This study was funded by the Bureau of Ocean Energy Management, Regulation, and Enforcement, U.S. Department of the Interior, Washington, D.C., under Contract Number M10PC00108.
Disclaimer

This report has been reviewed by the BOEMRE and approved for publication. Approval does not signify that the contents necessarily reflect the views and policies of the Service, nor does mention of the trade names or commercial products constitute endorsement or recommendation for use.
**Summary:**
The Bureau of Ocean Energy Management, Regulation, and Enforcement (BOEMRE) contracted DNV to perform a state-of-the-art comparison of API, ISO, and NORSOK existing offshore structural standards. The comparison identifies the differences and attempts to explore the reasons and if possible recommends areas of improvement with application to the US Gulf of Mexico and the West Coast offshore areas.

The study showed that even though there may be significant differences in the adopted design approach being Working Stress Design (WSD) or Load and Resistance Factor Design/Limit State Design (LRFD/LSD) and the regional design criteria, the formulations for calculating member and joint or plate/shell stresses are similar in all three standards.

It is recommended that further efforts be directed towards the harmonization of the standards. A significant step has been the recent collaboration between API and ISO and to a certain degree NORSOK to adopt a common approach to the development of future offshore structural standards. It appears that the LRFD/LSD methodology will eventually prevail and be applied to future GOM and West Coast offshore fixed and floating structures as it had for Atlantic and Arctic regions.

The limited case studies performed using a GOM fixed platform and a spar deepwater floating structure indicate that design environmental criteria are based on similar reliability analyses and definition of probability of failure. Jacket member utilization comparison indicates that both ISO and NORSOK give significantly more conservative formulation for members with cone transitions compared to API. Member and joint utilizations were noted to vary by up to 53% for members and 29% for joints. No one standard was found to be always more conservative than the other two. A single GOM spar case study showed that the ISO/NORSOK LRFD approach gives yield and buckling utilizations that are within about ±10%. Further investigations are recommended for more in-depth evaluation to reach more general conclusions.

### Indexing terms
- Structural Standards Comparison
- Offshore Fixed and Floating Structures
- API, ISO, NORSOK
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- Strictly confidential
- Unrestricted distribution
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**Nomenclature**

- Ca, Cv: site coefficients
- CSF: safety factor
- d: brace outside diameter
- D: chord outside diameter
- De: equivalent quasi-static action representing dynamic response defined in 9.8.1
- Do: equivalent quasi-static action representing dynamic corresponding to Eo
- Ee: extreme environmental quasi-static action
- Eo: environmental action or loading
- Fd: action effect
- Fd: design action
- G: Ratio of effective horizontal ground acceleration to gravitational acceleration
- G1, G2: permanent actions or gravity loads
- GT: the action imposed either by the weight of the structure in air, or by the submerged weight of the structure in water
- kDAF: dynamic amplification factor; 1.10 for heavy lift by semi-submersible crane vessel for in air offshore lifts or in air onshore or in sheltered waters; 1.30 in other cases for offshore in air.
- Q1, Q2: variable actions or live loads
- QT: the action imposed by the weight of the temporary equipment or other objects, including any rigging installed or carried by the structure
- Rd: design strength
- S: internal force

- Sa,map(0.2) = 1000 year rock outcrop spectral acceleration obtained from maps in Annex 2 of ISO 19901-2 associated with a single degree of freedom oscillator period 0.2 s
- Sa,map(1.0) = 1000 year rock outcrop spectral acceleration obtained from maps in Annex 2 of ISO 19901-2 associated with a single degree of freedom oscillator period 1.0 s
- Sa,site (T): site spectral acceleration corresponding to a return period of 1000 years and a single degree of freedom oscillator period T
- t: brace wall thickness at intersection
- T: chord wall thickness at intersection
- T: natural period of a simple, single degree of freedom oscillator
- Z: Zone or relative seismicity factor

\[ \beta = \frac{d}{D} \]
\[ \gamma = \frac{D}{2T} \]

\[ \gamma_{f,E}, \gamma_{f,D} \] are the partial action factors for the environmental actions discussed in 9.9 and for which appropriate values shall be determined by the owner.

\[ \gamma_{f,Eo}, \gamma_{f,Ee} \] partial action factors applied to the total quasi-static environmental action plus equivalent quasi-static action representing dynamic response for operating and extreme environmental conditions.

\[ \gamma_{f,G1}, \gamma_{f,Q1}, \gamma_{f,Q2} \] partial action factors for the various permanent and variable actions.

\[ \gamma_{f,dl} \] the rigging factor, 1.10 for a dual lift; 1.00 for single crane.

\[ \gamma_{f,lf} \] local factor, for lifting attachments, spreader beams, and internal members attached to lifting point: 1.25 (for a lift in open waters), 1.15 (for a lift on shore or in shelter waters); 1.00 for other structures.

\[ \gamma_{f,sun} \] partial factor, 1.30.

\[ \tau = \frac{t}{T} \]
EXECUTIVE SUMMARY

The Bureau of Ocean Energy Management, Regulation, and Enforcement (BOEMRE) contracted DNV to perform a state-of-the-art comparison of API, ISO, and NORSOK existing offshore structural standards. The comparison identifies the differences and attempts to explore the reasons and if possible recommends areas of improvement with application to the US Gulf of Mexico and the West Coast offshore areas.

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1 INTRODUCTION

1.1 Background

As stated in the Bureau of Ocean Energy Management, Regulation, and Enforcement (BOEMRE) Contract No. M10PC00108 documentation and the DNV proposal No 1-2Q1N5T-01, the objective of this work presented herein is to perform a state-of-the-art review of existing API, NORSOK, and ISO offshore structural standards. The comparison identifies the differences and makes recommendations for their possible resolution with application to the US Gulf of Mexico and the West Coast offshore areas.

The main scope of work entails the following ten tasks:

1. Environmental Loads
2. Loading Conditions
3. Structural Steel Design
4. Connections
5. Fatigue
6. Foundation Design
7. In-service Inspection and Maintenance
8. Assessment of Existing Platforms and Floaters
9. Fire, Blast and Accidental Loadings
10. Installation and Temporary Conditions

The approach employed in the study was described in DNV proposal No 1-2Q1N5T-01 and is summarized below for completeness sake.

1.2 Objective

The objective of the work is to perform a state-of-the-art review of existing API, NORSOK, and ISO offshore structural standards with respect to structural integrity aspects and produce a comparison report identifying differences and recommendations for their possible resolution for application in US Gulf of Mexico and the West Coast.

1.3 Codes and Standards

Table 1-1 lists all documents reviewed as part of this study. Only current revisions in use were considered even though many of these recommended practices (RP’s) and standards are currently under review and may be re-issued in the near future.

These standards are also included as references in Section 13.
Table 1-1: Main Design Codes

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<th>Revision</th>
<th>Title</th>
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<td>API RP 2T</td>
<td>3rd Edition July 2010</td>
<td>Planning, Designing, and Construction Tension Leg Platforms</td>
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<tr>
<td>API RP 2FPS</td>
<td>1st Edition March 2001</td>
<td>Recommended Practice for Planning, Designing, and Constructing Floating Production Systems</td>
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<td>API Bulletin 2INT-MET</td>
<td>May 2007</td>
<td>Interim Guidance on Hurricane Conditions in the Gulf of Mexico</td>
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<td>May 2007</td>
<td>Interim Guidance for Assessment of Existing Offshore Structures for Hurricane Conditions</td>
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<tr>
<td>ISO 19902</td>
<td>1st Edition December 2007</td>
<td>Fixed Steel Offshore Structures</td>
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<tr>
<td>NORSOK Standard N-001</td>
<td>7th Edition June 2010</td>
<td>Integrity of Offshore Structures</td>
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<td>NORSOK Standard N-003</td>
<td>2nd Edition September 2007</td>
<td>Action and Action Effects</td>
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<tr>
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<td>2nd Edition October 2004</td>
<td>Design of Steel Structures</td>
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DNV Reg. No.: EP034373-2011-01
Revision No.: 1
Date 2012-01-12
It should be noted that as part of the collaboration efforts between ISO TC67/SC7 and API SC2 offshore Structures committees, a standard harmonization scheme has been adopted whereby the ISO standards have utilized existing API documents as starting point in developing the ISO standards. API will subsequently adopt relevant ISO documents with modification to adapt to Gulf of Mexico and other US offshore areas.

The chart presented in Figure 1-1 was presented in the last API SC2 meeting and shows the status of the harmonization efforts as of February 2011. The figure helps identify the one to one correspondence between ISO and API documents.

1.4 Design Philosophy

Although the scope of work covered only API RP 2A (WSD) for comparison with ISO and NORSOK standards, it was decided to include API RP 2A (LRFD) published in 1993 and reaffirmed in 2003 even though it was withdrawn by API in 2010 in this comparison. The reason being that API RP 2A (LRFD) was utilizing the same design philosophy adopted in ISO and NORSOK; namely, the Limit States or Load and Resistance Factor Design (LRFD) methodology. The API Subcommittee on offshore structures (SC2) has established a Task Group (TG 19) to address the transition from WSD to LRFD adopting the ISO 19902 methodology as basis.

The utilization of LRFD/Limit States Design allows the allocation of different safety factors to the different types of loadings/actions depending on the degree of uncertainty associated with each type.
of loading. By contrast, the Working/Allowable Stress Design (WSD/ASD) methodology combines all load types with a single safety factor applied on the calculated combined stress. Therefore, the WSD method can produce less conservative designs than the LRFD methodology for storm conditions when the stress due to environmental loading is significantly higher than that associated with well-defined dead loads or weights and vice versa.

Figure 1-2 shows a comparison between LRFD and WSD when applied to design of structures also utilizing AISC steel design code for beam type members (see 30/ for more detailed discussion). Load conditions a) and b) are: a) functional loads and b) combination of maximum environmental loads and associated functional loads. The AISC 13th Ed. did not allow 1/3 increase in allowable stress to be applied only to the environmental portion of the stress and not to the static load as was allowed in the 9th Ed. version.

![Figure 1-2 Schematic of LRFD vs. WSD Methods](image)

It should also be noted that the LRFD or Limit State design method allows yielding to be reached or exceeded in such a way that the structure is still capable of resisting further loads but may encounter high levels of deformation without reaching an unstable mechanism. Unfortunately all standards do not adequately address this acceptability criterion.
1.5 Report Organization

This report is composed of twelve sections and a references section. Following this introductory section, Section 2 addresses the comparison of the environmental criteria and the associated loading conditions and applicable load and resistance factors. This covers Tasks 1 and 2, see Sec. 1.1 above. It should be noted that the term “Action” is the preferred terminology adopted by ISO and NORSOK. However, the API terminology “Load” is utilized here for convenience. Section 3 looks at the steel design formulae used to calculate the member and joint stresses and utilization ratio and as such completes Tasks 3 and 4. As a verification tool, MathCAD sheets were also developed for member and joint checks and are given in Appendices A. Non-tubular members and connections as well as plated structures are also addressed in Section 3. Section 4 compares the fatigue requirements (Task 5) while Section 5 is dedicated to the geotechnical and foundations design requirements (Task 6). Section 6 compares the in-service inspection requirements (Task 7) and the assessment criteria for existing fixed and floating offshore structures is described in Section 7 (Task 8).

The fire, blast and accidental loading criteria are discussed in Section 8 and the installation and temporary conditions comparison is given in Section 9, which address Tasks 9 and 10, respectively. Seismic requirements are discussed separately in Section 10.

Two case studies were undertaken for an 8-legged fixed platform and a SPAR floater in order to perform numerical results comparison of application of the three codes. The details of these examples are given in Section 11.

The conclusions and recommendations are listed in Section 12 and the references are given in Section 13. Appendix A contains MathCAD sheets developed by DNV in order to verify and compare the member and joint code check formulations given in the three standards.
2 ENVIRONMENTAL CRITERIA AND LOADING CONDITIONS

2.1 Environmental Criteria

A direct comparison of environmental (Metocean) loads as stated in the three standards (API, NORSOK, and ISO) was carried-out and is presented in this section. In addition, the direct comparison is supported by case studies where environmental loads were calculated and compared using the three standards separately. The provisions that have impact on the magnitude of environmental loads e.g. directional wave criteria were reviewed and compared. The components that comprise the total environmental forces/actions include wind, waves, tides, currents, and earthquakes.

For the purpose of structural design and analysis, the governing weather condition (e.g. survival load case) is taken into account. Other load conditions (e.g., operating load case) may also be considered if found necessary due to the associated safety factors and relative value of the environmental to permanent loading.

Code requirements for strength and ductility level earthquakes, SLE (or extreme ELE in ISO) and DLE (or abnormal ALE in ISO) were also compared. Seismic criteria code comparison is given Section 10.

The code environmental criteria comparison indicates the following:

1. The design environmental loads such as wind, wave, and current depend on geographical locations. In absence of site-specific data, regional information is defined in all three codes that give minimum requirements of the extreme environmental conditions:

   • API RP 2A provides Gulf of Mexico hurricane criteria (2.3.4c for new structures & 17.6.2a for assessment of existing structures). API Bulletin 2INT-MET replaced the criteria for new structures by including the recent extreme hurricanes in the database. Other API standards such as RP 2T and RP 2FPS refer to RP 2A for environmental criteria definition. API RP 2MET will be applicable to all units intended for the Gulf of Mexico.

   • ISO 19901-1 provides environmental guidelines for the regions all over the world including North-west Europe, West coast of Africa, US Gulf of Mexico, US Coast of California, and East coast of Canada. The new edition of ISO 19901-1 will adopt the new API RP 2MET for the Gulf of Mexico scheduled for publication in 2011. The current ISO 19901-1 GOM environmental criteria (see Annex C.4 and Table C.21) is higher than that given in API RP 2A but will not be as severe is the new 2MET criteria.

   • NORSOK N-003 mainly focuses on Northwest coast of Europe and refers to ISO for details.

2. For snow and Ice, NORSOK N-003 and ISO 19901-1 provide more specific information compared to API RP 2A. For Arctic and Atlantic coast regions ISO 19906/API RP 2N and ISO 19902 would be applicable.
3. For earthquake; ISO 19902 and ISO 19901-2 give clearer and more comprehensive design guidelines when compared with API or NORSOK standards.

Further details of environmental criteria are also given in Sections 2.2 to 2.4 while discussing the loading conditions from the API, ISO, and NORSOK codes.

Load and material resistance factors were compared for the various elements of the structure (e.g. jacket, hull, deck, foundations, etc.). The manner in which the codes require the combination of appropriate loads is also directly compared. This includes the following main load categories: operational environmental, design environmental, dead, live, and temporary. Fire, blast and accidental loadings are considered separately.

2.2 Loading Conditions - API

2.2.1 API RP 2A and 2INT-MET, 2INT-DG, and 2INT-EX

API RP 2A for fixed platforms states that the loading conditions should include environmental conditions combined with appropriate dead and live loads in the following four combinations:

1) Operating environmental conditions combined with dead loads and maximum live loads appropriate to normal operations of platform.

2) Operating environmental conditions combined with dead loads and minimum live loads appropriate to normal operations of platform.

3) Design environmental conditions with dead loads and maximum live loads appropriate for combining with extreme conditions.

4) Design environmental conditions with dead loads and minimum live loads appropriate for combining with extreme conditions.

Typically, a one to five year winter storm is used as an operating condition in the Gulf of Mexico. DNV has noted through projects with some GOM operators that the 10-year winter storm has conservatively been employed as the operating criteria. This is particularly true after the 2005 severe hurricane season.

As stated in Section 2.1, the extreme environmental conditions for the Gulf of Mexico specified in API RP 2A (Section 2.3.4c and 17.6.2a) have been replaced by increased criteria in a central zone of the GOM in API Bulletin 2INT-MET /11/. The change was necessary in order to account for the high activity hurricane seasons of 2004 and 2005 with Category 4 and 5 hurricanes. The Gulf of Mexico (GOM) was divided into four zones with different severity of the hurricane conditions. Four zones and three transition zones are defined in API RP 2INT-MET, Figure 2-1 with different environmental criteria. These will be further reduced to only three zones and (two transition ones) in the new API RP 2MET by combining the West and West Central zones. The three approximate gulf areas are:

- Western Gulf, between 92° W and 98° W
- Central Gulf, between 86.5° W and 89.5° W
- Eastern Gulf, between 82° W and 84° W
Figure 2-1 Gulf of Mexico Zones in API Bulletin 2INT-MET

Table 2-1 shows the API Bulletin 2INT-MET hurricane winds, waves, currents and surge for the central zone of the GOM which has the most severe conditions that have changed significantly from previous criteria. The environmental conditions in the other zones were affected only slightly. Figure 2-2 shows the original design maximum wave height specified in the API RP 2A for GOM structures. It is noted; e.g., that in the Central region, the significant wave height was increased from 12m (40 ft) to 15.8m (52 ft) for 100 year return period for high consequence L-1 structures.

Two additional interime documents were issued by API in May of 2007 ahead of the hurricane season to address requirements for design of new structures Bulletin 2INT-DG /32/, and assessment of existing structures Bulletin 2INT-EX /33/. These bulletins gave guidance, at high level, on design using the new metocean criteria of 2INT-MET and significantly increased the requirement for deck height elevation by adding 15% to the maximum wave crest for local effects. The 1000-year wave crest was also recommended for robustness consideration.

The API Bulletin 2INT-EX is discussed in Sec. 7 of this report.
Table 2-1 Central Zone Hurricane and Environmental Conditions

Table 4.5.3-1A—Independent Extreme Values for Hurricane Winds, Waves, Currents and Surge,
Central Gulf of Mexico (89.5°W to 88.5°W)

<table>
<thead>
<tr>
<th>Return Period (Years)</th>
<th>10</th>
<th>25</th>
<th>50</th>
<th>100</th>
<th>200</th>
<th>1000</th>
<th>2000</th>
<th>10000</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Wind (10 m Elevation)</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1-hour Mean Wind Speed (m/s)</td>
<td>33.0</td>
<td>40.1</td>
<td>44.4</td>
<td>48.0</td>
<td>51.0</td>
<td>60.0</td>
<td>62.4</td>
<td>67.2</td>
</tr>
<tr>
<td>10-min Mean Wind Speed (m/s)</td>
<td>36.5</td>
<td>44.9</td>
<td>50.1</td>
<td>54.5</td>
<td>58.2</td>
<td>69.5</td>
<td>72.5</td>
<td>78.7</td>
</tr>
<tr>
<td>1-min Mean Wind Speed (m/s)</td>
<td>41.0</td>
<td>51.1</td>
<td>57.4</td>
<td>62.8</td>
<td>67.4</td>
<td>81.6</td>
<td>85.6</td>
<td>93.5</td>
</tr>
<tr>
<td>3-sec Gust (m/s)</td>
<td>46.9</td>
<td>59.2</td>
<td>66.9</td>
<td>73.7</td>
<td>79.4</td>
<td>97.5</td>
<td>102.5</td>
<td>112.8</td>
</tr>
<tr>
<td><strong>Waves, WD ≥ 1,000 m</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Significant Wave Height (m)</td>
<td>10.0</td>
<td>13.3</td>
<td>14.8</td>
<td>15.8</td>
<td>16.5</td>
<td>19.8</td>
<td>20.5</td>
<td>22.1</td>
</tr>
<tr>
<td>Maximum Wave Height (m)</td>
<td>17.7</td>
<td>23.5</td>
<td>26.1</td>
<td>27.9</td>
<td>29.1</td>
<td>34.9</td>
<td>36.3</td>
<td>39.1</td>
</tr>
<tr>
<td>Maximum Crest Elevation (m)</td>
<td>11.8</td>
<td>15.7</td>
<td>17.4</td>
<td>18.6</td>
<td>19.4</td>
<td>23.0</td>
<td>23.8</td>
<td>25.6</td>
</tr>
<tr>
<td>Peak Spectral Period (s)</td>
<td>13.0</td>
<td>14.4</td>
<td>15.0</td>
<td>15.4</td>
<td>15.7</td>
<td>17.2</td>
<td>17.5</td>
<td>18.2</td>
</tr>
<tr>
<td>Period of Maximum Wave (s)</td>
<td>11.7</td>
<td>13.0</td>
<td>13.5</td>
<td>13.9</td>
<td>14.1</td>
<td>15.5</td>
<td>15.8</td>
<td>16.4</td>
</tr>
<tr>
<td><strong>Currents, WD ≥ 150 m</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Surface Speed (m/s)</td>
<td>1.65</td>
<td>2.00</td>
<td>2.22</td>
<td>2.40</td>
<td>2.55</td>
<td>3.00</td>
<td>3.12</td>
<td>3.36</td>
</tr>
<tr>
<td>Speed at Mid-Profile (m/s)</td>
<td>1.24</td>
<td>1.50</td>
<td>1.67</td>
<td>1.80</td>
<td>1.91</td>
<td>2.25</td>
<td>2.34</td>
<td>2.52</td>
</tr>
<tr>
<td>0-Speed Depth, Bottom of Profile (m)</td>
<td>69.3</td>
<td>84.2</td>
<td>93.2</td>
<td>100.8</td>
<td>107.1</td>
<td>126.0</td>
<td>131.0</td>
<td>141.1</td>
</tr>
<tr>
<td><strong>Currents, WD 10 m – 70 m</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Uniform Speed at 10 m Depth (m/s)</td>
<td>1.09</td>
<td>1.61</td>
<td>1.97</td>
<td>2.30</td>
<td>2.60</td>
<td>3.23</td>
<td>3.50</td>
<td>4.05</td>
</tr>
<tr>
<td>Uniform Speed at 70 m Depth (m/s)</td>
<td>0.98</td>
<td>1.45</td>
<td>1.77</td>
<td>2.07</td>
<td>2.34</td>
<td>2.91</td>
<td>3.15</td>
<td>3.65</td>
</tr>
<tr>
<td><strong>Water Level, WD ≥ 500 m</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Storm Surge (m)</td>
<td>0.32</td>
<td>0.52</td>
<td>0.66</td>
<td>0.80</td>
<td>0.93</td>
<td>1.13</td>
<td>1.22</td>
<td>1.41</td>
</tr>
<tr>
<td>Tidal Amplitude (m)</td>
<td>0.42</td>
<td>0.42</td>
<td>0.42</td>
<td>0.42</td>
<td>0.42</td>
<td>0.42</td>
<td>0.42</td>
<td>0.42</td>
</tr>
</tbody>
</table>

Notes:

Wind speeds for a given return period are applicable to all water depths throughout the region.
Crest elevation includes associated surge and tide.
See Figures 4.5.3-1A, 4.5.3-2A and 4.5.3-3A for wave and crest elevation values for water depths between 10 m and 1000 m.
The peak spectral period and period of maximum wave apply to waves in all water depths.
Currents in water depths between 70 m and 150 m should be estimated as described in 4.3.3.
See Figure 4.5.3-4A for surge and tide in water depths less than 500 m.
2.2.2 API RP 2T

API RP 2T is more comprehensive than RP 2A in defining the loads and load combinations due to the sensitivity of the Tendon Leg Platform to its payload. Table 2-2, Table 2-3, and Table 2-4 depict API RP 2T definition of load types, safety categories and annual probability of occurrence, and important parameters that critically impact the TLP global response.

