | 8 | 1.11 | 23 | DEn/ |
| X & DT | compression | | | 1.14 | 12 | CS |
| K & YT | compression | 1.17 | 17 | 1.34 | 16 | DEn/ |
| All | in-plane-bending[2] | 1.23 | 14 | 0.97 | 21 | CS |
| All | out-plane-bending | 1.09 | 27 | 1.17 | 15 | CS |

[1] All formulations except the one for X & DT joints and in-plane be are based on Yura. The model uncertainty is determined by using data basis.
[2] Based on a new numerical data basis, the bias and COV for $Q_u = k\beta\sqrt{\gamma}$, = 4.75, are found /97/ to be 1.02 and 7.8%.

While some qualitative information about the uncerta of interaction equation is given in /97/ more work is neces to quantify the uncertainty and choice of interac equations.

## 5.4 Other joints

In practice, other plane joint configurations than simple T, Y, X, KT and K are applied. For instance, KT joints may have to be classified according to the predomi loading, as shown in Fig. 9b. However, the presence o unloaded member would necessarily influence the strengt some extent, and, hence, the model uncertainty when formulae for simple joints are applied.

In connection with multiplanar joints, it is necessar distinguish between situations where joints in a plane loaded and hence refer back to the cases discussed, situations with truly three-dimensional loading. The effec the 3D-character of the loading is substantial for doubl joints as illustrated in Fig. 9c. Double K-joints discussed in /89/. Guidance on multiplanar joints is limi but some correction factors for multiplanar effects are g in /86,99,112/, however, with no explicit assessment uncertainties.

Code provisions for ring stiffened joints are also ra limited. Obviously, it is not practicle to estab parametric expressions for such joints, and one is referre using calculation procedures.

Clearly, there is a need for syster experimental/numerical studies, including some benchmark te of the implied uncertainty of current methods used to dete the strength of complex tubular joints.

## 6. FATIGUE LIMIT STATES FOR WELDED JOINTS

### 6.1 General

In the design of marine structures against failure under cyclic loading, high cycle fatigue in welded joints, is of main concern. In general, limit states for fatigue and fracture could most accurately be established by fracture mechanics methods, especially when the effect of inspection is to be taken into account. However, the basis for current design practice both for platforms and ships subject to variable amplitude loading, is still primarily SN curves combined with the hypothesis of linear cumulative damage according to Miner-Palmgren (MP) /16,17,106-110/. Despite the deficiency of the MP method often observed, there is no competitive approach for use in design.

Adopting the SN-MP approach, the limit state may then be written:

$$\sum \frac{n_i}{N_i} - D = 0, \quad N_i = KS_i^{-m} \qquad (25a,b)$$

where $n_i$ and $N_i$ are the number of cycles in stress range block (i) and $N_i$ the corresponding number of cycles to failure, K and m are material/geometrical parameters. Possibly different (K,m) sets can be applied for different regimes of S. D is the cumulative damage at failure. The stress (load effect) may either be a nominal member stress, often used for steel plated structures or a hot spot stress, e.g., used for tubular joints in offshore structures.

Fatigue failure in connection with constant amplitude loading is typically defined as visible crack, through thickness crack, or loss of strength of the specimens used. Hence, there is residual fatigue endurance both for tubular joints, and especially for steel-plated structures when the former two types of limits are reached. However, to utilize this residual life under variable amplitude loading, it is necessary to document that premature fracture does not take place due to an overload occuring in the random load history.

### 6.2 SN curves and stress concentration

The fatigue strength of welded steel joints depends largely upon their geometrical properties: the initial crack size, weld geometry and other geometrical parameters causing stress concentration. The fatigue (crack growth) strength is practically independent of, e.g., yield stress, even if ref. /115/ reports an increase in fatigue life by a factor of 1.5 to 2.0 for cruciform joints joints made of HT-steel compared to mild steel.

In the nominal stress approach, the SN curve refers to a concentration, while the effect of weld geometry etc. is t into account in the SN curve. In principle, an infinite nu of SN curves then need to be considered to represent variety of welded joints encountered in practice. However actual fatigue design codes, nine curves (BS 5400 /102/) fifteen curves (ECCS /103/, IIW /104/) have been found t appropriate. BS 5400 has been widely used for offshore c (e.g., BV /107/, HSE/DEn /16,10/, NPD /17/, DnV-CNF /10 together with an appropriate SN curve for tubular joints. recently, BS 5400 has been adopted as a basis for ship r (e.g., ABS /109/, DnV-S /110/). GL /15,111/ uses IIW / (ECCS) supplemented by a classification of many details typ for ship structures. API codes are based on ANSI/AWS c /112/, however, with their own version of SN curves for tub joints. In the recent revision of US bridge code /113/ curves were adjusted to conform with the EECS/IIW curves /1 In the studies /115-116/, an alternative fatigue j classification for ship details was proposed.