API 2FPS refers to both API RP 2A and 2T for guidance related to environmental conditions and load definition.
Table 2-2 API RP 2T Load Definition - Description

<table>
<thead>
<tr>
<th>Load Type</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>Dead loads</td>
<td>Nonvariable static weight of the platform structure and any permanent equipment that does not change during the life of the structure.</td>
</tr>
<tr>
<td>Live loads</td>
<td>Variable static loads that can be changed, moved or removed during the life of the structure. Maximum and minimum payloads should be considered.</td>
</tr>
<tr>
<td>Environmental loads</td>
<td>Loads on the structure due to the action of wind, wave, current, tide, earthquake, or ice.</td>
</tr>
<tr>
<td>Inertial loads</td>
<td>Motion induced loads that are consequences of the environmental loads.</td>
</tr>
<tr>
<td>Construction loads</td>
<td>Loads built into the structure during the fabrication and installation phases.</td>
</tr>
<tr>
<td>Hydrostatic loads</td>
<td>Buoyancy of, or submerged pressure on, submerged members.</td>
</tr>
<tr>
<td>Combined loads</td>
<td>The combination and severity of loads should be consistent with the likelihood of their simultaneous occurrence.</td>
</tr>
</tbody>
</table>

The safety categories A and B of Table 2-3 are equivalent to the API 2A’s operating and extreme conditions. However the survival intact condition is new in 2T 3rd Edition with 1000 year return period environment. The specified 17 design load cases are stated to be given only as example and that other criteria may be used if properly justified. The 2T 3rd Ed. added 5 more load cases compared to the 2nd Ed. These are one new damaged condition, three survival conditions, and one ductility level earthquake (DLE) condition.
Table 1—Project Design Load Cases

<table>
<thead>
<tr>
<th>Design Load Case</th>
<th>Safety Category</th>
<th>Project Phase</th>
<th>Platform Configuration*</th>
<th>Design Environment</th>
<th>Annual Probability of Exceedance</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>A</td>
<td>Construction</td>
<td>Various</td>
<td>Calm</td>
<td></td>
</tr>
<tr>
<td>2</td>
<td>A</td>
<td>Load out</td>
<td>Intact</td>
<td>Site specific</td>
<td></td>
</tr>
<tr>
<td>3</td>
<td>B</td>
<td>Hull/deck mating</td>
<td>Intact</td>
<td>Site specific</td>
<td></td>
</tr>
<tr>
<td>4</td>
<td>B</td>
<td>Tow/transportation</td>
<td>Intact/damaged</td>
<td>Route</td>
<td>Varies</td>
</tr>
<tr>
<td>5</td>
<td>A</td>
<td>Installation</td>
<td>Intact</td>
<td>Installation</td>
<td>Varies</td>
</tr>
<tr>
<td>6</td>
<td>A</td>
<td>In place</td>
<td>Intact</td>
<td>One-year normal</td>
<td>≤1</td>
</tr>
<tr>
<td>7</td>
<td>B</td>
<td>In place</td>
<td>Intact</td>
<td>100-year extreme</td>
<td>0.01</td>
</tr>
<tr>
<td>8</td>
<td>S</td>
<td>In place</td>
<td>Intact</td>
<td>100-year extreme</td>
<td>0.001</td>
</tr>
<tr>
<td>9</td>
<td>B</td>
<td>In place</td>
<td>Damaged—no compensation</td>
<td>One-year normal</td>
<td>≤ 0.01 a</td>
</tr>
<tr>
<td>10</td>
<td>S c</td>
<td>In place</td>
<td>Damaged—no compensation</td>
<td>10-year reduced extreme</td>
<td>≤ 0.001 a</td>
</tr>
<tr>
<td>11</td>
<td>B</td>
<td>In place</td>
<td>Damaged—compensation</td>
<td>10-year reduced extreme</td>
<td>≤ 0.01 b</td>
</tr>
<tr>
<td>12</td>
<td>S c</td>
<td>In place</td>
<td>Damaged—compensation</td>
<td>100-year extreme</td>
<td>≤ 0.01 a</td>
</tr>
<tr>
<td>13</td>
<td>B</td>
<td>In place</td>
<td>Tendon removed (planned)</td>
<td>10-year reduced extreme</td>
<td>≤ 0.01 a</td>
</tr>
<tr>
<td>14</td>
<td>S c</td>
<td>In place</td>
<td>Tendon removed (planned)</td>
<td>100-year extreme</td>
<td>≤ 0.001 a</td>
</tr>
<tr>
<td>15</td>
<td>C</td>
<td>In place</td>
<td>Intact</td>
<td>Annual scatter diagram</td>
<td>1</td>
</tr>
<tr>
<td>16</td>
<td>SLE d</td>
<td>In place</td>
<td>Intact</td>
<td>SLE seismic</td>
<td>Varies</td>
</tr>
<tr>
<td>17</td>
<td>DLE d</td>
<td>In place</td>
<td>Intact</td>
<td>DLE seismic</td>
<td>Varies</td>
</tr>
</tbody>
</table>

NOTE: This table is indicative of the types of load cases to be checked, and is not intended to imply adequate number of load cases.

a. Probability of exceedance includes nominal probability of damage or tendon removal occurring.
b. Pile check, if performed, in survival conditions uses reduced safety factor.
c. Survival check with damage or tendon removed is against disconnect (not zero tension) and may be response-based.
d. See Section 4 and API 2A-WSD for definition of SLE, DLE.
e. In all cases, platform configuration should consider both minimum weight and maximum weight variations.
Both API RP 2A and 2T utilize WSD approach for the design of the structure. Notably, the RP 2T 3rd Edition (latest) specified the limit states design approach for the tendon design which was not the case in the previous editions of the document.

The API RP 2T adopts the WSD design methodology for the deck and hull design, and refers to API RP’s 2A, 2U, 2V, and AISC (ASD) standards for the structural elements and states that applicable class society codes may be used for buckling design check.

For structural elements designed for Safety Criteria A, safety factors recommended in API 2A-WSD and AISC should be used for normal design conditions associated. For extreme design conditions associated with Safety Criteria B, the allowable stresses may be increased by one-third.

### 2.2.3 API RP 2FPS

The current first edition of API RP 2FPS 1st Edition issued in March 2001 refers to API RP’s 2A and 2T for the definition of the environmental criteria for GOM floating production systems. The second edition is due for publication in 2011 and will be based on the ISO 19904-1. The document refers to both API RP 2A and 2T valid editions in 2001 for the definition of the applicable environmental conditions. For Category 1 FPSs intended for field development the 100 year return period is specified. Lower criteria is stated to be acceptable for Categories 2 and 3 employed in earlier exploration and drilling phases of the development with durations of less than 5 years for Category 2 and 120 days for

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**Table 2-4 API RP 2T Environnemental Parameters**

<table>
<thead>
<tr>
<th>Environmental Condition</th>
<th>Environmental Parameter</th>
</tr>
</thead>
<tbody>
<tr>
<td>Wind</td>
<td>Mean wind speed, Mean wind direction, Wind power spectral density function</td>
</tr>
<tr>
<td>Wave</td>
<td>Significant wave height, Mean wave period, Wave elevation spectral density function, Mean wave direction, Wave directional spreading function</td>
</tr>
<tr>
<td>Current</td>
<td>Surface current (speed and direction), Current profile (speed and direction)</td>
</tr>
<tr>
<td>Tide</td>
<td>Astronomical tide, Storm surge</td>
</tr>
</tbody>
</table>

---
Category 3. Also lower criteria may be accepted if the platform is evacuated with adequate notice prior to the design storm. API RP 2FPS also refers to API RP 2N for specification of ice loading conditions.

The API RP 2FPS adopts the WSD design methodology for the hull design and refers to API RP’s 2A, 2T, 2U, 2V, the AISC (ASD) standards for the structural elements.

2.3 Loading Conditions - ISO

2.3.1 ISO 19900 – General Requirements

This general standard, applicable to all offshore structures, requires that the structural design be performed with reference to a specified set of limit states. For each limit state, design situations are required to be determined and an appropriate calculation model be established. ISO 19900 divides the limit states into four categories:

- a) Ultimate limit states (ULS)
- b) Serviceability limit state (SLS)
- c) Fatigue limit states (FLS)
- d) Accidental limit states (ALS)

The document gives general description of the environmental conditions that must be considered depending on the type of structure under consideration. These include wind, wave, current, water depth and sea level variations, marine growth, ice and snow, temperature, and other meteorological and oceanographic information such as fog, wind chill, and variability of seawater density.

2.3.2 ISO 19902 – Fixed Steel Offshore Structures

2.3.2.1 Actions for in-place condition

ISO 19902 Clause 9.4.1 states that one of three methods is normally used for defining an environmental action combination that generates the extreme direct action $E_e$ and generally also the extreme action effect, caused by the combined extreme wind, wave and current:

- a) 100 year return period wave height (significant or individual) with associated wave period, wind and current velocities;
- b) 100 year return period wave height and period combined with the 100 year return period wind speed and the 100 year return period current velocity, all determined by extrapolation of the individual parameters considered independently;
- c) any reasonable combination of wave height and period, wind speed and current velocity that results in
  - the global extreme environmental action on the structure with a return period of 100 years, or
  - a relevant action effect (global response) of the structure (e.g. base shear or overturning moment) with a return period of 100 years.

Further discussion of these methods is given in ISO 19901-1 and is summarized herein. Method a) using the 100 year return period wave with associated parameters estimated from correlations has been
used in Gulf of Mexico structures, while b) with 100 year return period wave, 100 year return period
wind, and 100 year return period current has been used in the North Sea and other areas. Method c)
employing the joint 100 year return period action or action effect is a more recent development, suitable
when a database of joint occurrence of wind, wave and current is available.

As stated in ISO 19902, additional considerations should be given to obtaining the extreme direct
action, $E_c$, for locations where there are strong currents that are not driven by local storms. Such
currents can be driven by tides or by deep water currents, such as the Loop Current in the Gulf of
Mexico, Figure 2-3 /21/. In this case, method a) would be acceptable if the storm generated conditions
are the predominant contributors to the extreme global environmental action (action effect) and if the
appropriate “associated” value of tidal and circulation current can be determined. However, method c)
is conceptually more straightforward and preferable. Method b) is the simplest method that ensures an
adequate design environmental action (action effect) since it is usually very conservative compared to
the true 100 year return period global environmental action (action effect).

![Figure 2-3 Loop Current (NOAA) /23/](image)

For some areas, substantial databases are becoming available with which it is possible to establish
statistics of joint probability of occurrence of wind, wave and current magnitudes and directions. When
such a database is available, it can be used to develop environmental conditions based on method c),
which provides the true 100 year return period extreme global environmental action on the structure.

Figure 2-4 reproduced from ISO 19902 shows the parameters that should be accounted for when
calculating the combined wave and current actions on a jacket structure. The figure was adopted from
The corresponding partial action factors to be used in conjunction with the 100 year return period global environmental action (action effect) are required to be determined using structural reliability analysis principles, in order to ensure that an appropriate safety level is achieved. This approach provides more consistent reliability (safety) for different geographical areas than has been achieved by the practice of using separate (marginal) statistics of winds, currents, and waves.

It should be noted that both API and NORSOK adopt similar definition of the extreme design environmental load conditions. However, ISO provides more guidance in this regard.

2.3.2.2 Partial Factor Design Format

The general equation for determining the design action \( (F_d) \) for in-place situations is given in ISO 19902, Equation 9.10-1, and the appropriate partial action factors for each design situation are given in ISO 19902, Table 9.10-1 shown here as Table 2-5:
\[ F_d = \gamma_{f,G1} G_1 + \gamma_{f,G2} G_2 + \gamma_{f,Q1} Q_1 + \gamma_{f,Q2} Q_2 + \gamma_{f,Eo} (E_o + \gamma_{f,D} D_o) + \gamma_{f,Ee} (E_e + \gamma_{f,D} D_e) \]  

(2.1)

where:

- \( G_1, G_2 \) are the permanent actions defined in 9.2;
- \( Q_1, Q_2 \) are the variable actions defined in 9.2;
- \( E_o \) is the environmental action due to the owner-defined operating wind, wave and current parameters;
- \( D_o \) is the equivalent quasi-static action representing dynamic response in accordance with 9.8, but caused by the wave condition that corresponds with that for \( E_o \);
- \( E_e \) is the extreme quasi-static action due to wind, waves and current as defined in 9.4 and taking account of the requirements of 9.5 to 9.7;
- \( D_e \) is the equivalent quasi-static action representing dynamic response defined in 9.8.1

\( \gamma_{f,G1}, \gamma_{f,G2}, \gamma_{f,Q1}, \gamma_{f,Q2} \) are the partial action factors for the various permanent and variable actions discussed in 9.9 and for which values for different design situations are given in Table 9.10-1 (see A.9.10.3.2.1)

\( \gamma_{f,Eo}, \gamma_{f,Ee} \) are partial action factors applied to the total quasi-static environmental action plus equivalent quasi-static action representing dynamic response for operating and extreme environmental conditions, respectively, and for which values for different design situations are given in Table 9.10-1 shown here as Table 2-5;

\( \gamma_{f,E}, \gamma_{f,D} \) are the partial action factors for the environmental actions discussed in 9.9 and for which appropriate values shall be determined by the owner.

All section referenced in above definitions refer to Clauses in ISO 19902.
The partial factors specified in Table 2-5 are almost identical to those given in API RP 2A LRFD for the gravity and variable actions; see Table 2-16 giving a summary of the comparison of the partial factors. However there are subtle differences in definition of actions related to operating environmental conditions and the inclusion of dynamic actions. The ISO 19902 treatise appears to be more comprehensive and logical to apply in design.

Values of the extreme environmental action factor $\gamma_{f,E}$ are given in Annex A (Sec. A.9.9.3.3) of the ISO 19902 for the north-west shelf of Australia (AUS), the UK sector of the North Sea (NS), and the Gulf of Mexico (GOM) for structures manned or unmanned during the design event. For manned installations of exposure level L1 $\gamma_{f,E}$ values of 1.59 for AUS and 1.40 for NS are specified corresponding to a target annual failure probability of $3 \times 10^{-5}$. These factors go down to 1.17 for AUS (and GOM) and 1.09 for NS unmanned or evacuated structures with annual failure probability of $5 \times 10^{-4}$. The latter is associated with L2 exposure category by definition.

It should be stated that ISO 19902 in the same Annex section referenced above specifies also RSR’s (Reserve Strength Factors defined as the ratio of the collapse capacity to the 100 year return period action) for each of the three regions and unmanned/manned conditions. However no guidance is given as to how the RSR is to be calculated. The calculation of RSR has high degree of variability regarding the assumptions to be applied in the pushover ultimate strength analysis.
2.3.2.3 Acceptable safety factors and allowable utilization factors

Table 10.5-1 in ISO 19902, Table 2-6 here, compares the requirements for extreme and abnormal environmental actions. The extreme environmental actions correspond to a minimum return period of 100 years while the abnormal actions have a 10,000 year return period.

Table 2-6 Extreme and Abnormal Conditions

<table>
<thead>
<tr>
<th>Requirement</th>
<th>Extreme environmental actions</th>
<th>Abnormal environmental actions</th>
</tr>
</thead>
<tbody>
<tr>
<td>Governing clause for actions</td>
<td>Clause 9</td>
<td>Clause 10</td>
</tr>
<tr>
<td>Limit state</td>
<td>ULS</td>
<td>ALS</td>
</tr>
<tr>
<td>Return period</td>
<td>100 years</td>
<td>See 10.1.5, default 10 000 years</td>
</tr>
<tr>
<td>Partial action factor</td>
<td>See Clause 9, default 1.35</td>
<td>1.0</td>
</tr>
<tr>
<td>Partial resistance factors</td>
<td>See Clauses 13, 14, 15, 17; generally 1.05 to 1.25 but up to 2.0</td>
<td>1.0</td>
</tr>
<tr>
<td>Wave crest height</td>
<td>Associated with 100 year return event</td>
<td>Associated with abnormal environmental event</td>
</tr>
</tbody>
</table>

2.3.3 ISO 19904-1 Floating Offshore Structures – Part 1: Monohulls, Semi-submersibles and Spars

ISO 19904-1 states that design checks can be undertaken using either the partial factor design format (Limit State Design or LSD) or the WSD format.

2.3.3.1 Partial factors (LSD) format - safety, and allowable utilization factors

Design checking shall be achieved by demonstrating that design values of action effects resulting from factoring the actions do not exceed the design value of the resistance variable being addressed for the limit state under consideration. The partial action factors required for design checks are presented in Table 2-7:
For the ULS, two action combinations are considered: one to reflect gravitational action-dominated conditions; the other to account for environmental action-dominated conditions. In Table 4 of ISO 19904-1, Table 2-7 above, these two combinations are denoted ULS-a and ULS-b, respectively. It should be noted that there are differences between these partial action factors and those proposed in ISO 19902 for fixed structures, Table 2-5. Note the 0.7 factor on the extreme environmental load E in ULS-a and the $0.9\gamma_{f,E}$ in operating situation of Table 2-5. There are differences also in the definition of the design limit states. ISO 19902 utilizes two extreme loading conditions (similar to API RP 2A LRFD) one with unfavourable and another with favourable gravity and variable actions on the response effect under consideration.

For ALS, two conditions are to be assessed. These are denoted in Table 2-7 as pre-ALS and post-ALS. The two accidental limit state conditions represent the structure at the time of the ALS event, and in the damaged condition, respectively.

The partial action factors stated in Table 2-7 for the pre-ALS condition apply to values of accidental event magnitudes that equate to a return period of the accidental event of 10,000 years (i.e. annual probability of exceedance = $10^{-4}$). If the return period exceeds 10,000 years, in some circumstances (such as to ensure a degree of robustness exists in the event of the accidental event occurring), it can be appropriate to combine the accidental event with a feasible environmental event such that the return period of the combined event on a joint probability basis is 10,000 years.

For ULS conditions in relation to steel structures, neither the partial resistance factor $\gamma_r$, nor the partial material factor, $\gamma_m$, is to be less than 1.15. Where the resistance concerns bolted connections and fillet and partial penetration welds, this minimum factor is to be increased to 1.30. Standards adopted for establishing structural strength could require increased partial resistance factors. In such cases, these increased factors shall be used instead of the minimum factors of 1.15 and 1.30, as appropriate.
2.3.3.2  WSD format - safety factors and allowable utilization factors

In the following table, the action combination factors applicable to the WSD format are listed for each limit state and for each combination of action categories.

<table>
<thead>
<tr>
<th>Limit state</th>
<th>Permanent (G)</th>
<th>Variable (Q)</th>
<th>Environmental (E)</th>
<th>Repetitive (R)</th>
<th>Accidental (A)</th>
</tr>
</thead>
<tbody>
<tr>
<td>ULS-a</td>
<td>1.0</td>
<td>1.0</td>
<td>—</td>
<td>—</td>
<td>—</td>
</tr>
<tr>
<td>ULS-b</td>
<td>1.0</td>
<td>1.0</td>
<td>1.0</td>
<td>—</td>
<td>—</td>
</tr>
<tr>
<td>SLS</td>
<td>1.0</td>
<td>1.0</td>
<td>1.0</td>
<td>1.0</td>
<td>—</td>
</tr>
<tr>
<td>Pre-ALS</td>
<td>1.0</td>
<td>1.0</td>
<td>—</td>
<td>—</td>
<td>1.0</td>
</tr>
<tr>
<td>Post-ALS</td>
<td>1.0</td>
<td>1.0</td>
<td>1.0</td>
<td>—</td>
<td>—</td>
</tr>
</tbody>
</table>

For ULS, two action combinations are to be considered: one to reflect the structure located in a calm sea with responses associated with static actions only; the other for the structure subjected to extreme environmental actions combined with relevant static actions. In Table 5 of ISO 19904-1, Table 2-8 above, these combinations are denoted ULS-a and ULS-b, respectively.

For ALS, two conditions are to be assessed. These are denoted in Table 2-8 as pre-ALS and post-ALS, which represent the structure at the time of the accidental event, and in the damaged condition following the accidental event, respectively.

Similar to the LSD format, the WSD action factors stated in Table 2-8 for the pre-ALS condition apply to values of accidental event magnitudes that equate to a return period of the accidental event of 10,000 years. If the return period exceeds 10,000 years, it can be appropriate to combine the accidental event with a feasible environmental event such that the return period of the combined event on a joint probability basis is 10,000 years.

In the design check, the acceptability of a comparison between design values of the action effects and of the strength is conditional upon the action effect \( F_d \) being less than the design strength \( R_d \) reduced by a safety factor greater than unity \( C_{SF} \), or the design strength \( R_d \) multiplied by a fraction less than unity \( \eta \). Thus, the design check may be expressed as
\[ F_d \leq \frac{R_d}{C_{SF}} \]  

(2.2a)

or, alternatively:

\[ F_d \leq \eta R_d \]  

(2.2b)

2.4 Loading Conditions - NORSOK

The principles of the limit state design (LSD) and the definitions of the four limit state categories are the same as given in the ISO 19900 discussed above; Sec. 2.3.1. All identified failure modes must be checked within the respective groups of limit states, i.e. ULS, SLS, FLS and ALS. It is required that the structure possesses sufficient ductility to develop the relevant failure mechanism.

2.4.1 N-003 - Action and Action Effects

The requirements and definitions regarding environmental and loading conditions are given in Section 6.7 of NORSOK N-003. Similar to ISO, NORSOK characteristic values of individual environmental actions are defined by annual exceedance probabilities of \(10^{-2}\) (for ULS) and \(10^{-4}\) (for ALS). The long-term variability of multiple actions is described by a scatter diagram or joint probability density function (PDF) including information about environmental direction. Contour curves or surfaces for more than two environmental parameters can then be derived which give combination of environmental parameters that approximately describe the various actions corresponding to the given exceedance probability. Alternatively, the exceedance probabilities can be referred to the action effects. This is particularly relevant when the direction of the action is an important parameter.

For fixed installations collinear environmental actions are normally most critical, and the action intensities for various types of actions can be selected to correspond to the exceedance probabilities given in Table 2-9 (N-003 Table 4). For other installations action combinations which involve a large difference in action direction need to be addressed.

Table 2-9 presents an alternative option for combining wave, wind, current, ice, snow, earthquake, and sea level elevations in design without resorting to joint probability evaluation or leaving its proper allocation to the operator as stated in ISO 19902, 19904-1 and API RP 2A. As indicated in the table, the ULS associates the 10 year conditions with the 100 year main action and the ALS condition associates both 100 year and 10 year conditions with the 10,000 year main action. This differs from API where only one year conditions are required to be associated with 100 year extreme conditions. This is believed to be a result of considering the Gulf of Mexico to be more benign that the North Sea when it comes to extreme environmental conditions. This assumption was disputed after the severe hurricane seasons of 2004 and 2005, See Figure 2-5 taken from /31/.
### Table 2-9 Action Combinations Annual Probabilities

<table>
<thead>
<tr>
<th>Limit state</th>
<th>Wind</th>
<th>Waves</th>
<th>Current</th>
<th>Ice</th>
<th>Snow</th>
<th>Earthquake</th>
<th>Sea level$^a$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Ultimate</td>
<td>$10^{-2}$</td>
<td>$10^{-2}$</td>
<td>$10^{-1}$</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>$10^{-2}$</td>
</tr>
<tr>
<td>Limit</td>
<td>$10^{-1}$</td>
<td>$10^{-1}$</td>
<td>$10^{-1}$</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>m</td>
</tr>
<tr>
<td>State</td>
<td>-</td>
<td>-</td>
<td>$10^{-2}$</td>
<td>-</td>
<td>$10^{-2}$</td>
<td>-</td>
<td>m</td>
</tr>
<tr>
<td>Accidental</td>
<td>$10^{-3}$</td>
<td>$10^{-2}$</td>
<td>$10^{-1}$</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>m$^*$</td>
</tr>
<tr>
<td>Limit</td>
<td>$10^{-2}$</td>
<td>$10^{-1}$</td>
<td>$10^{-1}$</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>m$^*$</td>
</tr>
<tr>
<td>State</td>
<td>$10^{-1}$</td>
<td>$10^{-4}$</td>
<td>$10^{-4}$</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>m$^*$</td>
</tr>
<tr>
<td></td>
<td>-</td>
<td>-</td>
<td>$10^{-4}$</td>
<td>-</td>
<td>-</td>
<td>$10^{-4}$</td>
<td>m$^*$</td>
</tr>
</tbody>
</table>

$^a$m - mean water level
m$^*$ - mean water level, including the effect of possible storm surge
Seismic response analysis should be carried out for the most critical water level.

### Figure 2-5 Regional Wave Design Criteria

2.4.2 N-001 - Integrity of offshore Structures

In Section 6.2 of NORSOK N-001 defines and specifies the partial action factors. When checking the ULS, SLS, ALS and FLS limit states, the ULS action factors to be used are given in Table 2-10 (N-001...
Table 1). Two ULS conditions are defined in Table 2-10; namely, “a” and “b” that correspond to a case with maximum gravity and variable loads with a reduced environmental load and a condition with realistically reduced gravity and variable loads combined with the maximum (extreme) environmental load, respectively.

The specified action factors are identical to those given in ISO 19904-1, see Table 2-7.

Table 2-10 Action Combinations – Limit States

<table>
<thead>
<tr>
<th>Limit state</th>
<th>Action combinations</th>
<th>Permanent actions (G)</th>
<th>Variable actions (Q)</th>
<th>Environmental actions (E)</th>
<th>Deformation actions (D)</th>
</tr>
</thead>
<tbody>
<tr>
<td>ULS</td>
<td>a</td>
<td>1.3</td>
<td>1.3</td>
<td>0.7</td>
<td>1.0</td>
</tr>
<tr>
<td>ULS</td>
<td>b</td>
<td>1.0</td>
<td>1.0</td>
<td>1.3</td>
<td>1.0</td>
</tr>
<tr>
<td>SLS</td>
<td></td>
<td>1.0</td>
<td>1.0</td>
<td>1.0</td>
<td>1.0</td>
</tr>
<tr>
<td>ALS</td>
<td>Abnormal effect</td>
<td>1.0</td>
<td>1.0</td>
<td>1.0</td>
<td>1.0</td>
</tr>
<tr>
<td>ALS</td>
<td>Damaged condition</td>
<td>1.0</td>
<td>1.0</td>
<td>1.0</td>
<td>1.0</td>
</tr>
<tr>
<td>FLS</td>
<td></td>
<td>1.0</td>
<td>1.0</td>
<td>1.0</td>
<td>1.0</td>
</tr>
</tbody>
</table>

For ship-shaped facilities, the action factor for environmental actions (E) may be reduced to 1.15 for action combination “b” when calculating longitudinal bending moment, if the still water bending moment represents between 20% and 50% of the total bending moment.

For steel structures the material factor specified is 1.15. In the case of geotechnical analyses, the material factor should not normally be lower than 1.25. For piles and anchors the material factor for soil is 1.3 which applies to pile groups. A material factor lower than 1.3 is permitted for individual piles if it can be documented it will not result in adverse behaviour.

2.5 Summary of Environmental Criteria and Loading Comparison

Side-by-side comparison of the requirements specified in the three codes is depicted in Table 2-11. The table shows that the wave kinematics factor is similar in the three standards varying from 0.85 to 0.95 for tropical storms. NORSOK requires 0.95 to be used for North Sea conditions. Marine growth is dependent on the regional conditions with about double the marine growth required in the North Sea compared to the GoM. The same drag and inertia coefficients are specified across the three codes. The conductor shielding factors, wind profiles and gusts, and wind spectra formulations are also the same in all three codes. The wind spatial coherence is the same in API and ISO but is more strict in NORSOK.
requiring 3s gust rather 5s gust for areas with length less than 50m. Also, NORSOK requires the use of the 1-min speed for global wind loads combined with waves. By contrast, both API and ISO allow 1-h wind for static conditions where dynamic aspects are not significant and 1-min wind when dynamic response is important.

The calculation of the wind force is equivalent in the three standards with difference only in presentation in NORSOK giving the force normal to the member instead of in direction of the wind. The current blockage factors are identical in the three standards. With regards to ice loading the API RP 2A and NORSOK N-003 refer to API RP 2N while the ISO 19902 points to the ISO 19906 standard.

With regards to deck clearance requirements, it is noted that all three codes require 1.5m (5 ft) air gap above the 100-year wave crest elevation. As stated in Section 2.2.1, Bulletin 2INT-DG gives guidance on design using the new metocean criteria of 2INT-MET and significantly increased the requirement for deck height elevation by adding 15% for local random wave crest to the maximum wave crest. The 1000-year wave crest was also recommended for robustness consideration. The ISO 19902 gives more details on how to calculate the deck elevation and has an additional criterion of 30% of wave crest elevation as governing clearance if greater than the 1.5m. The NORSOK N-003 and N-004 require a positive air gap for the 10,000 year wave crest in addition to the 1.5m above the 100-year wave crest requirement. It should be noted that there is a large difference between the three codes on this issue. This is important for the probability of failure. The requirement in ISO of 30% increase and in NORSOK of 10,000 year crest will add meters to the air gap. It is therefore not understandable that the old 1.5m requirement is still present in these two codes. For a fixed platform this may be the single requirement that is different in API and ISO (NORSOK) with the largest impact on the probability of failure.

API RP 2T is more comprehensive than RP 2A in defining the loads and load combinations due to the sensitivity of the Tendon Leg Platform to its payload. API 2FPS refers to both API RP 2A and 2T for guidance related to environmental conditions and load definition.

Both API RP 2A and 2T utilize WSD approach for the design of the structure. Notably, the RP 2T 3rd Edition (latest) specified the limit states design approach for the tendon design which was not the case in the previous editions of the document.

The current first edition of API RP 2FPS 1st Edition issued in March 2001 refers to API RP’s 2A and 2T for the definition of the environmental criteria for GOM floating production systems.
### Table 2-11 Comparison Table – Environmental Criteria and Loading Conditions

<table>
<thead>
<tr>
<th>Waves</th>
<th>API RP 2TA/API ZINT-ME1</th>
<th>ISO 19901-1/API 19902</th>
<th>NORSOK N-003</th>
</tr>
</thead>
<tbody>
<tr>
<td>Wave Kinematics factor</td>
<td>0.65 to 0.95 for tropical storms</td>
<td>0.65 to 1.00 for extra-tropical storms</td>
<td>0.95 for North Sea Conditions</td>
</tr>
<tr>
<td>Marine Growth</td>
<td>Table 2.1 for UK Sector</td>
<td>Table C.2 same as NORSOK for Areas offshore Norway, for Offshore southern and central California: 38m are common</td>
<td></td>
</tr>
<tr>
<td>Drag and Inertia Coefficient</td>
<td>smooth C[D]=0.65; C[D]=1.6</td>
<td>smooth C[D]=0.65; C[D]=1.6</td>
<td></td>
</tr>
<tr>
<td>Conductor Shielding Factor</td>
<td>Figure 2.3.1.4, applicable to 0.01°/TDP &gt; 0.5 (extreme waves)</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

**Wind**

<table>
<thead>
<tr>
<th>Wind profiles and Gusts</th>
<th>API RP 2TA, 2.3.2</th>
<th>ISO 19901-1, 1.7.3</th>
<th>NORSOK N-003</th>
</tr>
</thead>
<tbody>
<tr>
<td>B(z)= 0.044 x (Z/32.6)^0 + 0.02 x (Z/32.6)^0.80</td>
<td>0.05 x 1.04 x (Z/32.6)^0.80</td>
<td>0.05 x 1.04 x (Z/32.6)^0.80</td>
<td></td>
</tr>
</tbody>
</table>

**Wind Spectra**

<table>
<thead>
<tr>
<th>Wind spectra</th>
<th>API RP 2TA, 2.3.2</th>
<th>ISO 19901-1, 1.7.4</th>
<th>NORSOK N-003</th>
</tr>
</thead>
<tbody>
<tr>
<td>B(z)= 0.020 x (Z/32.6)^0.5 + 0.02 x (Z/32.6)^0.80</td>
<td>0.017 x (Z/32.6)^0.5 + 0.017 x (Z/32.6)^0.80</td>
<td>0.017 x (Z/32.6)^0.5 + 0.017 x (Z/32.6)^0.80</td>
<td></td>
</tr>
</tbody>
</table>
### Spatial Coherence

APR RP 2A, 2.3.2

- 3 second gusts are appropriate for determining the maximum quasi-static wind load on an individual member.
- 15 second gusts are appropriate for maximum total loads on structures whose maximum horizontal dimension is less than 164 ft (50 m).
- 5 minute sustained wind is appropriate for total static superstructure wind loads associated with maximum wave forces for structures that respond dynamically to wind excitation but which do not require a full dynamic wind analysis. One-hour sustained wind is appropriate for total static superstructure wind forces associated with maximum wave forces.

ISO 19902, 9.7

- F = \( \mu \cdot \alpha \cdot U \cdot C_{sh} \cdot A \)
- F = wind force; \( \mu \) = mass density of air (at standard temperature and pressure), 1.226 kg/m³; \( \alpha \) = wind speed; \( C_{sh} \) = shape coefficient; A = Area of object; normal to wind velocity direction

### Wind speed and force relationship

APR RP 2A, 2.3.2

- \( F = (\mu/2) \alpha \cdot U \cdot C_{sh} \cdot A \)
- \( F = \) wind force; \( U \) = wind speed; \( \alpha \) = wind speed direction.

ISO 19902, 9.7

- \( F = (\mu/2) \alpha \cdot U \cdot C_{sh} \cdot A \)
- \( F = \) wind force; \( U \) = wind speed; \( \alpha \) = wind speed direction.

### Shape Coefficient

#### Beams - 1.5

<table>
<thead>
<tr>
<th>Beam</th>
<th>Overall projected area of platform</th>
<th>1.0</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Flat walls of building - 1.50</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Overall projected area of structure - 1.00</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Cylinders - Smooth, Re &lt; 5 x 10⁶, 0.65</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Smooth, Re &lt; 5 x 10⁶, 1.20</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Rough, all Re, 1.05</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Covered with ice, all Re, 1.20</td>
<td></td>
</tr>
</tbody>
</table>

#### Cylinders

- Cₜ = 0.65 for Reynolds number > 5 x 10⁶
- Cₜ = 1.20 for Reynolds number < 5 x 10⁶

### Current Blockage Factor

<table>
<thead>
<tr>
<th>No. of Legs</th>
<th>Heading</th>
<th>Factor</th>
</tr>
</thead>
<tbody>
<tr>
<td>3</td>
<td>all</td>
<td>0.9</td>
</tr>
<tr>
<td>4</td>
<td>end-on</td>
<td>0.8</td>
</tr>
<tr>
<td></td>
<td>diagonal</td>
<td>0.85</td>
</tr>
<tr>
<td>6</td>
<td>end-on</td>
<td>0.75</td>
</tr>
<tr>
<td></td>
<td>diagonal</td>
<td>0.65</td>
</tr>
<tr>
<td>8</td>
<td>end-on</td>
<td>0.7</td>
</tr>
<tr>
<td></td>
<td>diagonal</td>
<td>0.86</td>
</tr>
<tr>
<td></td>
<td>broadside</td>
<td>0.8</td>
</tr>
</tbody>
</table>

### Snow and Ice

- Section 2.3.5 Ice, Refer to API Bulletin 2N.
- Section 6.4 with details data

Details refer to ISO 19906 - Arctic Offshore Structures:
- 19901-1 Annex C2.8.3 - UK sector
- 19901-1 Annex C6 - East Coast of Canada

Show 0.5 Kpa for the entire Norwegian Continental Shelf.
<table>
<thead>
<tr>
<th>Deck Clearance</th>
<th>API RP 2A, 2.3.4</th>
</tr>
</thead>
<tbody>
<tr>
<td>Considered guideline wave heights with a nominal return period of 100 years, together with the applicable wave theories and wave steepness should be used to compute wave crest elevations above storm water level, including guideline storm tide.</td>
<td></td>
</tr>
<tr>
<td>A safety margin, or air gap, of at least 5 feet should be added to the crest elevation to allow for platform settlement, water depth uncertainty, and for the possibility of extreme waves in order to determine the minimum acceptance elevation of the bottom beam of the lowest deck to avoid waves striking the deck.</td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>ISO 19902 A 6.3.3.2</th>
</tr>
</thead>
<tbody>
<tr>
<td>If storm surge is not expected to occur at the same time as the abnormal wave crest, deck elevation h = (a^2 + g^2 + f^2)^(1/2) + f</td>
</tr>
<tr>
<td>If storm surge is expected to occur at the same time as the abnormal wave crest, h = [(a + s)^2 + f^2]^(1/2) + f</td>
</tr>
<tr>
<td>Where: a is the abnormal wave crest height; s is the extreme storm surge; f is the maximum elevation of the tide relative to the mean sea level; f is the expected sum of subsidence, settlement and sea level rise over the design service life of the structure.</td>
</tr>
<tr>
<td>For deep and intermediate water depth a can be approximated to:</td>
</tr>
<tr>
<td>a &gt; 1.3 a_{20}</td>
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<td>a &gt; a_{20} + 1.5 m</td>
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<td>a_{20} is the extreme wave crest height with a return period of 100 years</td>
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<tr>
<th>Earthquake</th>
<th>API RP 2A, 2.3.6</th>
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<tbody>
<tr>
<td>Separate comparison table provided</td>
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<th>ISO 19902 Section 11 &amp; ISO 19901.2</th>
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<td>Separate comparison table provided.</td>
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<tr>
<th>Regional Design Metocean Criteria</th>
<th>Section 2.3.4 Hydrodynamic Force Guidelines for US Waters, and 2.3.4c Hurricanes to be replaced by API 2INT-MET</th>
</tr>
</thead>
<tbody>
<tr>
<td>Annex C Regional information including North-west Europe, West coast of Africa, US Gulf of Mexico, US West coast of California and East coast of Canada</td>
<td>NORSOK N-003 mainly focuses on North-West Coast of Europe and refers to ISO.</td>
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**Note:**
1. API RP 2A focuses on GoM and hurricane criteria(2.3.4c & 17.5.2a are replaced by API 2INT-MET).
2. ISO 19901-1 provides environmental guidelines for the regions all over the world including North-west Europe, West coast of Africa, US Gulf of Mexico, US Coast of California and East coast of Canada;
3. NORSOK N-003 mainly focuses on North west coast of Europe and refers to ISO;
4. Deck clearance criteria:
   a) 1.5 m or 5 ft air gap required for API/ISO;
   b) a different way of calculation the maximum wave crest; API and ISO use the 100yr return period waves; NORSOK provides two options: ULS: 10^2 wave crest + 1.5 m or free passage of 10^2 wave without hitting deck members.
The differences between WSD and LRFD design philosophies were briefly discussed in Section 1.4. It was explained there that WSD methodology suffers from the inability to allocate different safety factors to different loads depending on their uncertainty level. However the WSD is simpler in that it requires only one number as the safety factor. By contrast, the LRFD, or the LSD, methods have to define load/action factors plus one resistance factor for each design condition/limit state.

The operating WSD acceptable stress is normally set as 0.6 $F_y$ (where $F_y$ = yield strength) which would be equivalent to 1.45 load factor and 1.15 resistance factor. Therefore if the unfactored loads are the same, the WSD design should be more conservative. For the extreme condition the API 1/3rd increase in allowable stress leads to 0.8 $F_y$ as the acceptable stress and equivalent uniform load factor of 1.09 with 1.15 resistance factor indicating that the LRFD approach would be considerably more conservative for any significant environmental loading condition.

In order to calculate the load factors an acceptable failure probability is specified in the standards in the form of annual probability or reliability index as noted in Table 2-12, for API Section 17 and NORSOK, Table 2-13 from ISO 19906 which is also applicable to ISO19902, Table 2-14 from DNV CN 30.6 (2002), and Table 2-15 from DNV/Riso guidelines for wind turbine design. The reliability index $\beta$ is defined as

$$\beta = -\Phi^{-1}(P_f),$$

(2.3)

where $\Phi^{-1}$ is the inverse normal distribution function.

Table 2-12 does not represent any target reliability, but is a comparison of the probability of failure between API section 17 and NORSOK for two cases of uncertainty in the resistance formulation. This should not therefore be viewed as target reliability for NORSOK but only as an indication to that effect. The shown annual $P_f$ was calculated using probabilistic analysis software (PROBAN) with a limit state function that defines failure as action exceeding resistance.

These are shown to be very similar across the standards. The load/action factors are calculated using a calibration procedure described in; e.g., ISO 2394 (1998). The calibration procedure involves many assumptions and approximations that are not spelled out in the codes. However, the acceptability of the proposed factors is demonstrated by application to actual structures that exhibit adequate performance under actual design environmental conditions.

Table 2-16 compares the load/action factors specified in API, ISO, and NORSOK standards. Again this table presents a side-by-side comparison of the three codes. Because API RP 2A 21st Edition is a WSD code, the 1993 API RP 2A LRFD was used for the comparison with the ISO and NORSOK codes which use the LSD which is same as LRFD.
As noted earlier in Sections 2.2, 2.3 and 2.5, the load/action factors are similar in 2A LRFD and ISO 19902. It is our understanding that the API document was utilized as a starting point for the ISO standards development that started in the 1990’s. Therefore the ISO document have improved considerably on the 2A LRFD document not only in providing more guidance to the designer but also in correcting and clarifying several issues that existed in 2A LRFD such as the separation of the inertia component of the load with different load factor and the definition of an operating environmental condition.

| Table 2-12 Annual Pf in API Sec. 17 and NORSOK for unmanned and manned platforms |
|---------------------------------|-----------------|-----------------|
| CoV(Xf) Capacity | Unmanned | Manned | Unmanned | Manned (ULS) | Manned (ALS) |
| 0.2 | 5 \times 10^{-2} | 1 \times 10^{-3} | 1 \times 10^{-4} | 5 \times 10^{-5} | 2 \times 10^{-5} |
| 0.1 | 3 \times 10^{-2} | 3 \times 10^{-4} | 2 \times 10^{-4} | 6 \times 10^{-5} | 2 \times 10^{-5} |

*) Calibrated value

| Table 2-13 ISO 19906 Reliability Targets for ULS and ALS |
|---------------------------------|-----------------|
| Exposure Level | Maximum Acceptable Annual Failure Probability |
| L1* | 1.0 \times 10^{-5} |
| L2* | 1.0 \times 10^{-4} |
| L3* | 1.0 \times 10^{-3} |

*L1=high consequence/manned non-evacuated, L2=Medium consequence/manned evacuated or unmanned or Manned Evacuated with low consequence, and L3= low consequence unmanned structures.

| Table 2-14 DNV Classification Notes 30.6 (1992) Annual Pf and Target Reliability Indices |
|---------------------------------|-----------------|
| Class of Failure | Less Serious Consequence | Serious Consequence |
| I. Redundant structure | P_F = 10^{-3} | P_F = 10^{-4} |
| | $\beta = 3.09$ | $\beta = 3.71$ |
| II. Significant warning prior to occurrence of failure in a non-redundant structure | P_F = 10^{-4} | P_F = 10^{-5} |
| | $\beta = 3.71$ | $\beta = 4.26$ |
| III. No warning before the occurrence of failure in a non-redundant structure | P_F = 10^{-5} | P_F = 10^{-6} |
| | $\beta = 4.26$ | $\beta = 4.75$ |
Table 2-15 Guidelines for Design of Wind Turbines, DNV/Riso, 2002*

<table>
<thead>
<tr>
<th>Failure type</th>
<th>Less serious LOW SAFETY CLASS (small possibility for personal injuries and pollution, small economic consequences, negligible risk to life)</th>
<th>Serious NORMAL SAFETY CLASS (possibilities for personal injuries, fatalities, pollution, and significant economic consequences)</th>
<th>Very serious HIGH SAFETY CLASS (large possibilities for personal injuries, fatalities, significant pollution, and very large economic consequences)</th>
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</table>
| Ductile failure with reserve capacity (redundant structure) | $P_F = 10^{-3}$  
$\beta_T = 3.09$ | $P_F = 10^{-4}$  
$\beta_T = 3.72$ | $P_F = 10^{-5}$  
$\beta_T = 4.26$ |
| Ductile failure with no reserve capacity (significant warning before occurrence of failure in non-redundant structure) | $P_F = 10^{-4}$  
$\beta_T = 3.72$ | $P_F = 10^{-5}$  
$\beta_T = 4.26$ | $P_F = 10^{-6}$  
$\beta_T = 4.75$ |
| Brittle failure (no warning before occurrence of failure in non-redundant structure) | $P_F = 10^{-5}$  
$\beta_T = 4.26$ | $P_F = 10^{-5}$  
$\beta_T = 4.75$ | $P_F = 10^{-7}$  
$\beta_T = 5.20$ |


It should be noted that the probability of failure though defined in design codes as the probability the load/action exceeds the strength/resistance to avoid failure; the code rarely defines the failure itself. As noted in this report, even in Limit State Design philosophy, the load/action factors and resistance factors ensure the safety of the structure under extreme environmental conditions. The uncertainty in the loading would lead to ultimate strength response of the structure.
## Table 2-16 Load/Action Factors

<table>
<thead>
<tr>
<th>API RP 2A - NORSOK Section C</th>
<th>ISO 19900:2010</th>
<th>Part A: Partial action factors for basic situations and exposure level 1</th>
<th>Part B: Partial action factors for basic situations and exposure level 2</th>
<th>Part C: Partial action factors for basic situations and exposure level 3</th>
<th>NORSOK N-001</th>
<th>N-003</th>
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<tr>
<td>Design Conditions</td>
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<td>Limit State</td>
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**Legend:**
- **NORSOK N-001:** NORSOK Offshore Structural Standards
- **N-003:** NORSOK Offshore Structural Standards
- **Table 2-16:** Load/Action Factors
- **API RP 2A - NORSOK Section C:** API RP 2A - NORSOK Offshore Structural Standards
- **ISO 19900:2010:** ISO 19900:2010 - Load and Action Effects in Structures

---

### Determination of Load/Action Factors

1. **Typically:** The load in the offshore structure is determined by the operating conditions in the Gulf of Mexico.
2. **Sensitivity:** The load in the offshore structure is determined by the operating conditions in the Gulf of Mexico.
3. **Assumptions:** The load in the offshore structure is determined by the operating conditions in the Gulf of Mexico.
4. **Determinations:** The load in the offshore structure is determined by the operating conditions in the Gulf of Mexico.
5. **Methods:** The load in the offshore structure is determined by the operating conditions in the Gulf of Mexico.
6. **Implementation:** The load in the offshore structure is determined by the operating conditions in the Gulf of Mexico.

---

### Table 2-16: Load/Action Factors

<table>
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<tr>
<th>Condition</th>
<th>Load Factor</th>
<th>Methodology</th>
<th>Assumptions</th>
<th>Implementation</th>
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3 STRUCTURAL STEEL AND CONNECTIONS DESIGN

3.1 Tubular Members

The main differences among API WSD, API LRFD, ISO, and NORSOK are illustrated in this section. The comparison is also made through case studies presented in Section 11 herein.

a. Material Validity

ISO 19902 and NORSOK N-004 consider steel with yield strength of up to 500 MPa whereas in the API codes this limit is 414 MPa. It appears that API will adopt the 500 MPa limit on yield strength in the future.

b. Axial Tension

API LRFD, ISO and NORSOK formulations for axial tension, bending and hydrostatic pressure are identical. The allowable axial tensile stress in API WSD is naturally the lowest among all four codes because it is based on WSD methodology employing actual operating or extreme loads without any load factors (i.e. load factor = 1.0). The second lowest is given in NORSOK because it adopts a material factor of 1.15 which is higher than the resistance factor of 1.05 in ISO (same as 1/0.95 in API LRFD).

c. Overall Column Buckling

The same level of axial compression capacity is provided in both the API LRFD and the ISO. The range of material factors in NORSOK is 1.15 – 1.45, which is dependent on elastic local buckling strength and elastic hoop buckling strength.

d. Local Buckling

1. Local buckling check is based on only geometric parameters in API WSD whereas in API LRFD, ISO and NORSOK it depends on geometry and elastic modulus of members.

2. In local buckling equations, the API allows an upper limit of D/t ratio of up to 300 whereas ISO and NORSOK limit D/t to a maximum of only 120 which means that NORSOK is significantly more conservative. It should be remembered that NORSOK assumes that the platforms will be manned during an extreme environmental event.

e. Bending

1. The bending stress equations in API LRFD, ISO and NORSOK contain elastic section modulus, plastic section modulus and yield strength whereas API WSD equations only contain the yield strength. This is because the WSD methodology limits the stress to a fraction of the yield
whereas the LSD and LRFD approaches allow full plasticity in the section and therefore allow the section to go beyond first yield.

2. The same level of bending capacity is provided in the API LRFD as well as in the ISO. The range of material factors in NORSOK is 1.15 – 1.45, which is dependent on elastic local buckling strength and elastic hoop buckling strength. This is considerably higher than the resistance factor of 1.05 (1/0.95 in API LRFD) and therefore NORSOK is more conservative in capacity evaluation.

f. Hydrostatic Pressure

1. Critical hoop buckling stress $F_{hc}$ in API WSD is different from the other three codes. In API WSD, design formulae for critical hoop buckling strength are provided for four elastic stress ranges. The equations in API LRFD, ISO and NORSOK are identical. ISO and NORSOK provide three ranges of elastic hoop buckling strength for whereas API LRFD has two such ranges.