Tubular joints are treated by the hot spot stress con using one (e.g., DEn/HSE, NPD, ...) or two curves (e.g., which account for the local weld geometry. The stresses ( effects) applied are of nominal member stresses times st concentration factor which takes into the overall s behavior of the tubular joint. The T-curve used for tub joints by HSE/DEn and NPD/DnV-CNF, is practically identica the D-curve for steel-plated structures. This curve applie butt welds. API refers to one SN curve for tubular joints a smooth weld profile and one with no postweld treatm Clearly, in the hot spot stress approach, the SCF and SN c are inexorably linked. The determination of SCF is define DEn/HSE, but no explicit procedure is given by API; howe different definitions of hot spot stress is one reason for difference in SN curves. The hot spot stress approach recently also introduced for steel-plated joints in structures (DnV-S). Two basic SN curves - one for a sm welded joint and one for base material, are applied. Th curves are taken to be in-between the BS 5400 C and D cur Stress concentration factors due to gross geome eccentricities as well as weld geometry is applied on effects.

While the SN curves in the nominal stress approach directly based on experimental data, the hot spot st approach refers to a few SN curves and relies on a calcula measured SCF. Hence, it is crucial to calibrate the hot approach using experimental data for fatigue endurance.

Ref. /117/ describes an interesting correlation o curve based on the hot spot stress approach, to se experiences of fatigue cracking or no cracking. The pr encountered in this case is associated with the effect of variability or uncertainty on the results.

The basic SN curves are modified to account for:

- environmental conditions (air, cathodic protection, exposure to free corrosion)
- effect of variable amplitude loading
- thickness effect

The SN curves beyond $N > 10^7$ are commonly modified due to the effect of corrosion and variable amplitude loading. Although, under constant amplitude loading, welded steel joints exhibit a fatigue limit, this is not necessarily the case under variable loading, since cracks initiated by high stresses may subsequently propagate under low stress levels. This may be taken into effect by suitably extending the SN curve beyond the endurance level.

Another aspect of variable amplitude loading is stress interaction, i.e., influence of previous load cycles on subsequent damage. This effect is not accounted for by the Miner-Palmgren approach.

The basic SN-curves in BV-O /107/, DEn/HSE /16/, and DnV-S /110/ refer to joints in air or with cathodic protection, with a m=3 and m=5 for $N < 10^7$ and $N > 10^7$, respectively. In case of free corrosion, the fatigue endurance is reduced by a factor of 2 and no reduced slope for $N > 10^7$ is applied. NPD and DnV-CNF use a similar approach, except that for joints with cathodic protection, the curve with m=3 applies to $N = 10^8$, at which a fatigue limit is introduced. API /67/ recommends a corrosion limit at $N = 10^7$ or $2 \cdot 10^7$ for the X and X' curves in atmospheric service, while no endurance limit is used for free corrosion. ABS-S /109/ seems to refer only to one set of SN curves.

It is noted that for ships it is normally required that the fatigue analysis is carried out by reducing the nominal plate thicknesses with the corrosion allowance. The thickness effect is generally accounted for by a correction factor $(t_{ref}/t)^{m/4}$ on the fatigue life; however, the reference thickness $t_{ref}$ varies slightly in different codes. The API approach for tubular joints accounts for thickness and weld profile, as explained in /118/.

## 6.3 Model uncertainties

### 6.3.1 General

The limit state is described by SN data and the Miner-Palmgren approach. However, since stress concentration factors for nominal stresses are intimately related to the fatigue resistance, uncertainties in SCF's are briefly commented upon.

The BS 5400/DEn/HSE curves are given with uncertainty measures (bias, COV) for K, while m is considered constant (typically equal to m = 3). Uncertainty measures of the ANSI/AWS/API data are given in /119/. The uncertainty in the fatigue endurance, N may be modeled by Weibull or lognormal

distributions /120/. While the Weibull model is theoretic
preferable, the lognormal model is frequently used.

If the joints are classified according to characteristic fatigue strength, it should be noted that will imply that each class will include joints with diffe mean value and standard deviation, if only the characteri value (mean minus two standard deviations) is the same. would increase the scatter of the joint category.

The model uncertainty is determined by endurance dat the range of $10^5$ to $10^7$ cycles, and the uncertainty representative for this range. Extrapolation, e.g., beyond $10^7$, is obviously uncertain, especially because corros variable amplitude loading and other phenomena affect this of SN-curves. This fact is also implied by the diffe models used for this area, and should be reflected in a la model uncertainty for that range.

### 6.3.2 Stress concentration factors

Fatigue design approaches based on the nominal st approach, incorporate the effect of weld and other l geometry in the SN curves. Hence, only more "glo geometrical effects need to be accounted for in the effect. However, assigning (new) welded joints to one a several categories without using experimentally determine data, would imply a certain (model) uncertainty.