2. The formula for elastic hoop buckling strength is same in all four codes. However, in API WSD the elastic buckling coefficient $C_h$ is provided for five ranges, whereas API LRFD, ISO and NORSOK include four ranges for this parameter.

2. The formula for elastic hoop buckling strength is same in all four codes. However, in API WSD the elastic buckling coefficient $C_h$ is provided for five ranges, whereas API LRFD, ISO and NORSOK include four ranges for this parameter.

g. Shear

Shear stress factors in API LRFD and ISO 19902 are same, whereas NORSOK specifies reduced value due to the conservatism associated with the material factor as discussed earlier in this section. The API WSD allowable shear stress is much lower because it is to be compared with unfactored operating or extreme (with the 1/3 allowable stress increase) load conditions.

h. Combined Loads without Hydrostatic Pressure

i) Axial Tension and Bending

1. The formulae in all four codes are different. API LRFD adopts a cosine form equation. API WSD and ISO use linear formulae.

   ii) Axial Compression and Bending

1. The formulae from all four codes are different. As in i) above, API LRFD utilizes a cosine form equation while API WSD and ISO use a linear form.

2. When axial compressive stress is small ($fa/Fa <= 0.15$), API WSD provides an alternative equation.

3. All four codes provide the same formulae for moment reduction factor $C_m$. 

4. Effective Length Factor for Jacket brace buckling check exhibit differences as shown in

5. Table 3-2. NORSOK and ISO are same while API WSD and LRFD give slightly higher factor for X-brace longer segment length (0.9 vs. 0.8) and main diagonals (0.8 vs. 0.7). Also API WSD and LRFD give effective length factors for deck truss web members.

i. Combined Loads with Hydrostatic Pressure
   i) Axial Tension and Bending

   1. API WSD and LRFD provide the same formulae. However, the safety factor on resistance provided in API WSD is by definition higher than that in the API LRFD (1.67-2.00 vs. 1/0.95=1.05 and 1/0.80 = 1.25).

   2. Both ISO and NORSOK provide similar format. The only difference between these two codes is that the partial resistance factor in ISO is 1.05 for combined tension and bending and the material factor in NORSOK is in the range of 1.15 to 1.45.

   3. There are two methods provided in NORSOK for design axial stress in tension and compression respectively. In Method A, design axial stress excludes the effect of capped-end axial compression arising from external hydrostatic pressure. In Method B, the calculated member axial stress includes the effect of the hydrostatic capped-end axial stress.

   ii) Axial Compression and Bending

   1. API LRFD has a cosine format equation. NORSOK provides two methods for the combined stress formulae as noted in i) Axial Tension and Bending, item 3 above.

   2. The basic formulae in ISO and NORSOK are identical.

   3. As in i) Axial Tension and Bending, item 3 above, two methods A and B are provided in NORSOK for design axial stress in tension and compression respectively excluding or including the effect of capped-end axial compression arising from external hydrostatic pressure.

When the compressive stress combination is greater than half of hoop compressive stress, the formulae in the four codes are identical.

Interaction formulae for shear plus bending moment and shear plus bending moment and torsional moment are provided in NORSOK.
3.2 Tubular Joints

As shown in Table 3-5 the following may be noted:

1. API LRFD requires that the connections at the ends of tension and compression members develop the strength required by design loads, but not less than 50% of the effective strength of the member. There is no validity range provided in the code.

2. Formulae for joint basic capacity are identical in the four codes, but the API LRFD moment capacity equation includes the numerical factor of 0.8 on d in equation for $M_{uj}$.

3. For strength check, cosine format is presented in API LRFD. The formula is of the same format in API WSD, ISO and NORSOK. However, an additional formula is provided in ISO for critical joints to ensure that the joint strength exceeds the brace member strength. This is a subject of discussion in the ISO committee regarding the implication on design and the actual need for this conservatism.

4. In ISO and NORSOK, the strength factor $Q_u$ is identical. Different values are suggested in API WSD and API LRFD.

5. Formulations for chord load factor $Q_f$ in API WSD are very different from those in API LRFD, ISO and NORSOK. The same equation is used in API LRFD, ISO and NORSOK, but the coefficients “C” are different among the three codes.

3.3 Code Comparison Summary

The API RP 2A WSD, API RP 2A LRFD, ISO 19902 and NORSOK N-004 provisions for checking the adequacy of tubular members are similar in that all four codes give formulations for each load effect type acting alone and for all load effects acting in combination.

Table 3-1, Table 3-2, Table 3-3, and Table 3-4 summarize and compare the provisions of the four codes. Many of the provisions shown are similar or equivalent across all four codes. For instance the API LRFD, ISO and NORSOK formulations for axial tension, bending and hydrostatic pressure are identical. The most significant differences lie with axial compression, particularly with respect to local buckling, and with some of the combined effect interaction equations.

The overall column buckling formula in API WSD uses the AISC formulation and differs from API LRFD, ISO and NORSOK which are LSD or LRFD based. The API LRFD, ISO and NORSOK use a similar formula but employ different coefficients. The same capacity is given by API LRFD and ISO, while a lower capacity is given by NORSOK meaning that NORSOK is more conservative.

The local buckling strengths in API WSD and API LRFD are given by the same equations and, when expressed as a proportion of the yield stress, is only a function of geometry parameters. The local buckling strengths in ISO and NORSOK are given by the same equations and are noted as being a function of material as well as geometric properties.
The interaction formulae in API WSD, ISO and NORSOK are linear combinations, whereas the API LRFD used a cosine term in the interaction equation. The API 2A LRFD code is currently suspended and will be replaced by the API RP 2A 23rd Edition which will use the ISO 19902 as basis similar to other API RP’s currently being produced. The original intention of publishing only an API “wrapper” and attaching the ISO document to it has now been changed to reproducing the ISO standard edited to incorporate GOM and US west Coast specific requirements.
### Table 3-1 Tubular Member Design Check -1

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<tr>
<th>Stress/Parameter</th>
<th>API WSD</th>
<th>API LRFD</th>
<th>ISO</th>
<th>NORSOK</th>
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<td>$F_{C} = 0.6F_{a}$</td>
<td>$F_{C} = a_{r}F_{b}$</td>
<td>$F_{C} = a_{r}F_{b}$</td>
<td>$F_{C} = 0.6F_{a}$</td>
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<td>$F_{C} = a_{r}F_{b}$</td>
<td>$F_{C} = 0.6F_{a}$</td>
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<td>$D_{n} = 60$</td>
<td>$D_{n} = 60$</td>
<td>$D_{n} = 60$</td>
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<td>$F_{C} = 1.05F_{a} \sigma_{y} = 0.85$</td>
<td>$F_{C} = 1.05F_{a} \sigma_{y} = 0.85$</td>
<td>$F_{C} = 1.05F_{a} \sigma_{y} = 0.85$</td>
<td>$F_{C} = 1.05F_{a} \sigma_{y} = 0.85$</td>
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<td>$F_{C} = 0.08474D$</td>
<td>$F_{C} = 0.08474D$</td>
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**DNV Reg. No.:** EP034373 - 2011 - 01

**Revision No.:** 1

**Date:** 2012-01-12
### Table 3-2 Effective Length Factor

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<th>Situation</th>
<th>API WSD / API LRFD</th>
<th>ISO 19902</th>
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<td>Reduction Factor Cm</td>
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<td>Cm</td>
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<tr>
<td>Braced</td>
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<td>Braced Legs</td>
<td>Superstructure Legs</td>
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<td>a</td>
<td>a</td>
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<tr>
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<td>Jacket legs and piling</td>
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<td>Ungrouded Jacket Legs</td>
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<td>Ungrouded Piling Between Shim</td>
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<td>b</td>
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<td>Primary diagonals and horizontals</td>
<td>Structural brace members</td>
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<td>Primary diagonals and horizontals</td>
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<td>b or c</td>
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<td>K-Braces</td>
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<td>Longer segment length</td>
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<td>Secondary Horizontals</td>
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<td>Deck Truss Chord Members</td>
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<td>a, b or c</td>
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<tr>
<td>K Use Effective Length Alignment Chart in Commentary of AISC</td>
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<td>a</td>
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<tr>
<td>0.6 - 0.4 (M/My2), but not less than 0.4, nor more than 0.85</td>
<td>0.6 - 0.4 (M/My2), but shall not be larger than 0.85</td>
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<td>1 - 0.4FL/F2, or 0.85, whichever is less</td>
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<td>Use Effective Length Alignment in Clause 12</td>
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**Notes:**
- K and Cm are reduction factors specific to each scenario.
- The structural elements listed follow the design criteria outlined in the respective standards.
Table 3-3 Tubular Member Design Check (Contd.)

<table>
<thead>
<tr>
<th>Stress/Parameter</th>
<th>API WSD</th>
<th>API LRFD</th>
<th>ISO</th>
<th>NORSOK</th>
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<td>$f_{sh} = 0.025t_{w} F_{u}/3F_{y}$</td>
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<td>$F_{sh} = 0.025t_{w} F_{u}/3F_{y}$</td>
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<td>$M = L/D (2D)^{1/2}$</td>
<td>$M = L/D (2D)^{1/2}$</td>
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</tr>
<tr>
<td>$\sigma_{(2D)}^{(1)} = \sigma_{(2D)}^{(1)} + \tau_{(2D)}^{(1)} + \tau_{(2D)}^{(1)} = 1.0$</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
### Table 3-4 Tubular Member Design Check (Contd.)

<table>
<thead>
<tr>
<th>Parameter</th>
<th>API</th>
<th>LRFD</th>
<th>ISO</th>
<th>NORSOK</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>4.3.3</td>
<td>4.3.3</td>
<td>4.3.3</td>
<td>4.3.3</td>
</tr>
<tr>
<td></td>
<td>Compression, Bending and Hydrostatic Pressure</td>
<td>Compression, Bending and Hydrostatic Pressure</td>
<td>Compression, Bending and Hydrostatic Pressure</td>
<td>Compression, Bending and Hydrostatic Pressure</td>
</tr>
<tr>
<td></td>
<td>$f_{c} = f_{c0} / f_{c1}, f_{b} = f_{b0} / f_{b1}, f_{s} = f_{s0} / f_{s1}$</td>
<td>$f_{c} = f_{c0} / f_{c1}, f_{b} = f_{b0} / f_{b1}, f_{s} = f_{s0} / f_{s1}$</td>
<td>$f_{c} = f_{c0} / f_{c1}, f_{b} = f_{b0} / f_{b1}, f_{s} = f_{s0} / f_{s1}$</td>
<td>$f_{c} = f_{c0} / f_{c1}, f_{b} = f_{b0} / f_{b1}, f_{s} = f_{s0} / f_{s1}$</td>
</tr>
<tr>
<td>$f_{c}$ = $f_{c0} / f_{c1}$</td>
<td>$f_{b} = f_{b0} / f_{b1}$</td>
<td>$f_{s} = f_{s0} / f_{s1}$</td>
<td>$f_{c} = f_{c0} / f_{c1}$</td>
<td>$f_{b} = f_{b0} / f_{b1}$</td>
</tr>
<tr>
<td>$f_{c} = 0.85 f_{c0}$</td>
<td>$f_{b} = 0.85 f_{b0}$</td>
<td>$f_{s} = 0.85 f_{s0}$</td>
<td>$f_{c} = 0.85 f_{c0}$</td>
<td>$f_{b} = 0.85 f_{b0}$</td>
</tr>
<tr>
<td>$f_{c} = 0.5 f_{c0}$</td>
<td>$f_{b} = 0.5 f_{b0}$</td>
<td>$f_{s} = 0.5 f_{s0}$</td>
<td>$f_{c} = 0.5 f_{c0}$</td>
<td>$f_{b} = 0.5 f_{b0}$</td>
</tr>
<tr>
<td>$f_{c}$ = 0.25</td>
<td>$f_{b}$ = 0.25</td>
<td>$f_{s}$ = 0.25</td>
<td>$f_{c}$ = 0.25</td>
<td>$f_{b}$ = 0.25</td>
</tr>
<tr>
<td>$f_{c} = 1.25$</td>
<td>$f_{b} = 1.25$</td>
<td>$f_{s} = 1.25$</td>
<td>$f_{c} = 1.25$</td>
<td>$f_{b} = 1.25$</td>
</tr>
</tbody>
</table>

Note: The above table provides a comparison of design criteria for tubular members under compression, bending, and hydrostatic pressure as specified by API, LRFD, ISO, and NORSOK standards. The table includes formulas and methods for calculating design parameters to ensure structural integrity and safety in offshore applications.
### Table 3-5 Tubular Joint Check

<table>
<thead>
<tr>
<th></th>
<th>API</th>
<th>LRFD</th>
<th>ISO 19902</th>
<th>NORSOK N-004</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>WSD</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td><strong>4.3.1 Validity Range</strong></td>
<td>0.2 - 3 x 10^6</td>
<td>0.2 - 3 x 10^6</td>
<td>0.2 - 3 x 10^6</td>
<td>0.2 - 3 x 10^6</td>
</tr>
<tr>
<td><strong>0.2 - 3 x 10^6</strong></td>
<td>10 x 10^6</td>
<td>10 x 10^6</td>
<td>10 x 10^6</td>
<td>10 x 10^6</td>
</tr>
<tr>
<td><strong>0.2 - 3 x 10^6</strong></td>
<td>10 x 10^6</td>
<td>10 x 10^6</td>
<td>10 x 10^6</td>
<td>10 x 10^6</td>
</tr>
<tr>
<td><strong>4.3.1 TUBULAR JOINTS</strong></td>
<td>Ultimate Capacity</td>
<td>P_{u} = 0.75P_{L}G_{c}G_{t}</td>
<td>P_{u} = 0.75P_{L}G_{c}G_{t}</td>
<td>P_{u} = 0.75P_{L}G_{c}G_{t}</td>
</tr>
<tr>
<td><strong>P_{u} = 0.75P_{L}G_{c}G_{t}</strong></td>
<td>G_{c} = 0.75P_{L}G_{c}G_{t}</td>
<td>G_{c} = 0.75P_{L}G_{c}G_{t}</td>
<td>G_{c} = 0.75P_{L}G_{c}G_{t}</td>
<td></td>
</tr>
<tr>
<td><strong>4.3.2 Basic Capacity</strong></td>
<td>P_{u} = 0.75P_{L}G_{c}G_{t}</td>
<td>P_{u} = 0.75P_{L}G_{c}G_{t}</td>
<td>P_{u} = 0.75P_{L}G_{c}G_{t}</td>
<td></td>
</tr>
<tr>
<td><strong>P_{u} = 0.75P_{L}G_{c}G_{t}</strong></td>
<td>G_{c} = 0.75P_{L}G_{c}G_{t}</td>
<td>G_{c} = 0.75P_{L}G_{c}G_{t}</td>
<td>G_{c} = 0.75P_{L}G_{c}G_{t}</td>
<td></td>
</tr>
<tr>
<td><strong>4.3.3 Stress Check</strong></td>
<td>G_{c} = 0.75P_{L}G_{c}G_{t}</td>
<td>G_{c} = 0.75P_{L}G_{c}G_{t}</td>
<td>G_{c} = 0.75P_{L}G_{c}G_{t}</td>
<td></td>
</tr>
<tr>
<td><strong>G_{c} = 0.75P_{L}G_{c}G_{t}</strong></td>
<td>G_{c} = 0.75P_{L}G_{c}G_{t}</td>
<td>G_{c} = 0.75P_{L}G_{c}G_{t}</td>
<td>G_{c} = 0.75P_{L}G_{c}G_{t}</td>
<td></td>
</tr>
<tr>
<td><strong>4.3.3 Strength Factor Q_{b}</strong></td>
<td>Values for Q_{b}</td>
<td>Values for Q_{b}</td>
<td>Values for Q_{b}</td>
<td></td>
</tr>
<tr>
<td><strong>Q_{b} = 0.75(P_{L}G_{c}G_{t})</strong></td>
<td>for 2 x 10^6</td>
<td>for 2 x 10^6</td>
<td>for 2 x 10^6</td>
<td></td>
</tr>
<tr>
<td><strong>Q_{b} = 1.0</strong></td>
<td>for 2 x 10^6</td>
<td>for 2 x 10^6</td>
<td>for 2 x 10^6</td>
<td></td>
</tr>
<tr>
<td><strong>Q_{b} = 1.0</strong></td>
<td>for 2 x 10^6</td>
<td>for 2 x 10^6</td>
<td>for 2 x 10^6</td>
<td></td>
</tr>
<tr>
<td><strong>Where G_{c} = 0.75P_{L}G_{c}G_{t}</strong></td>
<td>for 2 x 10^6</td>
<td>for 2 x 10^6</td>
<td>for 2 x 10^6</td>
<td></td>
</tr>
<tr>
<td><strong>4.3.3 Load Case Value for Q_{b}</strong></td>
<td>Q_{b} = 0.75P_{L}G_{c}G_{t}</td>
<td>Q_{b} = 0.75P_{L}G_{c}G_{t}</td>
<td>Q_{b} = 0.75P_{L}G_{c}G_{t}</td>
<td></td>
</tr>
<tr>
<td><strong>Q_{b} = 0.75P_{L}G_{c}G_{t}</strong></td>
<td>for 2 x 10^6</td>
<td>for 2 x 10^6</td>
<td>for 2 x 10^6</td>
<td></td>
</tr>
<tr>
<td><strong>Q_{b} = 1.0</strong></td>
<td>for 2 x 10^6</td>
<td>for 2 x 10^6</td>
<td>for 2 x 10^6</td>
<td></td>
</tr>
<tr>
<td><strong>Where G_{c} = 0.75P_{L}G_{c}G_{t}</strong></td>
<td>for 2 x 10^6</td>
<td>for 2 x 10^6</td>
<td>for 2 x 10^6</td>
<td></td>
</tr>
</tbody>
</table>

**Notes:**
- TV joints under brace axial loading
- TV joints under brace moment loading
- TV joints under brace axial loading
- TV joints under brace moment loading
- TV joints under brace axial loading
- TV joints under brace moment loading
- TV joints under brace axial loading
- TV joints under brace moment loading
- TV joints under brace axial loading
- TV joints under brace moment loading
<table>
<thead>
<tr>
<th>API WSD</th>
<th>API LRFD</th>
<th>ISO 19902</th>
<th>NORSOK N-004</th>
</tr>
</thead>
<tbody>
<tr>
<td>3.4.1.c. Unstiffened Cone-cylinder Junctions</td>
<td>0.4.1.3 Unstiffened Cone-cylinder Junctions</td>
<td>13.6 Conical transitions</td>
<td>6.5 Strength of conical transition</td>
</tr>
<tr>
<td>1. Longitudinal Stress</td>
<td>1. Longitudinal Stress</td>
<td>$\alpha \leq 30$ deg</td>
<td>6.5.2.1 Equivalent design axial stress in the cone design</td>
</tr>
<tr>
<td>$L_1 = \frac{D_1}{D_0} \left( 1 - \Delta L_{10}^2 \right) + (\sigma_1 + \sigma_2 + \sigma_3) \tan \alpha L_1^2$</td>
<td>$L_1 = \frac{D_1}{D_0} \left( 1 - \Delta L_{10}^2 \right) + (\sigma_1 + \sigma_2 + \sigma_3) \tan \alpha L_1^2$</td>
<td>$\alpha = \frac{\sigma_1}{\sigma_2} \leq 1$</td>
<td>$G_{\text{eq,con}} = G_{\text{eq,con}} + \Delta G_{\text{eq,con}}$</td>
</tr>
<tr>
<td>2. Hoop Stress</td>
<td>2. Hoop Stress</td>
<td>$\alpha = \frac{\sigma_1}{\sigma_2} \leq 1$</td>
<td>$G_{\text{eq,con}} = G_{\text{eq,con}} + \Delta G_{\text{eq,con}}$</td>
</tr>
<tr>
<td>$L_2 = 0.45 \left( D_1^2 \tan \alpha + L_2^2 \right)$</td>
<td>$L_2 = 0.45 \left( D_1^2 \tan \alpha + L_2^2 \right)$</td>
<td>$L_2 = M_{\text{eq,con}} \Delta L_{10} + \Delta M_{\text{eq,con}} \tan \alpha L_2^2$</td>
<td>$G_{\text{eq,con}} = G_{\text{eq,con}} + \Delta G_{\text{eq,con}}$</td>
</tr>
<tr>
<td>$L_2 = 0.8 F_{\text{th}}$</td>
<td>$L_2 = 0.8 F_{\text{th}}$</td>
<td>$L_2 = 4M_{\text{eq,con}} \Delta L_{10} + \Delta M_{\text{eq,con}} \tan \alpha L_2^2$</td>
<td>$G_{\text{eq,con}} = G_{\text{eq,con}} + \Delta G_{\text{eq,con}}$</td>
</tr>
<tr>
<td>$L_2 = 0.5 F_{\text{th}}$</td>
<td>$L_2 = 0.5 F_{\text{th}}$</td>
<td>$L_2 = 4M_{\text{eq,con}} \Delta L_{10} + \Delta M_{\text{eq,con}} \tan \alpha L_2^2$</td>
<td>$G_{\text{eq,con}} = G_{\text{eq,con}} + \Delta G_{\text{eq,con}}$</td>
</tr>
<tr>
<td>$F_{\text{th}} = F_{\text{th}}$</td>
<td>$F_{\text{th}} = F_{\text{th}}$</td>
<td>$F_{\text{th}} = 0.5F_{\text{th}}$</td>
<td>$G_{\text{eq,con}} = G_{\text{eq,con}} + \Delta G_{\text{eq,con}}$</td>
</tr>
<tr>
<td>$F_{\text{th}} = 0.45 \bar{F}<em>{\text{th}} = 0.45 \bar{F}</em>{\text{th}}$</td>
<td>$F_{\text{th}} = 0.45 \bar{F}<em>{\text{th}} = 0.45 \bar{F}</em>{\text{th}}$</td>
<td>$F_{\text{th}} = 0.45 \bar{F}<em>{\text{th}} = 0.45 \bar{F}</em>{\text{th}}$</td>
<td>$G_{\text{eq,con}} = G_{\text{eq,con}} + \Delta G_{\text{eq,con}}$</td>
</tr>
<tr>
<td>$F_{\text{th}} = 1.21 F_{\text{th}} { 1.15 + \frac{F_{\text{th}}}{F_{\text{th}}} }$</td>
<td>$F_{\text{th}} = 1.21 F_{\text{th}} { 1.15 + \frac{F_{\text{th}}}{F_{\text{th}}} }$</td>
<td>$F_{\text{th}} = 1.21 F_{\text{th}} { 1.15 + \frac{F_{\text{th}}}{F_{\text{th}}} }$</td>
<td>$G_{\text{eq,con}} = G_{\text{eq,con}} + \Delta G_{\text{eq,con}}$</td>
</tr>
<tr>
<td>$F_{\text{th}} = 6.2F_{\text{th}}$</td>
<td>$F_{\text{th}} = 6.2F_{\text{th}}$</td>
<td>$F_{\text{th}} = 6.2F_{\text{th}}$</td>
<td>$G_{\text{eq,con}} = G_{\text{eq,con}} + \Delta G_{\text{eq,con}}$</td>
</tr>
<tr>
<td>$F_{\text{th}} = 0.4 \bar{F}_{\text{th}}$</td>
<td>$F_{\text{th}} = 0.4 \bar{F}_{\text{th}}$</td>
<td>$F_{\text{th}} = 0.4 \bar{F}_{\text{th}}$</td>
<td>$G_{\text{eq,con}} = G_{\text{eq,con}} + \Delta G_{\text{eq,con}}$</td>
</tr>
</tbody>
</table>

13.6.2.2 Bending stress

$\alpha = \frac{\sigma_1}{\sigma_2} \leq 1$ |

13.6.2.3 Hoop stresses

$\alpha = \frac{\sigma_1}{\sigma_2} \leq 1$ |

13.6.3.2 Local buckling

$\sigma_{\text{eq,con}} = \left( \frac{\sigma_{\text{eq,con}}}{\sigma_{\text{eq,con}}} \right)$ |

13.6.3.3 Junction yielding

If $\sigma_{\text{eq,con}}$ is tensile: |

If $\sigma_{\text{eq,con}}$ is compressive: |

$\left[ \frac{\sigma_{\text{eq,con}}}{\sigma_{\text{eq,con}}} \right] - \frac{\sigma_{\text{eq,con}}}{\sigma_{\text{eq,con}}} \leq L_{\text{crit}}$ |

$\left[ \frac{\sigma_{\text{eq,con}}}{\sigma_{\text{eq,con}}} \right] - \frac{\sigma_{\text{eq,con}}}{\sigma_{\text{eq,con}}} \leq L_{\text{crit}}$ |

13.6.4.4 Junction buckling

$\Delta L_{10} = \Delta L_{10}$ |

13.6.3 Strength requirements without external hydrostatic pressure

$\sigma_{\text{eq,con}} = \sigma_{\text{eq,con}} + \Delta \sigma_{\text{eq,con}}$ |

13.6.3.1 Local buckling under axial compression

$\sigma_{\text{eq,con}} = \left( \frac{\sigma_{\text{eq,con}}}{\sigma_{\text{eq,con}}} \right)$ |

$\Delta \sigma_{\text{eq,con}} = \left( \frac{\Delta \sigma_{\text{eq,con}}}{\Delta \sigma_{\text{eq,con}}} \right)$ |

13.6.4.1 Hoop buckling

$\Delta L_{10} = \Delta L_{10}$ |

13.6.3.3 Junction buckling

If $\sigma_{\text{eq,con}}$ is tensile: |

If $\sigma_{\text{eq,con}}$ is compressive: |

$\Delta \sigma_{\text{eq,con}} = \left( \frac{\Delta \sigma_{\text{eq,con}}}{\Delta \sigma_{\text{eq,con}}} \right)$ |

$\Delta \sigma_{\text{eq,con}} = \left( \frac{\Delta \sigma_{\text{eq,con}}}{\Delta \sigma_{\text{eq,con}}} \right)$ |

13.6.4 Strength requirements with external hydrostatic pressure

$\sigma_{\text{eq,con}} = \sigma_{\text{eq,con}} + \Delta \sigma_{\text{eq,con}}$ |

13.6.4.1 Hoop buckling

$\Delta L_{10} = \Delta L_{10}$ |

13.6.4.3 Junction buckling

If $\sigma_{\text{eq,con}}$ is tensile: |

If $\sigma_{\text{eq,con}}$ is compressive: |

$\Delta \sigma_{\text{eq,con}} = \left( \frac{\Delta \sigma_{\text{eq,con}}}{\Delta \sigma_{\text{eq,con}}} \right)$ |

$\Delta \sigma_{\text{eq,con}} = \left( \frac{\Delta \sigma_{\text{eq,con}}}{\Delta \sigma_{\text{eq,con}}} \right)$ |

13.6.4.4 Junction buckling

$\Delta L_{10} = \Delta L_{10}$ |

13.6.4 Strength requirements with external hydrostatic pressure

$\sigma_{\text{eq,con}} = \sigma_{\text{eq,con}} + \Delta \sigma_{\text{eq,con}}$ |

13.6.4.1 Hoop buckling

$\Delta L_{10} = \Delta L_{10}$ |
### 8.5.4.2. Junction yielding and buckling

\[ \sigma_{YJ} = \sigma_t \]

If \( \sigma_t \) is tensile:

\[ \sigma_{YJ} = \left( \sigma_t^2 - \sigma_t \sigma_{Rd,J} + \sigma_{Rd,J}^2 \right) ^{1/2} \times f_y / f_t, \]

for tensile \( \sigma_{Rd,J} \)

\[ \sigma_{YJ} = \left( \sigma_t^2 + \sigma_t \sigma_{Rd,J} + \sigma_{Rd,J}^2 \right) ^{1/2} \times f_y / f_t, \]

for compressive \( \sigma_{Rd,J} \)

If \( \sigma_t \) is compressive:

\[ a^2 + b^2 = 1.0 \]

\[ A = \frac{a \sigma_{Rd,J}}{f_t}, \]

\[ B = \frac{b \sigma_{Rd,J}}{f_t}, \]

\[ \sigma_{YJ} = \frac{a f_y}{f_t}, \]

for compressive \( \sigma_{Rd,J} \)

\[ \sigma_{YJ} = \frac{b f_y}{f_t}, \]

for compressive \( \sigma_{Rd,J} \)

\[ D_{YJ} = \frac{D_{t}}{f_t}, \quad \text{and} \quad D_{Rd,J} = \frac{D_{t}}{f_t}, \]

for compressive \( \sigma_{Rd,J} \)

\[ D_{YJ} = \frac{D_{t}}{f_t}, \quad \text{and} \quad D_{Rd,J} = \frac{D_{t}}{f_t}, \]

for compressive \( \sigma_{Rd,J} \)
4 FATIGUE

4.1 General

This section compares the fatigue requirements given in the three codes. The comparison addresses both simplified and detailed fatigue methodologies and associated fatigue criteria. The ISO does not give requirements for simplified fatigue because it mandates detailed fatigue for all structures. Only API WSD and NORSOK are compared in this case, Table 4-2 at end of this section.

The detailed fatigue requirements in API, ISO, and NORSOK are compared in Table 4-3.

It should be stated that DNV has published several state-of-the-art documents on fatigue that are instructive and provide supporting and more detailed methodology for fatigue assessment of offshore structures, see References 24 to 26.

4.2 Code Validity

API RP 2A mainly focuses on the fixed structure. Its fatigue assessment is based on the assumption that the connection has full-penetration single or double sided welding.

Basic WJ and CJ curves in API are based on steels with yield strength less than 72 ksi (500 MPa).

ISO 19902 and 19904 are applicable to the fatigue design of new structures as well as the fatigue assessment of existing structures. The fatigue assessment in ISO 19902 is based on the same assumption as API RP 2A and mainly gives guidance for fixed structures. ISO 19904 provides general guidance for plated structures and detailed analysis methods and procedures refer to Recognized Classification Society (RCS) rules, such as ABS, DNV etc..

In ISO, representative S-N curves for tubular joints (TJ), cast joints (CJ) and other joints (OJ) are based on steels with a yield strength less than 500 MPa.

NORSOK refers to DNV fatigue codes directly. Experience gained by DNV over the more than 60 years of offshore operation assessing the performance of both new and existing structures with respect to fatigue susceptibility has been incorporated in its most recent recommended practice RP-C203 (April 2010). Another RP-C206 (April 2007) gives guidance on “Fatigue Methodology of Offshore Ships” applicable to ship-shaped offshore units.

DNV-RP-C203 is valid for steel materials in air with yield strength less than 960 MPa. For steel materials in seawater with cathodic protection or steel with free corrosion the RP is valid up to 550 MPa. It may be used for stainless steel.

DNV-RP-C203 is valid for material temperatures of up to 100°C. For higher temperatures the fatigue resistance data may be modified with a reduction factor.

Finite element analysis and hot spot stress methodology is important for plated structures and this is included in DNV-RP-C203 but not in ISO 19902/19904 and hardly in API-RP2A.
4.3 Fatigue Parameter

4.3.1 Loading

API RP 2A recommends that steepness between 1:20 to 1:25 is generally used for the Gulf of Mexico and a minimum height equal one foot and a maximum height equal to the design wave height should be used.

ISO recommends that steepness between 1:20 to 1:25 is generally used and a wave height equal to the wave height with a one year return period should be normally be used as a maximum.

NORSOK states that the wave periods shall be determined based on a wave steepness of 1:20 in lack of site specific data.

Both ISO and NORSOK require that the partial action factors shall be taken as 1.0 and resistance factor shall also be taken as 1.0.

Hot spot stress formula for tubular joints in API and ISO are identical. For other than tubular joints, API RP 2A refers to ANSI/AWS D.1.1 for details.

4.3.2 Stress Concentration Factor

The Efthymiou’s equations are used in all three codes. SCF formulas for T/Y joints in all three codes are identical for T- and Y-joints at crown positions for long chord members where DNV-RP-C203 is improved. This is considered to reduce engineering work and improve the reliability of fatigue analysis.

All three codes give the same SCF formulas for X joints under the conditions of balanced axial load, in-plane bending and balanced out-of-plane bending. DNV-RP-C203 gives additional two sets of formulas for axial load in one brace only and out-of-plane bending on one brace only.

For K-joints and KT-joints, all three codes provide the same formulas for the conditions of balanced axial load, unbalanced in-plane bending and unbalanced out-of-plane bending. DNV-RP-C203 also gives additional three sets of formulas for axial load in one brace only, in-plane bending on one brace only and out-of-plane bending on one brace only.

4.3.3 S-N Curve

Fatigue analysis may be based on different methodologies depending on what is found most efficient for the considered structural detail. It is important that stresses are calculated in agreement with the definition of the stresses to be used together with a particular S-N curve. DNV-RP-C203 gives the three different concepts of S-N curves:

1. Nominal stress S-N curve: Normal stress is a stress in a component that can be derived by classical theory such as beam theory. In a simple plate specimen with an attachment, the nominal stress is simply the membrane stress that is used for plotting of the S-N data from the fatigue testing.
2. Hot spot stress S-N curve for plated structures and tubular joints: Hot spot stress is the geometric stress created by the considered detail.

3. Notch stress S-N curve: It can be used together with finite element analysis where local notch is modeled by an equivalent radius. This approach can be used only in special cases where it is found difficult to reliably assess the fatigue life using other methods.

S-N curves in all three codes are valid for high cycle fatigue. API RP 2A only gives two S-N curves for two joint classes (WJ for tubular joints and CJ for cast joints) and there is nothing for plated structures.

Except S-N curve for tubular joints and cast joints which are identical to API, ISO provides additional eight S-N curves for the other connection details.

In DNV-RP-C203, all tubular joints are assumed to be class T. Other types of joint, including tube to plate, fall in one of 14 classes depending on:

- The geometrical arrangement of the detail
- The directional of the fluctuating stress relative to the detail
- The method of fabrication and inspection of the detail

DNV-RP-C203 also gives some guidance on assessment of a design S-N curve based on a limited number of test data. Finite element analysis and hot spot stress methodology is important for plated structures. Only DNV-RP-C203 provides the guidance for the calculation of hot spot stress by finite element analysis.

When the thickness effects are considered, the reference material thickness is the same (16 mm) in API and ISO. In API-RP-2A, the reference thickness is 25 mm for welded connections other than tubular joints; 25 mm for tubular joints and bolts.

4.3.4 Design Fatigue Factors (DFF’s)

As shown in Table 4-1 and Table 4-3, NORSOK recommends DFF’s varying from 1, 2, 3, and 10 whereas API DFF are 2, 5, and 10 only. NORSOK has DFF ranges for below and above splash zone while API does not make this distinction. NORSOK considers all structural joints deeper 150m to be inaccessible for inspection.
4.3.5 Fatigue Damage Accumulation

All three design codes suggest that the fatigue life may be calculated based on S-N fatigue approach under the assumption of linear cumulative damage (Palmgren-Miner rule). Even though the cumulative fatigue damage passing criteria looks different, but the basic principle is all the same. Only difference is that where the design safety factor (DFF) is introduced.

4.4 Fatigue Analysis Methods

4.4.1 Simplified Fatigue

API allows simplified fatigue calculations only for Category L-3 template type platforms that are constructed of notch-tough ductile steels, have redundant inspectable structure, and have natural period of less than 3s or for preliminary design of all structure categories in water depth up to 400 ft (122m). As shown in Table 4-2 API RP 2A WSD defines in Section 5.1 and its commentary the fatigue design wave and allowable peak hot spot stresses. Simple tubular joints SCF formulas are also presented in addition to recommended DFF (Design Fatigue Factor) depending on criticality of the fatigue failure and accessibility for inspection see Table 4-2.

NORSOK refers to DNV-RP-C203, Section 5 for the details of the methodology and the allowable stress range as function of the Weibull shape parameter and applicable fatigue curve (depending on the joint detail and stress field configuration; i.e., the fatigue curve) for 20 years’ service life ($10^8$ cycles). The simplified fatigue given in DNV-RP-C203 is applicable to mass dominated structures such as Semisubmersible, ships, FPSOs and TLPs in conceptual design phase. It is less appropriate for drag dominated structures such as jackets and truss towers with slender tubular members.
4.4.2 Detailed Fatigue

The comparison made in Table 4-3 covers the assumptions, loading definitions, hot spot stress range calculation, stress concentration factor formulas, S-N curves for tubular joints and plated structures, and DFF required values. Detailed comparison has been given in Section 4.3.

4.4.3 Fracture Mechanics

Fracture mechanics may be used for fatigue analyses as supplement to S-N data. Fracture mechanics can be used to assess the acceptable defects, evaluate the acceptance criteria for fabrication and for planning in-service inspection.


4.5 Welding Improvement Techniques

In all three codes, the welding improvement techniques are all the same and the achievable improvement factors on fatigue performance are identical.

4.6 Summary of Fatigue Comparison

API allows simplified fatigue calculations only for Category L-3 template type platforms that are constructed of notch-tough ductile steels, have redundant inspectable structure, and have natural period of less than 3s or for preliminary design of all structure categories in water depth up to 400 ft (122m).

NORSOK refers to DNV-RP-C203, Section 5 for the details of the simplified fatigue methodology and the allowable stress range as function of the Weibull shape parameter and applicable fatigue curve (depending on the joint detail and stress field configuration; i.e., the fatigue curve) for 20 years’ service life (10^8 cycles).

Detail fatigue assumptions, loading definitions, hot spot stress range calculation, stress concentration factor formulas, S-N curves for tubular joints, and required DFF values are specified in all three codes. In addition, details of the spectral analysis, utilization of fracture mechanics, and fatigue life improvement techniques are also compared. The requirements are quite similar.

The Efthymiou’s equations are used in all three codes. SCF formulas for T/Y joints in all three codes are identical for T- and Y-joints at crown positions for long chord members where DNV-RP-C203 is improved. This is considered to reduce engineering work and improve the reliability of fatigue analysis.

Finite element analysis and hot spot stress methodology is important for plated structures and this is included in DNV-RP-C203 but not in ISO 19902/19904 and hardly in API-RP2A.
### Table 4-2 Simplified Fatigue

<table>
<thead>
<tr>
<th>API RP 2A, WSD</th>
<th>NORSOK N.001, H003/DNV RP C203</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Limitations</strong>:</td>
<td></td>
</tr>
<tr>
<td>1. Have been calibrated for the design wave climate</td>
<td></td>
</tr>
<tr>
<td>2. May be applied to tubular joints in Category 3 template type platforms as defined in Section 1.7</td>
<td></td>
</tr>
<tr>
<td>3. Are constructed of non-rough guidance surfaces</td>
<td></td>
</tr>
<tr>
<td>4. Have redundant, inspectable structural framing</td>
<td></td>
</tr>
<tr>
<td>5. Have natural periods less than 3 seconds</td>
<td></td>
</tr>
<tr>
<td>6. Particularly useful for preliminary design of all structure categories and types, in water depths up to 400 feet (122 m)</td>
<td></td>
</tr>
</tbody>
</table>

**Fatigue design wave**: The fatigue design wave is the reference level wave for the platform water depth as defined in Figure 2.3.4.3.

**Wave Forces**: Follow the procedures in Section 2.3.1 except that the omni-directional wave should be applied in all directions with wave kinematics factor equal to 0.09.

**Environmental Data**: Wave force should be applied to the structure without wind, current and gravity load effects.

In general, four wave approach directions (end-on, broadside and two diagonal) and sufficient wave positions relative to the platform should be considered to identify the peak load stress at each member end for the fatigue design wave.

The allowable peak hot spot stress, $S_p$, is determined from Figure C6-1 or C6-2.

Design charts for steel components in air and in seawater with cathodic protection are given.
The peak hot spot stress at a joint should be taken as the maximum value of the following expression calculated at both the chord and brace sides of the tubular joint:

\[ \sigma_{\text{chord}} = (\sigma_{\text{axial}} + \sigma_{\text{tension}}) + \sigma_{\text{bending}} \]

where \( \sigma_{\text{axial}} \), \( \sigma_{\text{tension}} \), and \( \sigma_{\text{bending}} \) are the axial, tension, and bending stresses, respectively.

\( \sigma_{\text{axial}}, \sigma_{\text{tension}}, \) and \( \sigma_{\text{bending}} \) are the corresponding stress concentrations factor for axial, tension, and bending stresses, respectively.

Table C5.1-1 includes SCFs developed from the referenced examples. To be used with equation (C5.1-1) for simple joints.

<table>
<thead>
<tr>
<th>Table 5.2.1.1 - Fatigue Life Safety Factors</th>
</tr>
</thead>
<tbody>
<tr>
<td>Failure-critical</td>
</tr>
<tr>
<td>-----------------</td>
</tr>
<tr>
<td>No</td>
</tr>
<tr>
<td>Yes</td>
</tr>
</tbody>
</table>

OFF Table above is for assessment of Category C structures. A reduced safety factor is recommended for Category C2 and L0 conventional steel jacket structures on the basis of in-service performance data: SF = 1.0 for redundant diver or ROV inspectable framing, with safety factors for other classes being half those in the table.

Design fatigue factors in DNV-RP-C209 refers to DNV-OS-C101 Section 6, Table A1, which is valid for units with low consequence of failure and where it can be demonstrated that the structure satisfies the requirements to damaged condition according to the AS1 with failure in the actual element as the defined damage.

Table A1 Design fatigue factors (DF)

<table>
<thead>
<tr>
<th>Structural elements</th>
</tr>
</thead>
<tbody>
<tr>
<td>1. Internal structure, accessible and not welded directly to the submerged part.</td>
</tr>
<tr>
<td>2. Internal structure, accessible for regular inspection and repair in dry and clean conditions.</td>
</tr>
<tr>
<td>3. External structure not accessible for inspection and repair in dry and clean conditions.</td>
</tr>
<tr>
<td>4. Non-accessible areas, areas not planned to be accessible for inspections and repair during operations.</td>
</tr>
</tbody>
</table>

NORSOK N-004, Tables 8.1 and K-4.1 also gives the DFs and are included in Detailed Fatigue comparison table.

Table C5.1-1 - Selected SCF Formulas for Simple Joints

<table>
<thead>
<tr>
<th>Load Type</th>
<th>Stress</th>
<th>SCF Chord</th>
<th>SCF Axial</th>
</tr>
</thead>
<tbody>
<tr>
<td>X</td>
<td>4.0</td>
<td>3.0</td>
<td>1.0</td>
</tr>
<tr>
<td>Y</td>
<td>5.0</td>
<td>2.0</td>
<td>1.0</td>
</tr>
<tr>
<td>Z</td>
<td>6.0</td>
<td>1.0</td>
<td>1.0</td>
</tr>
</tbody>
</table>

Since SCF = 1.0, Table 5.2.1.1 is applicable. The SCFs are given in DNV-RP-C209 Appendix B. SCFs for Tubular Joints, SCFs for Penetrations with Reinforcements are given in Appendix C.

\[ SCF_{\text{axial}} = 1.0 + 0.378 \left( \frac{D}{d} \right)^{0.5} \left( \frac{t}{D} \right)^{0.5} \]
### Table 4-3 Detailed Fatigue

<table>
<thead>
<tr>
<th>ISO/19922</th>
<th>NORSOK M-004/DNV RP-C203</th>
</tr>
</thead>
<tbody>
<tr>
<td>API 2A 2X-WIM</td>
<td></td>
</tr>
</tbody>
</table>

#### Nutect
- The requirements in this ISO are applicable to the fatigue design of new structures as well as the fatigue assessment of existing structures. However, they only relate to fatigue evaluations of uncracked locations; therefore, in the case of existing structures, the provision is that there be no crack already present.
- The fatigue assessment of welded joints is based on the assumption that the connection has full-penetration single or double sided welding.

| Wave force calculations should follow the procedures described in Section 2.3.1 with the following exceptions: |
| Current - may be neglected and considerations for apparent wave period and current magnitude are not required. |
| For the Gulf of Mexico a steepness between 1:20 and 1:25 is generally used. A minimum height one foot and a maximum height equal to the design wave height should be |
| Wave kinematics factor = 1.0 |
| Conductor shearing factor = 1.0 |
| For small waves (L = K 0.6 for platform legs at mean water level), values of Cw = 2.0, Ck = 1.5 for rough members and Cw = 0.5 for smooth members |
| Use 60 to 150 sea states each with its wave energy spectrum |

- In determining stress variations for a fatigue analysis the partial action factors shall be taken as 1.0.
- The partial resistance factor on the fatigue assessment shall also be taken as 1.0.

#### Geometric stress (Gk) and geometric stress range (Gk)

#### Tubular Joints

- A minimum of eight stress range locations need to be considered around each chord/brace intersection in order to adequately cover all relevant locations. These are: chord crowns (2), chord saddles (2), brace crowns (2) and brace saddles (2).
- HSS for saddle and crown are given by:
- $HSS = SCF_{max} - SCF_{min} + CE$
- Where:
  - $SCF_{max}$ = maximum stress
  - $SCF_{min}$ = minimum stress
  - $CE$ = effect of nominal cyclic stress in the chord
- Other HSS are also required.

#### Stress Range

- Where variations of stress are applied to conventional weld details, identified in the ANSI/AWS D1.1 - 2002 Table 2.4.
- The associated $S-N$ curves provided in Figure 2.11 should be used, dependent on degree of redundancy.

#### Welded connections other than tubular joints

- In plate structures, three types of hot spots at weld toes can be identified:
- a) at the weld toe on the plates surface at endring attachment
- b) at the weld toe around the plate edge of an endring attachment
- c) along the weld of an attached plate (weld toes on both the plate and attachment surface)
SCF - HSSR at location (excluding notch effect) - Nominal brace stress range

1. The SCF should include all stress raising effects associated with the joint geometry and specified loading, except the local (microscopic) weld notch effect, which is included in the S/N curve.

2. The geometric stress or strain is defined as the total range that would be measured by a strain gauge adhered to the face of the weld and oriented perpendicular to the weld, so as to reflect the stress which will be amplified by the weld toe discontinuities. Typical geometric stress gauges are located within 0.5 mm to 0.1 mmB from the weld toe with a gauge length of 5 mm. I refer to the outside radius and thickness of the member instrumented, whether chord or brace.

3. The SCF equations in Table 6.3.2.1 to 6.3.2.4 are recommended because this set of equations is considered to offer the best option for all joint types and load types and is only the weld notched welds that covers overlapped K and K7 joints.

4. The validity ranges for the SCF parametric equations are as follows:

   - S < 0.2 to 1.0
   - Y < 0.2 to 1.0
   - r < 0.2 to 1.0
   - a < 4.0
   - e < 20 to 90 degrees
   - z < 0.0 to 1.0

5. For all welded tubular joints under all three types of loading, a minimum SCF of 1.5 should be used.

6. SCFs for internally ring-diffused joints can be determined by applying the Lloyds reduction factors to the SCFs for the equivalent unstiffened joint. For ring-diffused joints such methods, the minimum SCF for the brace side under axial or OPB loading should be taken as 2.0. A minimum value of 1.5 is recommended for all other locations.

The biaxial joint S/N curve has been derived from an analysis of data on tubular joints manufactured using welds conforming to a standard flat profile given in A8/2.

The basic S/N curve is of the form:

\[ \log N = (k_1 \log S_k + k_2) / M \]

where:
- \( N \) = the predicted number of cycles to failure under constant amplitude stress range \( S \)
- \( k_1 \) = a constant
- \( k_2 \) = the inverse slope of the S/N curve
- \( M \) = the constant amplitude stress range, which is the geometrical stress range

### Table 5.5.1.1 - Basic Design S/N Curves

<table>
<thead>
<tr>
<th>Curve</th>
<th>log ( a / (k_1) )</th>
<th>log ( a / (k_2) )</th>
<th>( S ) in ksi</th>
<th>( M ) in ksi</th>
</tr>
</thead>
<tbody>
<tr>
<td>Welded Joints (WJ)</td>
<td>14.02</td>
<td>12.48</td>
<td>3 to 4 x 10^3</td>
<td>5.0 x 10^3</td>
</tr>
<tr>
<td>Cast Joints (CJ)</td>
<td>11.00</td>
<td>16.13</td>
<td>4 to 5 x 10^3</td>
<td>5.0 x 10^3</td>
</tr>
</tbody>
</table>

The basic design S/N curves given in Table 5.5.1.1 are applicable for joints in air and unstiffened coated joints.