The hot spot approach (used so far for tubular joint offshore platforms and for ships, e.g., by DnV) is based on or a few basic SN-curves and carefully determined st concentration factors. Parametric formulae generated systematic experiments and finite element analyses, are g for common cases, while otherwise an ad hoc determinatio SCF by experimental or numerical methods is required. Tab compares model uncertainties for some parametric formulae SCF's in tubular joints achieved in /121/. It should be n that the uncertainty varies with the SCF level, indicating the formulae are not fully optimized. Also, it is noted most design codes accept the use of different parame formulae, which obviously will increase the uncertainty imp by the code. No measures of the uncertainty in the SCF's steel-plated joints in ships /110/ seem to be available.

For the cases when, say, finite element methods applied to determine the SCF, it is necessary to have a mea for the uncertainty of the calculation procedure. Obviou this is only possible if the method is clearly specified of element, mesh size, extrapolation of stress to hot s etc.).

Table 7 Measured to predicted SCF's in tubular T- and Y-joints, based on data given in /121/

| Load | | Method | | | |
|---|---|---|---|---|---|
| SCF range | Uncertainty | Gibstein | Kuang | Smedley | UCL |
| Axial load<br>SCF: 3.3 to 13.7<br>24 specimens | Bias<br>St. dev. | 0.97<br>0.22 | 0.92<br>0.17 | 0.85<br>0.11 | 0.79<br>0.13 |
| In-plane b.<br>SCF: 1.1 to 4.9<br>12(8) specimens | Bias<br>St. dev. | 0.89<br>0.19 | 0.96<br>0.20 | 0.81<br>0.19 | 0.70<br>0.14 |
| Out-plane b.<br>SCF: 1.6 to 10.6<br>25 specimens | Bias<br>St. dev. | 0.98<br>0.18 | 1.01<br>0.25 | 0.91<br>0.16 | 0.81<br>0.11 |

### 6.3.3 Miner-Palmgren hypothesis of cumulative damage

The uncertainty in using the MP model can be assessed by calculating $D_i$ (Eq. (6.1)) at failure for specimens (i) subject to variable amplitude loading, based on the actual load spectra and SN-curves for the actual joints. The model uncertainty may be determined by the sample statistics of $D_i$ and the assumption that the random uncertainty ($V_D$) in D is composed of the model uncertainty of the MP approach ($V_\Delta$) and the uncertainty in the constant amplitude (SN) data ($V_{SN}$), as follows: $V_D^2 = V_\Delta^2 + V_{SN}^2$ /119/. The data summarized in /119/ indicate a mean Δ in the range 0.8 to 1.6 and a COV in the range of 20-30% for each data set. However, most of these data refer to block loading which yields systematically higher damage ratios than random loading and is, hence,not representative for random loading /122,123/. It should be observed that the model uncertainty of the MP approach, is directly linked to how the SN curve is modeled outside the range of available data (say, $>10^7$); and how the number of cycles are counted in broad-band loading. Broad-band loading seems to give low damage ratios at failure /122,123/. However, no explicit measures of the model uncertainty of the Miner Palmgren approach is given in these references.

## 7. CONCLUSIONS AND RECOMMENDATIONS

Ultimate and fatigue limit states for establishing de equations, and failure functions for reliability analysis been reviewed, with a focus on limit states for marine s structures.

While experimental results traditionally have been for validating or calibrating analytical limit s formulations, recent developments in numerical methods them a useful supplement. Numerical methods agree well carefully executed experiments and can be efficiently use systematically study the influence of geometrical and mate parameters.

It is (still) customary to estimate the uncerta associated with limit state equations by assuming that experiments represent the "truth". However, this is not al the case - uncertainties in the experiments will often a the estimated model uncertainty and make it conservative.

The adequacy of limit state functions should be asse by calculating the bias and COV in relation to experimenta numerical data, possibly as a function of slenderness or relevant parameters. As the bias represents a di correction to the model, it is particularly importan achieve models with a low COV, $V_R$. The resistance factor a direct function of $V_R$, e.g., $\gamma_R \simeq \exp[(\alpha_R\beta_T - k_R)V_R]$ whe is the importance factor of the resistance, and $\beta_T$ is target reliability index (typically in the range 3.2 to While $\alpha_R$ may be relatively small ($\alpha_R \leq 0.4$) for beam-colum jackets, it may be 0.7 to 0.8 for steel-plated panels shells in floating structures.

Several accurate formulations exist for axially l stiffened flat panels, with a bias and COV of 1.1 and Accurate prediction of stiffener tripping by strengt materials type formulations, still remains unsolved. Cu formulations for interaction between axial and lateral loa stiffened panels imply very large uncertainties, and fu experimental/numerical studies should be carried out to re this problem.