The S/N curves are given in ksi. The reference thickness is the thickness of the S/N curve.

The S/N curve is based on a reference thickness of 0.38 in (9.7 mm).

- The reference thickness is the thickness of the member instrumented.
- The reference thickness for the fatigue life is provided.
- The reference thickness for the weld is provided.
- The reference thickness is adjusted accordingly.
The cumulative fatigue damage ratio, \( D \), is given by
\[
D = \sum n_i (\Delta N)
\]
(5.2.4-1)

A linear accumulation of fatigue damage under constant amplitude stress ranges, according to the Palmgren-Miner rule.

Under the assumption of linear cumulative damage (Palmgren-Miner rule):

The cumulative fatigue damage ratio, \( D \), is calculated as a sum of the number of cycles, \( n_i \), at each stress level, \( \Delta N \), according to the Palmgren-Miner rule.

The factors given in Table A. 16-12-1 should be considered for exposure Level L1, but also should be used for exposure levels L2 and L3. There is currently insufficient background to establish different factors for lower exposure levels.

The factors given in Table A. 16-12-1 are recommended for Category L-2 and L-3 conventional steel jacket structures on the basis of in-service performance data if \( S = 1.0 \) for redundant diver or ROV inspection, safety factors for other cases being half those in the table.
1. Transfer functions developed using regular waves in the time domain:
   - Characterize the wave climate using either the two, three, four and eight parameter format.
   - Select a sufficient number of frequencies to define all the peaks and valleys inherent in the jackup response function.
   - Select a wave height corresponding to each frequency.
   - For a 1-foot wave height, use at least two, three, four and eight parameter functions.
   - A minimum of one foot and a maximum height equal to the design wave height should be used.
   - Compute a stress time transfer function at each point where fatigue damage is to be accumulated.
   - A practical method that is best able to represent the random nature of the wave environment.
   - For the time domain, the transfer function is linearized.
   - A maximum of 12 fatigue cycles generated for the analysis.

2. Transfer functions developed using regular waves in the frequency domain:
   - This approach is similar to method (1) except that the analysis is linearized prior to the calculation of structural response.
   - Compute the stress response spectra.

3. Transfer functions developed using random waves in the time domain:
   - Nonlinearity arising from wave-structure interaction can be taken into account and difficulties in selecting wave and frequency terms for transfer function generation can be avoided.
   - Characterize the wave climate in terms of sea state scatter diagram.
   - Simulate random wave time histories of finite length for a few selected reference sea states.
   - Computer response stress time histories at each point of a structure where fatigue life is to be determined and transform the response stress time histories into response spectra.
   - Generates exact transfer functions from wave and response spectra.
   - Calculate pseudo transfer functions for all the remaining sea states in the scatter diagram using the "exact" transfer functions.
   - Calculate pseudo response spectra as described in Section C5.2.2.1.

The details refer to ISO 19900 Clause A.16.15

Specifications, tolerances, and test parameters for fatigue analysis:

- Typical applications:
  1) To assess the fitness-for-purpose of a component with or without defects
  2) To assess the inspection requirements for a component with or without known defects
  3) To assess the inspection requirements for components which may not be subjected to PWHT
  4) To assess the structural integrity of castings
  5) The principal modes of failure in offshore structures
  6) Fracture toughness of castings followed by the onset of fracture due to coalescence of the fracture toughness at a critical crack size (not necessarily through-thickness)
  7) The occurrence of plastic collapse
  8) Fatigue crack growth law

- This method is recommended for use in assessment of acceptability defects, evaluation of acceptance criteria for fabrication and for planning in-service inspections.
- This can be achieved by performing the analysis according to the following procedures:
  1) Crack growth parameter C determined as mean plus or minus standard deviation
  2) A careful evaluation of the defects that might be present in the structure when taking into account the actual NDE inspection method used to detect cracks during fabrication
  3) Usage of geometry factors that are on the safe side
  4) Usage of utilization factors similar to those used when the fatigue analysis is based on S-N data
  5) The Paris equation may be used to predict the crack propagation or the fatigue life.
- Welding profiling - Weld Toe Grinding

Experimental data indicate that this technique can lead to an increase in the fatigue performance by a factor of 2.

The grinding procedure should ensure that all defects in the weld toe region have been removed by grading to a depth not less than 0.5 mm below the bottom of any visible undercut or defect. The maximum depth of local grinding should not exceed 2 mm or 5% of the plate thickness, whichever is less. NOE of the joint is required after grinding to verify that no significant defects remain, for flat-welded connections. It is important that the required throat size is maintained.

- Full Profile Grinding, e.g., Butt Welds

For welds with tubular nodes, full grinding of the surface profile to a radius of not less than 0.5 times the nominal wall thickness of the weld metal should be required. The grinding should be done to a depth of at least 0.5 mm, but not exceed 0.5 mm. The recommended fatigue performance improvement factor is 8.

- Post-weld heat treatment (PWHT) - have a beneficial effect on the fatigue behavior of welded joints, the knowledge of the residual stress distribution including the contribution of long-range fit-up stresses is required.

- Welding profiling - no clear evidence that welding profiling leads to improved fatigue performance.

- Weld toe grinding of tubular joint welds - especially beneficial at low stress ranges. Experimental data indicate that this technique can lead to an increase in the fatigue performance by a factor of 2.

- Grinding of butt welds - to improve the joint classifications.

- Hammer peening - The objective is to obtain a smooth groove at the weld toe. The groove depth should be at least 0.3 mm, but not exceed 0.5 mm. The recommended fatigue performance improvement factor is 4.

The benefits of hammer peening on fatigue performance can only be realized through adoption of adequate quality control procedures.

Table 5.3.1—Factors on Fatigue Life for Weld Improvement Techniques

<table>
<thead>
<tr>
<th>Weld Improvement Technique</th>
<th>Improvement Factor</th>
<th>Improvement Factor in %</th>
</tr>
</thead>
<tbody>
<tr>
<td>Weld toe edge grinding</td>
<td>2</td>
<td>200%</td>
</tr>
<tr>
<td>Hammer peening</td>
<td>4</td>
<td>400%</td>
</tr>
</tbody>
</table>

Table 16.14—Achievable improvement factors on fatigue performance for weld improvement techniques

- Welding profiling by machining and grinding

The maximum improvement factor from the grinding only should be limited to a factor 2 on fatigue life.

Weld toe grinding

Where local grinding of the weld toe below any visible undercut is performed the fatigue life may be increased by a factor given in Table 1.1.

- TIG dressing

- Hammer peening with the following limitations:
  1. only used on members where failure will be without substantial consequences
  2. overloads in compression must be avoided
  3. it is recommended to grind a steaming groove by means of a rotary burr of a diameter suitable for the hammer head to be used for the peening. The peening tip must be small enough to reach the weld toe.
5 FOUNDATION DESIGN

A comparison is made between requirements given to pile foundation design by the following codes:

- ISO 19902:2007(E), Petroleum and natural gas industries — Fixed steel offshore structures
- NORSOK standard N-001, Structural design, Rev. 4, February 2004, in combination with NORSOK standard N-004, Design of steel structures, Rev. 2, October 2004, and in particular the Annex K therein, Special design provisions for jackets.

The comparison focused mainly on safety format related to axial pile capacity and to requirements and recommendations for calculation of axial pile capacity.

5.1 Comparison of Safety Format and Safety Level

The main difference between the codes is that the API-RP 2A WSD makes use of total safety factors whereas the ISO and the NORSOK standards use load and resistance factor design. Table 5-1 gives the comparison of the load combinations among three codes.

<table>
<thead>
<tr>
<th>Code – design condition</th>
<th>Load combinations</th>
<th>Pile material coefficient</th>
</tr>
</thead>
<tbody>
<tr>
<td>ISO – extreme condition</td>
<td>$1.1D_1 + 1.1D_2 + 1.1L_1 \tau E_a (E_a + \tau D_a)$ or $0.9D_1 + 0.9D_2 + 0.8L_1 \tau E_a (E_a + \tau D_a)$</td>
<td>1.25</td>
</tr>
<tr>
<td>ISO – operating condition</td>
<td>$1.3D_1 + 1.3D_2 + 1.5L_1 + 1.5L_2 + 0.9/3E_a (E_a + \tau D_a)$</td>
<td>1.5</td>
</tr>
<tr>
<td>API LRFD – extreme condition</td>
<td>$1.1D_1 + 1.1D_2 + 1.1L_1 + 1.35(E_a + 1.25D_a)$ or $0.9D_1 + 0.9D_2 + 0.8L_1 + 1.35(E_a + 1.25D_a)$</td>
<td>1.25</td>
</tr>
<tr>
<td>API LRFD – operating condition</td>
<td>$1.3D_1 + 1.3D_2 + 1.5L_1 + 1.5L_2 + 1.2(E_a + 1.25D_a)$</td>
<td>1.5</td>
</tr>
<tr>
<td>NORSOK – extreme ULS</td>
<td>$1.0D_1 + 0.1L_1 + 1.3E$ or $1.3D_1 + 1.3L_1 + 0.7E$</td>
<td>1.3 (applied to pile group)</td>
</tr>
<tr>
<td>NORSOK – 10 000 y ALS</td>
<td>$1.0D_1 + 1.0L_1 + 1.0E$</td>
<td>1.0</td>
</tr>
</tbody>
</table>

The limit state design condition can be formulated as follows for design in accordance with the three codes considered. Note that notations used below generally differ from those used in the codes since the three codes use different notations. It is rather chosen to use the same notations for all three codes.
5.1.1 API-RP 2A WSD:

\[ F_{c,ax} = P + V + E \leq Q_{c,ax} / SF \]  

(5.1)

P is permanent load, V is variable load and E is environmental load. \( Q_{c,ax} \) is characteristic axial pile capacity.

SF shall be taken equal to 1.5 for extreme condition and 2.0 for operating condition.

5.1.2 ISO 19902:2007:

\[ F_{d,ax} = \gamma_{f,P} \cdot P + \gamma_{f,V} \cdot V + \gamma_{f,E} \cdot (E_s + \gamma_{f,Ed} \cdot E_d) \leq Q_{d,ax} = Q_{c,ax} / \gamma_m \]  

(5.2)

Here \( E_s \) and \( E_d \) are static respectively dynamic part of environmental load. Note that ISO defines two types of permanent as well as of variable loads, but this relates to which part should be included in different phases. The load factors do not differ between the two types of P or V loads.

5.1.3 NORSOK:

\[ F_{d,ax} = \gamma_{f,P} \cdot P + \gamma_{f,V} \cdot V + \gamma_{f,E} \cdot E \leq Q_{d,ax} = Q_{c,ax} / \gamma_m \]  

(5.3)

Table 5-2 gives load factors defined by ISO and NORSOK for different loading conditions.

<table>
<thead>
<tr>
<th></th>
<th>( \gamma_m )</th>
<th>( \gamma_{f,P} )</th>
<th>( \gamma_{f,V} )</th>
<th>( \gamma_{f,E} ) or ( \gamma_{f,Es} )</th>
<th>( \gamma_{f,Es} )</th>
</tr>
</thead>
<tbody>
<tr>
<td>NORSOK comb.a</td>
<td>1,30</td>
<td>1,30</td>
<td>1,30</td>
<td>0,70</td>
<td>n.a.</td>
</tr>
<tr>
<td>NORSOK comb.b</td>
<td>1,30</td>
<td>1,00</td>
<td>1,00</td>
<td>1,30</td>
<td>n.a.</td>
</tr>
<tr>
<td>ISO, extreme-c</td>
<td>1,25</td>
<td>1,10</td>
<td>1,10</td>
<td>1,35</td>
<td>1,25</td>
</tr>
<tr>
<td>ISO, extreme-t</td>
<td>1,25</td>
<td>0,90</td>
<td>0,80</td>
<td>1,35</td>
<td>1,25</td>
</tr>
<tr>
<td>ISO, operation</td>
<td>1,50</td>
<td>1,30</td>
<td>1,50</td>
<td>1,22</td>
<td>1,25</td>
</tr>
</tbody>
</table>

Generally according to ISO \( \gamma_{f,Es} \) and \( \gamma_{f,Ed} \) are to be defined by National Authorities, but in the Appendix to the standard \( \gamma_{f,Es} = 1.35 \) and \( \gamma_{f,Ed} = 1.25 \) are recommended for Gulf of Mexico and Extreme condition. For operating condition \( \gamma_{f,Es} = 0.9 \cdot 1.35 = 1.22 \) is recommended.

By comparing required characteristic axial capacity \( Q_{c,ax} \) from above limit state formulations one can calculate equivalent total safety factor \( SF_{eqv} \) corresponding to the partial safety factors defined by ISO or NORSOK for defined loading conditions. For simplicity all weights are defined as permanent load, i.e. neglecting the difference between load factors for permanent and variable loads defined by ISO. The equivalent safety factors can then be expressed as follow.
NORSOK: 

\[ SF_{eqv} = \gamma_m \cdot \frac{\gamma_f,E \cdot E + \gamma_f,P+V \cdot (P+V)}{E + (P+V)} \]  

ISO: 

\[ SF_{eqv} = \gamma_m \cdot \frac{\gamma_f,E \cdot E / DAF \cdot (1 + \gamma_f,D \cdot (DAF - 1)) + \gamma_f,P+Q \cdot (P+V)}{E + (P+V)} \]  

\[ DAF = \frac{E_s + E_d}{E_s} = \frac{E}{E_s} \]  

where

\( SF_{eqv} \) has been calculated as function of \( E/(P+V) \) for extreme as well as operating condition. For the ISO calculations for DAF = 1.0 and 1.3 are presented. The results of the calculations are shown on Figure 5-1 for extreme condition and Figure 5-2 for operating condition. As \( E \) approaches \(-(P+V)\), the calculated \( SF_{eqv} \) approaches \(-\infty\) or \(+\infty\). The various curves for the NORSOK and the ISO combinations always change sign at \( E/(P+V)= -1 \), since that corresponds to the characteristic load \( E+P+V=0 \). Negative value means that the factored design load has different sign than the characteristic load. The range \(-1 \leq E/(P+V) \leq 0\) is of no interest for piles since here the pile is in compression with a force lower than for static weight. Apart from for structures in very benign areas the extreme condition is governing for design of piles. Typically for governing piles in compression \( E/(P+V) \) is between 0.5 for platforms with heavy topside to 2 or maybe 3 for platforms with very light topside. From Figure 5-1 it is seen that there are generally small differences between the three standards for these conditions. Whereas platforms with heavy topside may not have piles in tension \( E/(P+V) \) always bigger than \(-1\) piles of light weight platforms may be governed by tensile capacity. This is particularly so when the capacity in compression has a large contribution from end bearing. It is seen from Figure 5-2 that in that case there is a significant difference in safety requirements between API on one hand and NORSOK and ISO on the other hand. This reflects the weakness of the allowable stress standards for design of elements where the load effect results from a difference between large load contributions.
Figure 5-1 Comparison of safety levels for axial pile capacity design – extreme condition

Figure 5-2 Comparison of safety levels for axial pile capacity design – operating condition
5.2 Axial Pile Capacity – Methods of Calculation

The following main comparison points are made:

- API and ISO prescribe the same "traditional API methods" for calculation of pile capacity in sand and in clay.
- Both API and ISO allow for alternative methods and in particular describe four alternative methods for calculation of pile capacity in sand in the commentary part of the standard.
- Both API and ISO require that the designer shall evaluate in each case whether higher resistance factors are required when using these methods. They do not state whether one or all methods shall be checked and how different capacities from different methods shall be handled.
- NORSOK does not prescribe specific methods but provides references to alternative methods and states: "The relevance of alternative methods should be evaluated related to actual design conditions. The chosen method should as far as possible have support in a data base which fits the actual design conditions related to soil conditions, type and dimensions of piles, method of installation, type of loading etc.."

5.3 Pile Structure Interaction and Definition of Pile Failure

The following main comparison points are made:

- All codes specifies that the nonlinear soil resistance shall be accounted for in the pile structure interaction model.
- NORSOK specifies that pile structure interaction shall be based on characteristic soil resistance. API and ISO is not specific on that although common practice. Doing pile structure interaction based on factorized soil resistance could be very un-conservative by allowing for redistribution of pile forces and thus removing the redundancy in the pile system.
- NORSOK specifies that the resistance factor for axial pile capacity shall apply to the total pile group axial force, and thus allows for lower resistance factor on individual piles. This is not specified by API or ISO – practice is varying.

Table 5-3 includes additional comparison of pile design requirements specified in the three codes (API, ISO, and NORSOK) from a structural perspective.
5.4 Summary

The following summarizes the results from the comparison made in this section:

- Code requirements and recommendations are very comparable between the three standards, and the choice of standard will not be decisive for the safety related to pile design.
- No calibration of safety factors towards probability of failure is performed (documented) as background for the chosen safety factors of the standards.
- A small structure with few legs/piles has less redundancy than a structure with many legs and piles and correspondingly a higher probability of failure.
- The designers choice of relevant pile capacity calculation method and of related soil shear strength parameters are more important for the overall safety related to pile foundation.
- Effects not normally accounted for in pile design may have large influence on the ‘real safety’, such as ageing effects and effects of cyclic loading.
### Table 5-3 Pile Design Formula Comparison

<table>
<thead>
<tr>
<th></th>
<th>API WSD</th>
<th>API LRFD</th>
<th>ISO 19902</th>
<th>NORSOK N-004</th>
</tr>
</thead>
<tbody>
<tr>
<td>3.2.1.4 Cylinder Head Pile</td>
<td>$u_0 (D_d)_d (s_d) + (u_n + f_d) (T_d) \leq 1.0$</td>
<td>$P_{av} = 0.05 \times 2.5 (D_d)^{1.5}$</td>
<td>$P_{av} = 0.05 \times 2.5 (D_d)^{1.5}$</td>
<td>$P_{av} = 0.05 \times 2.5 (D_d)^{1.5}$</td>
</tr>
<tr>
<td></td>
<td>$P_{av} = F_{av} \times 0.2 \times (D_d)^{1.5}$</td>
<td>$P_{av} = F_{av} \times 0.2 \times (D_d)^{1.5}$</td>
<td>$P_{av} = F_{av} \times 0.2 \times (D_d)^{1.5}$</td>
<td>$P_{av} = F_{av} \times 0.2 \times (D_d)^{1.5}$</td>
</tr>
<tr>
<td></td>
<td>(Only for D_d &lt; 500; not to be used for D_d &gt; 500)</td>
<td>(Only for D_d &lt; 500; not to be used for D_d &gt; 500)</td>
<td>(Only for D_d &lt; 500; not to be used for D_d &gt; 500)</td>
<td>(Only for D_d &lt; 500; not to be used for D_d &gt; 500)</td>
</tr>
<tr>
<td>3.3.4.2 Pile Foundation</td>
<td>The allowable pile capacities are determined by dividing the ultimate pile capacity by appropriate factors of safety which should not be less than the following values:</td>
<td>3.3.2.2 Pile Foundation Capacity</td>
<td>The axial pile capacity should satisfy the following conditions:</td>
<td>3.3.2.1 Axial Pile Resilience</td>
</tr>
<tr>
<td></td>
<td>1. Design environmental conditions with appropriate driving loads 1.5</td>
<td>$P_{av} \leq Q_{av}$</td>
<td>$P_{av} \leq Q_{av}$</td>
<td>For determination of design soil resistance against axial pile loads in ULS design, a material factor $\gamma = 1.2$ is to be applied to all characteristic values of soil resistance.</td>
</tr>
<tr>
<td></td>
<td>2. Operating environmental conditions during driving operations 2</td>
<td>$P_{av} \leq Q_{av}$</td>
<td>$P_{av} \leq Q_{av}$</td>
<td>$P_{av} \leq Q_{av}$</td>
</tr>
<tr>
<td></td>
<td>3. Design environmental conditions with appropriate loading conditions 1.6</td>
<td>where $Q_{av}$ is the ultimate axial pile capacity determined from a coupled linear structural and nonlinear foundation model using factored loads</td>
<td>$P_{av} \leq Q_{av}$</td>
<td>$P_{av} \leq Q_{av}$</td>
</tr>
<tr>
<td></td>
<td>4. Operating environmental conditions during loading operations 2</td>
<td>$\phi_5 = \phi_{5D} = 0.45$</td>
<td>$P_{av} \leq Q_{av}$</td>
<td>$P_{av} \leq Q_{av}$</td>
</tr>
<tr>
<td></td>
<td>5. Design environmental conditions with minimum loads (for published) 1.5</td>
<td>$\phi_5 = \phi_{5D} = 0.45$</td>
<td>$P_{av} \leq Q_{av}$</td>
<td>$P_{av} \leq Q_{av}$</td>
</tr>
<tr>
<td>3.1.6 Pile Wall Thickness</td>
<td>For piles that are to be installed by driving where sustained hard driving (250 blows per 1000 blows per meter with the largest size hammer to be used) is anticipated, the minimum pile wall thickness used should not be less than:</td>
<td>0.16.6 Minimum Wall Thickness</td>
<td>For piles that are to be installed by driving where sustained hard driving (250 blows per 1000 blows per meter with the largest size hammer to be used) is anticipated, the minimum pile wall thickness used should not be less than:</td>
<td>0.16.6 Minimum Wall Thickness</td>
</tr>
<tr>
<td></td>
<td>$t = 0.25 \times D_{100}$ or $t = 0.35 \times D_{100}$ (Metric Formula)</td>
<td>$t = 0.25 \times D_{100}$ or $t = 0.35 \times D_{100}$ (Metric Formula)</td>
<td>$t = 0.25 \times D_{100}$ or $t = 0.35 \times D_{100}$ (Metric Formula)</td>
<td>$t = 0.25 \times D_{100}$ or $t = 0.35 \times D_{100}$ (Metric Formula)</td>
</tr>
<tr>
<td></td>
<td>$t = \text{wall thickness, in. (mm)}$</td>
<td>$t = \text{wall thickness, in. (mm)}$</td>
<td>$t = \text{wall thickness, in. (mm)}$</td>
<td>$t = \text{wall thickness, in. (mm)}$</td>
</tr>
</tbody>
</table>
6 IN-SERVICE INSPECTION AND MAINTENANCE

A comparison of the requirements for the in-service inspection and maintenance is carried out based on the API, ISO, and NORSOK standards discussed in Sections 6.1 to 6.3 below. A summary of the comparison is given in Table 6-2.

6.1 API 2A, 2T, and 2FPS

The API RP-2A (WSD) is the main document where specific guidance with regards to the in-service inspection scope and frequency for fixed platforms is available. The in-service inspection requirements in the API standards addressing the design and operational aspects for floating production units (2FPS) and tension leg platforms (2T) are also included herein and compared to those given in the API RP-2A.

Generally speaking, the API RP-2A represents the traditional approach for the in-service survey requirements, focusing on the minimum intervals for the different inspection levels. Section 14 states that the time interval between periodical in-service inspections for the fixed platforms should not exceed the intervals presented in Table 6-2. The frequency of the in-service inspections/surveys is based on the exposure categorization and the consequence of failure considerations. These intervals may be adjusted based on case-by-case evaluation if justification of different intervals can be supported by engineering calculations or operational experience. No specific guidance for the evaluation procedure supporting the adjustment of the inspection frequency is however presented.

API RP-2A also specifies in detail the scope for each of the inspection levels (see Table 6-2).

In addition to the periodical survey program, the RP highlights the need for special surveys which should be conducted following:

- design environmental event, such as hurricane or earthquake (minimum Level I survey is recommended).
- severe accidental loading that could lead to structural degradation; e.g., boat collision or dropped objects (Level II survey is recommended)
- structural repairs of the members/areas critical for the structural integrity of the platform, approximately 1 year after the repairs (Level II or Level III survey, in presence of an extensive marine growth)

API RP-2T 3rd Edition gives detailed in-service inspection requirements in Section 15 that covers annual, intermediate and special periodical surveys with 1, 2-3, and 5 year intervals. The API RP 2T allows also a ‘continuous survey’ as an alternative to the special periodical survey. The requirements cover internal and external examinations, joints and connections, tendons, flex joints and foundations and include underwater inspections. The requirements do not however distinguish between the type of TLP and the built in redundancies and robustness.
The API RP 2FPS provides only a high-level guidance regarding survey requirements (see Section 7.6 of the RP). Alternatively, 2FPS allows the preparation of the in-situ inspection and maintenance program following the guidelines of Recognized Classification Societies (RCS). This approach is preferred by a large percentage of owners (operators) since it is mandatory for all units certified by RCS. It should however be stated that significant risk management requirements are present in the 2FPS document (Sec. 14) in comparison to 2A (Sec. 18.5) which may account for some reduction in the survey requirements if one applies the Risk Based Inspection (RBI) principles.

6.2 ISO 19902 and 19904-1

As discussed below, the ISO approach to in-service inspection requirements adopts the Structural Integrity Management (SIM) methodology and also applies RBI procedures.

The main focus of Clause 23 of the ISO 19902 is on the detailed guidance and the requirements for the SIM system for fixed offshore structures. The standards specify that they apply to “fixed steel offshore structures located anywhere in the world, built to any design and fabrication standards, and of any age”, highlighting inherently that the degradation mechanisms and failure modes of structures installed in the marine environment are similar, regardless of the basis for design. However, consideration needs to be given to the specifics of the installation degradation rates, resulting from the design and site environmental factors, the loading and operating history, and the effectiveness of the preventive measures (i.e. coating or cathodic protection system). The high-level schematic of the SIM cycle is shown in Figure 6-1. The four stages of the SIM are shown to involve data collection and evaluation and development of inspection strategy and a detailed inspection program.

![Figure 6-1 Phases of a structural integrity management cycle (ISO 19902)](image-url)
The ISO 19902 also provides detailed description of each these four activities within the SIM programme. The standard states that the inspection strategy should contain scheduled and unscheduled inspections. The scheduled inspections are divided into the following sub-categories:

- Baseline inspection – inspection conducted as soon as practical after installation and commissioning (if possible, within first year of operation) to establish the as-installed condition
- Periodic inspection – regular in-service inspection, with timing and scope of work determined based on the inspection strategy and inspection programme
- Special inspection– to monitor known defects, damage, scour, etc. and to assess the performance of repairs undertaken to assure fitness-for-purpose of the structure (conducted approximately 1 year after completion of the repair)

Similar to API RP 2A, the unscheduled inspections are required to evaluate a structure’s condition following an environmental event (i.e. hurricane) or incident (i.e. boat collision).

As an alternative to the SIM, the ISO 19902 presents the requirements for the default periodical inspection requirements. As shown in Table 6-2, the ISO standard follows the API RP-2A philosophy (API requirements were directly adopted, with some minor changes for inspection intervals where ISO requires the lower bounds of the API allowable timeframe for corresponding inspection levels).

The philosophy of the in-service inspections and maintenance of the floating installations presented in the ISO 19904-1 (Clause 18 of the standard), follows the above discussed philosophy of the ISO 19902. The default inspection intervals and scope are also presented. However, similar to ISO 19902, the ISO 19904 focuses on the requirements for the SIM program. It is also stated that the requirements of the RCS which classified the unit should be implemented in the inspection program. A separate issue of the inspection planning related to confined spaces and usually closed areas is also addressed.

6.3 NORSOK N-005

The N-005 standard /27/ presents only high level requirements regarding in-service inspection program. The platform operator is responsible for preparing this program based on the characteristic of the structure, loading history, and inspection findings. No specific requirements regarding the inspection intervals are presented. However the document includes details regarding preparations for inspection and underwater inspection methods (see Table 6-1). More detailed guidelines regarding the in-service inspection of various types of structures (jackets, column stabilized units, ship-shaped vessels and concrete structures) are presented in the normative Annexes C through F, of the standard, respectively.
Table 6-1 Underwater Inspection Methods

<table>
<thead>
<tr>
<th>Methods/techniques</th>
<th>Capability</th>
<th>Technical description</th>
<th>ROV applicability</th>
<th>Cost related aspects</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Visual</strong></td>
<td>Without cleaning</td>
<td>Scour, sea floor instability, gross damage, signs of gross damage, existence of anodes</td>
<td>Video based reporting.</td>
<td>ROV</td>
</tr>
<tr>
<td></td>
<td>With cleaning</td>
<td>Follow-up investigation of general damage</td>
<td>Video based reporting</td>
<td>Work class ROV</td>
</tr>
<tr>
<td><strong>Electronic</strong></td>
<td>MPE</td>
<td>Fatigue damage</td>
<td>Existence and length of surface crack</td>
<td>Work class ROV</td>
</tr>
<tr>
<td></td>
<td>EC</td>
<td>Fatigue damage</td>
<td>Length of surface crack. May also be used for depth measurement. Independent evaluation possible</td>
<td>ROV</td>
</tr>
<tr>
<td></td>
<td>ACPD</td>
<td>Fatigue damage</td>
<td>Depth of surface crack. Supporting MPE</td>
<td>Work class ROV</td>
</tr>
<tr>
<td></td>
<td>ACFM</td>
<td>Fatigue damage</td>
<td>Both length and depth of surface crack. Independent evaluation possible</td>
<td>ROV</td>
</tr>
<tr>
<td><strong>Ultrasound</strong></td>
<td></td>
<td>Wall thickness, corrosion, fatigue and fabrication defect, post-repair inspection</td>
<td>Embedded defects.</td>
<td>Work class ROV, Usually performed by diver</td>
</tr>
<tr>
<td><strong>FMD</strong></td>
<td>Fatigue damage and post-event damage</td>
<td>Through thickness crack. Excellent tool for rapid screening.</td>
<td>ROV</td>
<td>Fast</td>
</tr>
<tr>
<td><strong>Cathodic Potential</strong></td>
<td>Corrosion</td>
<td>Anode performance. Often combined with visual inspection</td>
<td>ROV</td>
<td></td>
</tr>
<tr>
<td><strong>Other</strong></td>
<td>Dimensional measurement</td>
<td>Scour, subsidence, marine growth, dent, out-of-straightness, corrosion pit size, etc</td>
<td>For special purposes.</td>
<td>ROV</td>
</tr>
<tr>
<td></td>
<td>Stress &amp; def. monitoring</td>
<td>Structural behaviour monitoring</td>
<td>For special purposes.</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Radiography</td>
<td>E.g., testing of hyperbaric welds</td>
<td>Internal defects</td>
<td></td>
</tr>
</tbody>
</table>

6.4 Summary

All standards emphasize the importance of keeping records of performed in-service inspections, maintenance and structural modifications of the platform. The synergy between different phases of the structural integrity management is highlighted as one of the most important factors extending the lifetime of the structure and increasing the safety of operations.

The standards also define Owner’s responsibility for preparation and proper execution of the inspection program, which may result in decreased (or increased, depending on an outcome of the evaluation and findings of the historical surveys) pre-defined frequencies for different levels of in-service inspections. It is also highlighted that analyzing the inspection findings and implementing a SIM program can reduce costs related to maintenance.

In the ISO and NORSOK standards more significant attention is given to the risk assessment and probability based inspection methods. This approach often requires using of advanced analysis methods and is aligned with current trends and developments in the field of structure integrity management. The API current standard, representing more traditional approach, allows for the adjustment of the inspection scope and frequency. However, it does not provide requirements for
SIM. Detailed guidance for SIM and the Risk Based Inspection (RBI) planning will be implemented into the API system in the planned API RP-2SIM for publication in 2011 or 2012.

All standards also include list of preselected areas and minimum inspection requirements for periodical inspections. These minimum requirements for floating installations should also be reviewed and updated based on the requirements of the RCS classifying the vessel, if they are found to be more conservative.
## Table 6-2 In-Service Inspection Requirements Comparison

<table>
<thead>
<tr>
<th>API RP 2A WSD/LRFD</th>
<th>ISO 19902</th>
<th>NORSOK N-005</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>LEVEL I</strong></td>
<td><strong>LEVEL I</strong></td>
<td><strong>LEVEL I</strong></td>
</tr>
<tr>
<td>Below water verification of performance of the cathodic protection system (i.e., dropped cell), and an above water visual survey to determine the effectiveness of corrosion protection system, detected blistering coating systems, excessive corrosion, and bent, misaligned and damaged members. General examination of structural members in splash zone and above water, concentrating on condition of the more critical areas such as deck legs, girders, stringers, etc. Survey should identify indications of obvious overloading, design deficiencies, and use inconsistent with the platform's original purpose. If above-water damage is detected, NDT should be used when visual inspection cannot fully determine the extent of damage. Should Level I survey indicate that underwater damage could have occurred, a Level II inspection is required.</td>
<td>Below water verification of performance of the cathodic protection system (i.e., dropped cell), and an above water visual survey to determine the effectiveness of corrosion protection system, detected blistering coating systems, excessive corrosion, and bent, misaligned and damaged members. General examination of structural members in splash zone and above water, concentrating on condition of the more critical areas such as deck legs, girders, stringers, etc. Survey should identify indications of obvious overloading, design deficiencies, and use inconsistent with the platform's original purpose. If above-water damage is detected, NDT should be used when visual inspection cannot fully determine the extent of damage. Should Level I survey indicate that underwater damage could have occurred, a Level II inspection is required.</td>
<td>General underwater visual inspection by divers or ROVs to detect presence of excessive corrosion, accidental or environmental overloading, stress and seawater fatigue, design or construction deficiencies, presence of debris, and excessive marine growth. This survey should include measurement of existing structural and non-structural areas. Selection of significant structural damage during Level II survey should become the basis for initiation of Level III survey, which should be conducted as soon as conditions permit.</td>
</tr>
<tr>
<td><strong>LEVEL II</strong></td>
<td><strong>LEVEL II</strong></td>
<td><strong>LEVEL II</strong></td>
</tr>
<tr>
<td>General underwater visual inspection by divers or ROVs to detect presence of excessive corrosion, accidental or environmental overloading, stress and seawater fatigue, design or construction deficiencies, presence of debris, and excessive marine growth. This survey should include measurement of existing structural and non-structural areas. Selection of significant structural damage during Level II survey should become the basis for initiation of Level III survey, which should be conducted as soon as conditions permit.</td>
<td>General underwater visual inspection by divers or ROVs to detect presence of excessive corrosion, accidental or environmental overloading, stress and seawater fatigue, design or construction deficiencies, presence of debris, and excessive marine growth. This survey should include measurement of existing structural and non-structural areas. Selection of significant structural damage during Level II survey should become the basis for initiation of Level III survey, which should be conducted as soon as conditions permit.</td>
<td>General underwater visual inspection by divers or ROVs to detect presence of excessive corrosion, accidental or environmental overloading, stress and seawater fatigue, design or construction deficiencies, presence of debris, and excessive marine growth. This survey should include measurement of existing structural and non-structural areas. Selection of significant structural damage during Level II survey should become the basis for initiation of Level III survey, which should be conducted as soon as conditions permit.</td>
</tr>
<tr>
<td><strong>LEVEL III</strong></td>
<td><strong>LEVEL III</strong></td>
<td><strong>LEVEL III</strong></td>
</tr>
<tr>
<td>An underwater visual inspection of preselected areas and/or, based on results of Level II survey, areas shown to have suspected damage. Such areas should be sufficiently cleaned of marine growth to permit thorough inspection. PDR can provide an acceptable alternative to close visual inspection (CVI). CVI for corrosion monitoring should be included as part of Level III survey. Selection of significant structural damage during Level III survey should become the basis for initiation of Level IV survey, where CVI alone cannot determine the extent of damage. Level IV survey, if required, should be conducted as soon as conditions permit.</td>
<td>An underwater visual inspection of preselected areas and/or, based on results of Level II survey, areas shown to have suspected damage. Such areas should be sufficiently cleaned of marine growth to permit thorough inspection. PDR can provide an acceptable alternative to close visual inspection (CVI). CVI for corrosion monitoring should be included as part of Level III survey. Selection of significant structural damage during Level III survey should become the basis for initiation of Level IV survey, where CVI alone cannot determine the extent of damage. Level IV survey, if required, should be conducted as soon as conditions permit.</td>
<td>An underwater visual inspection of preselected areas and/or, based on results of Level II survey, areas shown to have suspected damage. Such areas should be sufficiently cleaned of marine growth to permit thorough inspection. PDR can provide an acceptable alternative to close visual inspection (CVI). CVI for corrosion monitoring should be included as part of Level III survey. Selection of significant structural damage during Level III survey should become the basis for initiation of Level IV survey, where CVI alone cannot determine the extent of damage. Level IV survey, if required, should be conducted as soon as conditions permit.</td>
</tr>
<tr>
<td><strong>LEVEL IV</strong></td>
<td><strong>LEVEL IV</strong></td>
<td><strong>LEVEL IV</strong></td>
</tr>
<tr>
<td>An underwater NDT of preselected areas and/or, based on results of Level III survey, areas known or suspected damage. Level IV survey should also include detailed inspection and measurements of damaged areas. A Level III and/or Level IV survey of fatigue-sensitive joints and/or areas susceptible to cracking could be necessary to determine if damage has occurred. Monitoring fatigue-sensitive and/or reported crack-like indications can be an acceptable alternative to analytical verification.</td>
<td>An underwater NDT of preselected areas and/or, based on results of Level III survey, areas known or suspected damage. Level IV survey should also include detailed inspection and measurements of damaged areas. A Level III and/or Level IV survey of fatigue-sensitive joints and/or areas susceptible to cracking could be necessary to determine if damage has occurred. Monitoring fatigue-sensitive and/or reported crack-like indications can be an acceptable alternative to analytical verification.</td>
<td>An underwater NDT of preselected areas and/or, based on results of Level III survey, areas known or suspected damage. Level IV survey should also include detailed inspection and measurements of damaged areas. A Level III and/or Level IV survey of fatigue-sensitive joints and/or areas susceptible to cracking could be necessary to determine if damage has occurred. Monitoring fatigue-sensitive and/or reported crack-like indications can be an acceptable alternative to analytical verification.</td>
</tr>
</tbody>
</table>
### 14.4 SURVEY FREQUENCY

<table>
<thead>
<tr>
<th>Exposure Category</th>
<th>I</th>
<th>II</th>
<th>III</th>
<th>IV</th>
</tr>
</thead>
<tbody>
<tr>
<td>L-1</td>
<td>1 yr</td>
<td>5-6 yr</td>
<td>10-15 yr</td>
<td>*</td>
</tr>
<tr>
<td>L-2</td>
<td>1 yr</td>
<td>6-10 yr</td>
<td>15-20 yr</td>
<td>*</td>
</tr>
<tr>
<td>L-3</td>
<td>1 yr</td>
<td>10 yr</td>
<td>20 yr</td>
<td>*</td>
</tr>
<tr>
<td>L-4</td>
<td>1 yr</td>
<td>15 yr</td>
<td>30 yr</td>
<td>*</td>
</tr>
</tbody>
</table>

Survey should be performed if required for lower level inspection findings.

### 23.7 DEFAULT PERIODIC INSPECTION REQUIREMENTS

In the absence of an in-service structural inspection strategy, default requirements shall apply. These requirements address only the concerns of safeguarding life and protecting the environment. Additional inspections can be needed to meet statutory requirements, a client's corporate policy or industry standards/practice. Compliance with default requirements does not guarantee structural reliability or fitness-for-purpose.

<table>
<thead>
<tr>
<th>Survey Intervals</th>
<th>Exposure Level</th>
<th>I</th>
<th>II</th>
<th>III</th>
<th>IV</th>
</tr>
</thead>
<tbody>
<tr>
<td>L-1</td>
<td>Annual</td>
<td>3 yrs</td>
<td>5 yrs</td>
<td>7 yrs</td>
<td>10 yrs</td>
</tr>
<tr>
<td>L-2</td>
<td>Annual</td>
<td>5 yrs</td>
<td>10 yrs</td>
<td>not required</td>
<td>not required</td>
</tr>
<tr>
<td>L-3</td>
<td>Annual</td>
<td>7 yrs</td>
<td>15 yrs</td>
<td>not required</td>
<td>not required</td>
</tr>
<tr>
<td>L-4</td>
<td>Annual</td>
<td>10 yrs</td>
<td>not required</td>
<td>not required</td>
<td>not required</td>
</tr>
</tbody>
</table>

### 4.3 CONDITION MONITORING PRINCIPLES

Operators shall carry out the condition monitoring installation in a systematic manner. This may include development of overall philosophy and strategy for condition monitoring, establishing in-service inspection systems and long-term inspection programs, etc.

The condition monitoring programme is subject to continuous updating as it involves many factors, such as environmental conditions, failure probabilities, damage development, etc. In addition, a revision of the programme may also be necessary as a result of development of tools and methods.

### 5.3 INTERVALS FOR CONDITION MONITORING

**Periodic framework programme – usually 3-5 years**

- Initial condition survey – within 1 year after installation
- Periodic condition monitoring – shall be carried out regularly

### 14.5 SELECTED SURVEY AREAS

- Critical areas
- Areas affected by external loads
- Areas affected by welding
- Areas affected by structural changes
- Areas affected by fatigue

### 14.6 RECORDS

Records of all surveys shall be retained by the operator for the lift platform. Such records should contain detailed accounts of survey findings, including video tapes, photographs, measurements, and other pertinent survey results. Records should also identify the survey levels performed.

### 23.2 DATA COLLECTION AND UPDATE

Records of all original design analyses, fabrication, transportation, installation, and in-service inspection, engineering evaluations, reports, and incidents shall be retained by the owner for the life of the structure and transmitted to new owners as necessary.

### 6.4 INSPECTION RECORD

Operators shall maintain an up-to-date filing system for records and evaluations from condition monitoring programmes throughout the lifetime of the installation. The data may include video tape, inspection logs, final hand-inspection report, evaluation and recommendations.

Such records shall also include test techniques employed, planned and actual course of work and description of findings and any anomalies discovered.
7 ASSESSMENT OF EXISTING PLATFORMS AND FLOATERS

7.1 General

Offshore structures have been built since the 1940’s in the GOM and around the world. Design codes evolve as knowledge is attained through actual operational experience. The first Edition of API RP 2A was published in 1969 with a total of 16 pages. The API RP 2A 21st Edition of 2000 with supplements added in 2002, 2005, and 2007 is now 274 pages long which indicates that considerable guidance and experience have been accomplished. Naturally some existing structures will not meet the full requirements of newer editions of the standards and hence the need for a methodology for the assessment of these existing structures.

The assessment of existing structures methodology applies risk based principles and even though it allows the use of reduced assessment design criteria compared to the criteria for design of a new structure, it stipulates that the risks are managed effectively and that consequences of damage/or failure are acceptable to the operators and regulators.

Table 7-1 depicts a comparison between requirements for assessment of an existing structure compared to design criteria for a new structure covering environmental criteria, loading conditions, foundation design, modeling, stress analysis, and acceptance criteria. The table demonstrates that for assessment of existing structures actual platform data and experience is taken into account thus eliminating some of the conservatism employed in design of a new structure. The acceptance of local yield is possible for an existing structure provided alternative load paths through redundancy are demonstrated by analysis.
### Table 7-1 Assessment versus Design Comparison

<table>
<thead>
<tr>
<th>Design Criteria</th>
<th>Assessment Criteria</th>
</tr>
</thead>
<tbody>
<tr>
<td>Environment</td>
<td>Forecast from existing data collection</td>
</tr>
<tr>
<td>Loading</td>
<td>Possibly conservative evaluation from proposed use of structure</td>
</tr>
<tr>
<td>Foundation</td>
<td>Forecast from site investigation and laboratory testing of soils</td>
</tr>
<tr>
<td>Modeling</td>
<td>Topology and dimensions may be changed. No service inspection available. Conservative modeling using global percentages to cover not-finalized details and simple geometric assumptions</td>
</tr>
<tr>
<td>Stress Analysis</td>
<td>The time for analysis is critical. Strict compliance with code of practice and regulatory documents.</td>
</tr>
<tr>
<td>Results Evaluation</td>
<td>Structure has members and joints with acceptable utilization.</td>
</tr>
</tbody>
</table>
7.2 API RP 2A and Bulletin 2INT-EX

This standard gives detailed existing structures assessment procedures included in Section 17 of API RP 2A and its commentary section. Section 17 defines reduced design criteria for assessment purposes that are applicable only for the assessment of platforms designed in accordance with the 20th or earlier editions and prior to the first edition of API RP 2A. The specified reduced environmental criteria are not intended to be used to justify modifications or additions to a platform that will result in an increased loading on the structure for platforms that have been in service less than five years. For structures designed according to the 21st or later Editions, assessment is required to be in accordance with the criteria originally used for the design of the platform, unless a special study can justify a reduction in Exposure Category as defined in Section 1 of API RP 2A.

The trigger elements of selection of platforms for assessment, categorization of safety level for the installation and condition assessment in API RP 2A do not differ from those given in ISO 19902. The assessment process is depicted in the flow charts given in Figure 7-1 and Figure 7-2 taken from Section 17 of API RP 2A.

As stated in Section 2.2.1 API issued Bulletin 2INT-EX in order to provide guidance to operators and designers on the application of the new Metocean criteria given in API Bulletin 2INT-MET which had significant wave height increase of ~ 30% in the central region. The Bulletin described assessment ultimate strength procedures and recommended a minimum reserve strength ratio (RSR) of 1.2 for A-1 or L-1 structures. The RSR is defined as the ratio of the ultimate lateral load the structure can sustain before collapse to the base shear calculated for the 100-year Metocean condition.

Both API RP 2A Section 17 and API Bulletin 2INT-EX will be replaced by the upcoming API RP 2SIM which will employ Structural Integrity Management (SIM) and Risk Based Inspection (RBI) methodologies in performing the assessment.

There are two potential sequential analysis checks mentioned in API RP 2A WSD, a design level analysis and an ultimate strength analysis. The analysis itself seems to be the same as mentioned in ISO 19902, but the environmental loads are different. The environmental load in API RP 2A may be reduced to 85% of the 100-yr condition for high consequence platforms, and to 50% for low consequence platforms in other U.S. areas except GoM. API states that the design level analysis is not applicable for platforms with inadequate deck height and the one-third increase in allowable stress is permitted for design level analysis (all categories).

As defined above, in the ultimate strength analysis, the Reserve Strength Ratio (RSR) is the ratio of the platforms ultimate lateral load carrying capacity to its 100-yr environmental condition lateral loading. As noted in Figure 7-1, an RSR of 1.6 is required for high consequence platforms and 0.8 for low consequence platforms in US waters other than the GOM. No RSR values are specified however for the GOM structures. Instead the ratio of the maximum wave height corresponding to ultimate capacity and the design wave height is evaluated to be about 1.3. The reduced assessment criteria are given the Commentary to Section 17 as shown in Table 7-2 for 400 ft. water depth.
Table 7-2 Comparison of Wave Criteria for New L-1 and Assessment Criteria

<table>
<thead>
<tr>
<th>API RP 2A Criteria</th>
<th>Wave Height Criteria</th>
<th>Design Level Assessment Height / Annual Return Period</th>
<th>Ultimate Strength Assessment Height / Annual Return Period</th>
</tr>
</thead>
<tbody>
<tr>
<td>New Design (Section 2, L-1)</td>
<td>Gulf of Mexico, 400 ft. Water Depth*</td>
<td>70 ft / 100 yr.</td>
<td>Not Applicable</td>
</tr>
<tr>
<td>A-1 High (Section 17)</td>
<td></td>
<td>57 ft / 30 yr.</td>
<td>74 ft / 200 yr.</td>
</tr>
<tr>
<td>A-2 Medium (Section 17)</td>
<td></td>
<td>48 ft / 15 yr.</td>
<td>62 ft / 45 yr.</td>
</tr>
<tr>
<td>A-3 Low (Section 17)</td>
<td></td>
<td>38 ft / &lt;10 yr.</td>
<td>48 ft / 15 yr.</td>
</tr>
</tbody>
</table>

The requirement to deck elevations versus water depth is provided for GoM in API Figures 17.6.2-2b 17.6.2-3b. The following guidelines are recommended in the code:

1. The ultimate strength of undamaged members, joints, and piles can be established using the formulas of Sections 3, 4, 6 and 7 (API) with all safety factors removed. The ultimate strength of joints may also be determined using a mean “formula or equation” versus the lower bound formulas for joints in Section 4 (API).
2. The ultimate strength of damaged or repaired elements of the structure may be evaluated using a rational, defensible engineering approach, including special procedures developed for the purpose.
3. Actual (coupon test) or expected mean yield stresses may be used instead of nominal yield stresses. Increased strength due to strain hardening may also be acknowledged if the section is sufficiently compact, but not rate effects beyond the normal (fast) mill tension tests.
4. Studies and tests have indicated that effective length (K) factors are substantially lower for elements of a frame subjected to overload than those specified in 3.3.1d (API). Lower values may be used if it can be demonstrated that they are both applicable and substantiated.