For merchant ships, resistance formulations base (linear) buckling analysis with a correction for plasticit still applied in some codes. Such formulations shoul replaced by the more modern approaches used for off structures. This will yield more rational design, and r the likelihood of errors (by limiting the numbe formulations used) and in general ease the transfe technology between these areas.

In the last decade, ultimate strength formulation stiffened plate girders subject to in-plane bending and have emerged in the civil engineering community. The uncertainty of the most accurate approach corresponds to a and COV of 1.0 and 7%, respectively. Such models also h potential use in marine structures, which should be explor

While adequate ultimate strength formulations are available for cylindrical columns and beam columns, the formulation for the effect of hoop compression on beam-column behavior used in existing codes, should be improved.

It is particularly important to have accurate formulations for stiffened, large diameter cylinders because the uncertainty in load effects for such components is relatively small compared to that of the resistances, and the uncertainty has a significant influence on the partial safety factor for the resistance.

The API codes LRFD, WSD and 2U contain ultimate strength criteria for cylindrical shells subjected to basic and combined loads. Formulations are more empirical fits to data than desirable, and the difference between LRFD and WSD codes is unnecessary. Among other codes, the DnV-CNB and NPD offer advantages, by applying a generalized Merchant-Rankine approach, which facilitates interaction between elastic and plastic failure, and between different load types. However, the model uncertainties in this formulation should be reduced by introducing appropriate knock-down factors.

With regard to unstiffened cylinders, ISO activities aim at harmonizing formulations (API, NPD) for an international offshore fixed steel structures code by the end of 1994. Some venues for improvement of existing formulations for ring stiffened shells are indicated when the ISO harmonization continues. The largest uncertainties for unstiffened/ring stiffened shells are associated with axial tension and hoop compression as well as bending and hoop compression. Improvement in collapse loads for orthogonally stiffened shells with radial, and radial and axial loading, is also desirable. The treatment of tripping failure of stiffeners also has to be resolved. However, it may be necessary to include more detailed information about imperfections and residual stresses and also systematically use nonlinear FEM to reach such a goal for stiffened thin-walled cylindrical shells.

A major step in developing ultimate strength formulations for tubular joints took place when the results of Yura's reassessment of experimental data were introduced in the API codes in the early 1980's. Several other codes: CSA, DnV, DEn/HSE and NPD have since then emerged, proposing different expressions for ultimate strength, with different range of validity. New experimental data and the quality and systematic use of nonlinear finite element analyses make it appropriate to establish a more universal code for the static strength of tubular joints, with a minimum of uncertainty. Besides rectifying some deficiencies in limit states for simple, unstiffened joints, a more explicit evaluation of the uncertainty in the determination of ultimate capacity of complex, stiffened joints is required.

Limit states for fatigue design of marine structures subject to wave loading, are specified in terms of SN curves and the Miner-Palmgren approach. For offshore structures, formulations based on nominal stress are applied for plated structures, while the hot spot stress method is used tubular joints, and exceptionally for other components. W Germanischer Lloyd for a long time has been applying nominal stress approach for welded joints in ships, the spot stress approach has also been adopted in at least one for ship structures. It is, therefore, desireable to harmo approaches for fatigue design. This would imply agreein when nominal and hot spot stress approaches should be used well as joint classification in the nominal stress approach data and procedures for calculating SCF's in the hot stress approach.

The uncertainties associated with SN curves, to a l extent, relate to geometrical stress concentration. W explicit uncertainty measures are available for approa based on the nominal stresses (and typically nine or more joint classes), less information is available for hot stress approaches - in which the major uncertainty transferred to the SCF (which is normally associated with load effect). It should be noted that the approach base nominal stress and joint classes, imply an uncerta associated with selecting weld joint class. SN-curves sho as far as possible, be based on nominal stresses experimentally justified SN curves. It is necessary to use spot stress approaches for new joints, but an estimate of uncertainties involved should be obtained and used. Due to limited amount of experimental data and a complex interac between variable amplitude loading and corrosion effects the range of $10^7$ cycles and beyond, a larger model uncerta should be assigned to this part of the SN curve.

When using the limit state equations in reliab analyses, an estimated model uncertainty may have to adjusted to reflect a different level of geomet imperfections and other parameters in actual struc components compared to that in the specimens or nume models. Also, model uncertainties for interaction equa need to be carefully implemented.

Hopefully, (all) codes in the future will inclu commentary which will be useful in future revisions of codes.

A benchmark study concerned with the ultimate streng stiffened panels conducted by this committee, demonstrated the implementation of a method for ultimate strength predi in a computer program, and its use, may lead to e (departure from the intended procedures).

To reduce such errors, methods and computer codes ne be validated against benchmark tests, e.g., by ISSC. addition, quality assurance of actual calculation tasks to be exercised. Yet, ultimate strength and fatigue estimates will still be affected by such "human factors".

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