In addition, three alternative assessment procedures subject to specified limitations are considered as acceptable:

- assessment of similar platforms by comparison
- assessment through the use of explicit probabilities of failure
- assessment based on prior exposure, surviving actual exposure to an event that is known with confidence to have been either as severe or more severe than the applicable ultimate strength criteria based on the exposure category

The assessment process described in Section 17 of API RP 2A WSD and API Bulletin 2INT-EX include significant detail covering initiators, categories A-1 to A-3, surveys, environmental loading, structural analysis, and mitigation alternatives. The assessment also includes fatigue, and strength evaluations. The ultimate strength analysis is only required to determine the RSR as stated above.
Figure 7-1 Platform Assessment Process
Figure 17.5.2—Platform Assessment Process—Metoean Loading (Continued)

Figure 7-2 Platform Assessment Process (Continued)
7.3 ISO 19902 (2007)

Assessment of existing structures is covered in Clause 24 of ISO 19902. It states that the owner shall maintain and demonstrate the fitness-for-purpose of the structure for its specific site conditions and operational requirements. A structure is deemed fit-for-purpose when the risk of structural failure leading to unacceptable consequences is sufficiently low. The acceptable level of risk depends on regulatory requirements supplemented by regional or industry standards and practice. The aims and procedures are applicable to the assessment of existing fixed steel offshore structures as well as topsides structures. The ISO 19902 states that it is permissible to accept limited individual component “failure” for existing structures, provided that both the reserve strength against overall system failure and deformations remain acceptable. The ISO 19902 assessment procedure includes both a check of the ultimate limit state and the fatigue limit state.

A flow chart of the assessment is shown in Figure 7-3 (see ISO 19902 Fig. 24.2-2). Three potential assessment checks are specified in order of complexity:

a) Screening the structure in comparison with similar structures
b) Design level analysis: a check of the structure following the same approach as for a new design
c) Ultimate level analysis: intended to demonstrate that a structure has adequate strength and stability to withstand a significant overload. Local overstress and potential local damage are acceptable, but total collapse or excessive/damaging deformations shall be avoided.

Further details of the assessment initiators, acceptance criteria, platform condition, actions, resistance, and screening, design level, ultimate strength assessment parameters are given in the Table 7-2. ISO does not give any specific requirement to the reliability of existing platforms. The owner needs to develop them in addition to the code. Making reference to only ISO can lead to any level of safety.
Figure 7-3 Flow chart of the assessment process
7.4 NORSOK N-006

This standard was first published in 2009 and covers general principles and guidelines for assessment of existing offshore structures as a supplement to high level NORSOK N-001. NORSOK N-006 should be used in conjunction with NORSOK N-003, NORSOK N-004 and NORSOK N-005 on actions, design, and condition monitoring of offshore structures, respectively.

The general principles given in this standard are applicable to:

- All types of offshore structures including bottom founded structures as well as floating structures
- Different types of materials used including steel, concrete, aluminum, etc.
- The assessment of complete structures including substructures, topside structures, vessel hulls, foundations, marine systems, mooring systems, subsea facilities and mechanical outfitting that contributes to maintain the assumed load conditions of the structure

The initiation elements for selection of platforms for assessment for the installation and condition assessment do not differ much from those given in ISO 19902 and API RP 2A.

The flow chart of the assessment process is shown in Figure 7-4 and it is applicable to all relevant limit states. The same principles for check of ULS, ALS and FLS as for design of new structures apply to assessment of existing structures.

There is some special guidance in this standard not covered in API and ISO:

- The resistance of damaged steel members and corroded steel members can be calculated according to NORSOK N-004, Section 10
- Resistance to cyclic storm actions included in Section 8.4
- Risk based inspection is included in Section 9.
- Existing facilities where the primary structure does not meet the criteria for ULS or ALS related to environmental actions that can be forecast like wave and wind actions, may continue to be used if the following four requirements are fulfilled:
  - shut-down and unmanning procedures are implemented. The procedure for shut down and unmanning should meet criteria given in 6.3.
  - requirements to unmanned facilities according to NORSOK N-001 are satisfied.
  - the environmental actions will not jeopardize any other main safety function (other than structural integrity) relevant for the facility during the storm, see 6.4
  - the risk of significant pollution is found acceptance. see 6.5
Figure 7-4 Flow Sheet of the Assessment Process
More detailed comparison of the assessment requirements is given in Table 7-3 for the three codes and methods for assessing damaged/corroded members and damaged joints are presented in Table 7-4 for the ISO and NORSOK. No such guidance is given in API. It should be noted that unlike API and ISO, NORSOK does not allow lower assessment criteria than the highest L-1 or A-1 for manned platforms. However, in N-001 relaxed requirements are formulated for platforms that are normally unmanned.

7.5 Summary of Assessment of Existing Platforms Comparison

The methodology for assessment of existing structures applies risk based principles. Reduced assessment design criteria (compared to the criteria for design of a new structure) are specified in API 2A Section 17.

The ISO 19902 does not have reduced assessment criteria but allows local damage to be sustained provided reserve strength is verified. NORSOK requires that existing structures be able to resist ULS and ALS conditions at same safety level as new structures. If they fail to meet these requirements, mitigation measures must be implemented. ISO does not give any specific requirements for assessment of existing structures.
Table 7-3 Assessment of Existing Platforms and Floaters Comparison

<table>
<thead>
<tr>
<th>Limitation</th>
<th>API RP 2A- WSD Section 17 (To be replaced by API RP 2SDM)</th>
<th>ISO 19902</th>
<th>NORSOK N-006</th>
</tr>
</thead>
<tbody>
<tr>
<td>- Only for the assessment of the following platforms:</td>
<td>- The assessment of existing fixed steel offshore structures to demonstrate their fitness-for-purpose</td>
<td>- applicable to all types of offshore structures, including bottom-founded structures as well as floating structures; As the majority of ageing facilities are fixed structures of the jacket type, the detailed recommendations given are most relevant for this type of structure;</td>
<td></td>
</tr>
<tr>
<td>- designed in accordance with the provisions in the 20th and earlier editions</td>
<td>- applicable to topsides structures</td>
<td>- applicable to different types of materials used including steel, concrete, aluminium;</td>
<td></td>
</tr>
<tr>
<td>- the platforms designed prior to the first editions</td>
<td>- applicable to topsides structures</td>
<td>- applicable to the assessment of complete structures including substructures, topsides structures, vessel hulls, foundations, marine systems, mooring systems, subsea facilities and mechanical outfitting that contributes to maintain the assumed load conditions of the structure</td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Assessment Process</th>
<th>24.2</th>
<th>4.5</th>
<th>4.2</th>
</tr>
</thead>
<tbody>
<tr>
<td>1. Platform selection (Section 17.2)</td>
<td>a) assemble data on the structure, its history and exposure level, see 24.3</td>
<td>- design, fabrication and installation resume and as-built drawings</td>
<td>a) changes from the original design or previous assessment basis, including</td>
</tr>
<tr>
<td>2. Classification (Section 17.3)</td>
<td>b) determine if any assessment initiators are triggered, see 24.4</td>
<td>- documentation of as-is condition</td>
<td>- modification to the facilities,</td>
</tr>
<tr>
<td>3. Condition assessment (Section 17.4)</td>
<td>c) determine acceptance criteria, see 24.5</td>
<td>- planned changes and modifications of the facility</td>
<td>- more onerous environmental conditions and/or criteria,</td>
</tr>
<tr>
<td>4. Design basis check (Sections 17.5 and 17.6)</td>
<td>d) assess the condition of the structure, see 24.6</td>
<td>- updated design basis and specifications</td>
<td>- more onerous component or foundation resistance data</td>
</tr>
<tr>
<td>5. Analysis check (Sections 17.6 and 17.7)</td>
<td>e) assess the actions, see 24.7</td>
<td>- calibration of analysis models to measurements of behavior if such measurements exist</td>
<td>- physical changes to the structure's design basis</td>
</tr>
<tr>
<td>6. Consideration of mitigations (Section 17.8)</td>
<td>f) screen the structure in comparison with similar structures, see 24.8</td>
<td>- the history of degradations and incidents</td>
<td>- inadequate deck height</td>
</tr>
<tr>
<td></td>
<td>g) perform a resistance assessment, see 24.9 using</td>
<td>- prediction of future degradations and incidents</td>
<td>- b) damage or deterioration of a primary structural component</td>
</tr>
<tr>
<td></td>
<td>1) design level analysis</td>
<td>- the effect of degradation on future performance of the structure</td>
<td></td>
</tr>
<tr>
<td></td>
<td>2) ultimate strength level analysis</td>
<td>- a documentation of technical and operational integrity</td>
<td></td>
</tr>
<tr>
<td></td>
<td>3) prevention and mitigation, see 24.10</td>
<td>- planned mitigations</td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Section 17.2</th>
<th>24.4</th>
<th>4.7</th>
</tr>
</thead>
<tbody>
<tr>
<td>Definition of Significant: The total of the cumulative changes in greater than 10%</td>
<td>a) changes from the original design or previous assessment basis, including</td>
<td>a) changes from the original design or previous assessment basis, including</td>
</tr>
<tr>
<td>- Additional of personnel: life safety level changed to a more restrictive level</td>
<td>modification to the facilities</td>
<td>- modification to the facilities,</td>
</tr>
<tr>
<td>- Addition of facilities: addition of facilities or the consequence of failure level changed significantly</td>
<td>1) addition of personnel or facilities</td>
<td>- more onerous environmental conditions and/or criteria,</td>
</tr>
<tr>
<td>- Increased loading on structure: the new combined environmental/operational loading significantly increased</td>
<td>2) modification to the facilities</td>
<td>- more onerous component or foundation resistance data</td>
</tr>
<tr>
<td>- Inadequate deck height: platform with inadequate deck height for its exposure category and not designed for the impact of wave loading on the deck</td>
<td>3) more onerous environmental conditions and/or criteria</td>
<td>- physical changes to the structure's design basis</td>
</tr>
<tr>
<td>- Damage found during inspections: significant damage to primary structural components found during any inspection</td>
<td>4) more onerous component or foundation resistance data and/or criteria</td>
<td>- inadequate deck height</td>
</tr>
<tr>
<td></td>
<td>5) physical changes to the structure's design basis, e.g. excessive scour or subsidence</td>
<td>b) damage or deterioration of a primary structural component</td>
</tr>
<tr>
<td></td>
<td>6) inadequate deck height, such that waves associated with previous or new criteria will impact the deck, and provided such action was not previously considered</td>
<td>c) exceedance of design service life</td>
</tr>
<tr>
<td></td>
<td>a) Changes from the original design or previous assessment basis, including</td>
<td>- the fatigue life (including safety factors) is less than the required extended service life</td>
</tr>
<tr>
<td></td>
<td>b) physical changes to the structure's design basis, e.g. excessive scour or subsidence</td>
<td>- degradation of the structure due to corrosion is present, or is likely to occur, within the required extended service life</td>
</tr>
<tr>
<td></td>
<td>c) exceedance of design service life, if either</td>
<td>- degradation of the structure beyond design allowances, or is likely to occur within the required extended service life</td>
</tr>
<tr>
<td></td>
<td>d) exceedance of design service life</td>
<td>- the remaining fatigue life (including design fatigue factors) is less than the required extended service life</td>
</tr>
<tr>
<td></td>
<td>e) the fatigue life (including safety factors) is less than the required extended service life</td>
<td>- the fatigue life (including safety factors) is less than the required extended service life</td>
</tr>
<tr>
<td></td>
<td>f) exceedance of design service life, if either</td>
<td>- degradation of the structure beyond design allowances, or is likely to occur within the required extended service life</td>
</tr>
<tr>
<td></td>
<td>g) exceedance of design service life</td>
<td>- the remaining fatigue life (including design fatigue factors) is less than the required extended service life</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Section 17.3</th>
<th>24.5</th>
<th>4.8</th>
</tr>
</thead>
<tbody>
<tr>
<td>Assessment categories based on: Life safety, Consequence of failure</td>
<td>Acceptance criteria for assessment depend on the exposure level of the platform.</td>
<td>Acceptance criteria for assessment depend on the exposure level of the platform.</td>
</tr>
<tr>
<td>Life Safety</td>
<td>Life safety category 1 - High consequence</td>
<td>NORSOK defines lower safety factors for unmanned platforms</td>
</tr>
<tr>
<td>Manned-Non-Evacuated</td>
<td>Life safety category 2 - Medium consequence</td>
<td></td>
</tr>
<tr>
<td>Manned-Evacuated</td>
<td>Life safety category 3 - Low consequence</td>
<td></td>
</tr>
<tr>
<td>Unmanned</td>
<td>Life safety category 1 - High consequence</td>
<td></td>
</tr>
<tr>
<td>A-1 - High Assessment Category: existing major platforms and/or those platforms that have potential for well flow of either oil or sour gas in the event of failure. All platforms in water depths greater than 400 ft are considered A-1</td>
<td>Life safety category 2 - Medium consequence</td>
<td></td>
</tr>
<tr>
<td>A-2 - Medium Assessment Category: existing platforms where production would be shut-in during the design event; existing platforms that do not meet the A-1 or A-3 definitions</td>
<td>Life safety category 3 - Low consequence</td>
<td></td>
</tr>
<tr>
<td>A-3 - Low Assessment Category: existing platforms where production would be shut-in during the design event;</td>
<td>24.6</td>
<td>See 4.1</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Section 17.4</th>
<th>24.6</th>
<th>See 4.1</th>
</tr>
</thead>
<tbody>
<tr>
<td>Condition of the structure</td>
<td>Topsores surveys</td>
<td>- Topsores surveys</td>
</tr>
<tr>
<td>- Only for the annual Level I survey: topside arrangement and configuration, platform exposure category, structural framing details etc.</td>
<td>- Underwater and splash zone surveys: Level II inspection as a minimum</td>
<td></td>
</tr>
<tr>
<td>- Underwater - Level II survey</td>
<td>- Foundation data: available on-site or near-site soil borings shall be reviewed</td>
<td></td>
</tr>
<tr>
<td>- Soil Data - Available off- or near-site soil borings and geophysical data should be reviewed</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
Methods for determining acceptance (24.5).

- Inspections of existing facilities shall be performed in accordance with the requirements of Section 7.
- The assessment process described herein is applicable for areas outside the U.S., with the exception of the reduced criteria which are applicable for indicated U.S. areas only.

Meteocean parameters and Environmental Actions (24.7.2)

The Metocean data required for an assessment are the same as for a new structure. In some cases, a reduced return period may be considered for assessment, see Clause 24.5.

Deck Elevation and Environmental Actions (24.7.3)

If wave inundation of the deck is expected, resistance assessment shall be based on the ultimate strength analysis.

Fitness-for-purpose (24.8.1)

- A platform owner should take into account the higher risk of platform failure in extreme hurricanes, in comparison to new design, when using the revised Section 17 criteria.

Acceptance criteria may be developed for different exposure levels in terms of reduced actions to be applied in the assessment, e.g. corresponding to shorter return periods.

- Increased resistance criteria, e.g. reduced RSRs.

Existing structures shall meet the requirements of NORSOK N-001.

- Existing facilities where the primary structure does not meet the criteria for ULS or ALS related to environmental actions that can be forecast like wave and wind actions, may continue to be used if the following four requirements are fulfilled:
  - Shut-down and unmanned procedures are implemented. The procedure for shut down and unmanned should meet criteria given in 6.3.
  - Requirements to unmanned facilities according to NORSOK N-001 are satisfied - the environmental actions will not jeopardize any other main safety function (other than structural integrity) relevant for the facility during the storm, see 6.4
  - The risk of significant pollutant is found acceptable. see 6.5

- Assessment for Metocean Loading

- The Metocean data required for an assessment are the same as for a new structure.

- Assessment through the use of explicit probabilities of failure by comparison with similar platforms, a structure shall not be considered as fit for purpose by comparison to another structure.

- Assessment based on prior exposure, surviving actual exposure to an event that is known with confidence to have been as severe as, or more severe than, the event that would be considered in the actual ultimate strength analysis.

- Acceptance criteria may be developed for different exposure levels in terms of reduced actions to be applied in the assessment, e.g. corresponding to shorter return periods.

- The assessment process described herein is applicable for areas outside the U.S., with the exception of the reduced criteria which are applicable for indicated U.S. areas only.

- No DFF should be applied when checking the cyclic storm actions.

- Structures that are checked in ULS and ALS by use of linear analyses need normally not to be checked for cyclic failures during a storm.

- Further cyclic checks are usually not required in cases where the structural resistance is restricted to all of the following requirements:
  - no structural components will experience local or global buckling determined according to NORSOK N-004,
  - the cyclic check of the dimensioning storm should be made on loadings reduced by 10%.

- Local overstress and potential local damage are acceptable, but total collapse or excessive/damaging deformations shall be avoided.

Ultimate Strength Analysis

- Local overstress and potential local damage are acceptable, but total collapse or excessive/damaging deformations shall be avoided.

- The action and material factors according to NORSOK N-001 shall be used for structures that are assessed according to N-006.

- Resistance of damaged steel members can be calculated according to NORSOK N-004, Section 10.

- The same principles for check of ULS and ALS as given in NORSOK N-001, NORSOK N-003 and NORSOK N-004 apply to assessment of existing structures.

- Structures that are checked in ULS and ALS by use of linear analyses need normally not to be checked for cyclic failures during a storm.

- Further cyclic checks are usually not required in cases where the structural resistance is restricted to all of the following requirements:
  - no structural components will experience local or global buckling determined according to NORSOK N-004,
  - the cyclic check of the dimensioning storm should be made on loadings reduced by 10%.

- Local overstress and potential local damage are acceptable, but total collapse or excessive/damaging deformations shall be avoided.

- The action and material factors according to NORSOK N-001 shall be used for structures that are assessed according to N-006.

- Resistance of damaged steel members can be calculated according to NORSOK N-004, Section 10.

- The same principles for check of ULS and ALS as given in NORSOK N-001, NORSOK N-003 and NORSOK N-004 apply to assessment of existing structures.
**Seismic design consideration (24.7.4)**

The considerations are as given in Clause 11.

A two-level seismic design procedure shall be followed:

- **Ultimate limit state (ULS)** for strength and stiffness when subjected to an extreme level earthquake (ELE), from which it should sustain little or no damage.

- **Abnormal level earthquake (ALE)** to ensure that it meets reserve strength and energy dissipation requirements. The structure may sustain considerable damage from ALE, but structural failures causing loss of life and/or major environmental damage shall not be expected to occur.

Fatigue Limit State

- The results of a fatigue assessment in accordance with Clause 16 shows that the fatigue lives of all members and joints are at least equal to the total design service life, and the inspection history shows no fatigue cracks or unplanned damage.

- Fatigue assessment in accordance with Clause 16 has identified the joints with the lowest fatigue lives and periodic inspection of these joints finds no fatigue cracks or unplanned damage.

- Where fatigue lives of any members and joints are calculated to be less than the total design service life of the structure and fatigue damage has been identified, the structure may be assumed to be fit-for-purpose, provided conservative fracture mechanics predictions of fatigue crack growth demonstrate adequate future life and capacity.

**Assessment for Seismic Loading**

1. Assessment for seismic loading is not a requirement for seismic zones 0, 1 and 2.

2. Assessment for metocean loading should be performed for all seismic zones.

3. Perform assessment for ice loading, if applicable.

4. Design basis check - the platforms are acceptable to seismic loading if no significant new faults in the local area have been discovered, or any other information regarding site seismic hazard characterization has been developed that significantly increases the level of seismic loading used in the platform's original design.

5. Design level analysis - to be an operator's economic decision and not applicable for seismic assessment purposes.

6. Ultimate strength analysis - is required if the platform does not pass the design level check or screening. Level A-1 platforms withstand loads associated with a median 1000-yr return period earthquake without system collapse; Level A-3 platforms withstand loads associated with a median 500-yr return period earthquake without system collapse.

**Assessment for Ice Loading**

Follow API RP 2N for guidance on the selection of appropriate ice criteria and loading.

**Ice Conditions and Actions due to Ice (24.7.5)**

Guidance on ice conditions and actions due to ice is given in ISO 19901-1 for certain areas.

---

**Table A.9.9-2 — Values of partial action factor, γ_c and RSR to achieve target failure rate A ≤ 10^−5/yr for new unmanned installations (exposure level L2)**

<table>
<thead>
<tr>
<th>Environment</th>
<th>Partial action factor, γ_c</th>
<th>Mean RSR</th>
</tr>
</thead>
<tbody>
<tr>
<td>A2G</td>
<td>1.17</td>
<td>1.60</td>
</tr>
<tr>
<td>A2H</td>
<td>1.17</td>
<td>1.60</td>
</tr>
<tr>
<td>NR</td>
<td>1.29</td>
<td>1.49</td>
</tr>
</tbody>
</table>

It is emphasized that the results in Table A.9.9-2 relate to new, unmanned (evacuated) structures.

For existing structures, the criteria may be relaxed, provided the risk is kept as low as reasonably practicable.

**Fatigue Limit State**

- The results of a fatigue assessment in accordance with Clause 16 shows that the fatigue lives of all members and joints are at least equal to the total design service life, and the inspection history shows no fatigue cracks or unplanned damage.

- Fatigue assessment in accordance with Clause 16 has identified the joints with the lowest fatigue lives and periodic inspection of these joints finds no fatigue cracks or unplanned damage.

- Where fatigue lives of any members and joints are calculated to be less than the total design service life of the structure and fatigue damage has been identified, the structure may be assumed to be fit-for-purpose, provided conservative fracture mechanics predictions of fatigue crack growth demonstrate adequate future life and capacity.

---

**Assessment for Ice Loading**

- Methodology for low cycle fatigue of joints is given in 8.4.

---

2) The time interval for inspection shall be planned such that potential fatigue cracks can be detected with a large certainty before they grow so large that the integrity of the structure is endangered.

3) Components where a failure can lead to substantial consequences and have passed their fatigue design life shall be inspected by an appropriate NDT method.

These components shall have a maximum inspection interval of 5 years if calculated interval gives a longer period.

4) If there is less than 5 years of corrosion allowance for the components that have experienced significant corrosion, corrosion inspections are required at intervals not exceeding 2 years.

5) Risk based inspection may be recommended for planning of in-service inspection for fatigue cracks.

6) The acceptance criteria when planning in-service inspection for fatigue cracks based on RBI is depending on consequence of failure. The risk of a structural failure due to fatigue cracks should not be larger than risk of other failure modes.

- Methodology for low cycle fatigue of joints is given in A.4.
### Table 7-4 Damaged Members Formula Comparison

#### ISO 19902: 2007

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Formulation</th>
<th>Limits</th>
</tr>
</thead>
<tbody>
<tr>
<td>T</td>
<td>$T = \frac{F}{A_{st} f_c}$</td>
<td>$T_{LSD} = 1.05, t_c = \frac{f_c}{E}$</td>
</tr>
<tr>
<td>A</td>
<td>cross-sectional area of the undamaged section</td>
<td></td>
</tr>
<tr>
<td>$\sigma_{LSD}$</td>
<td>the partial resistance factor for axial tension strength for damaged members</td>
<td></td>
</tr>
</tbody>
</table>

#### NORSOK N-004

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Formulation</th>
<th>Limits</th>
</tr>
</thead>
<tbody>
<tr>
<td>$N_{LSD}$</td>
<td>$N_{LSD} = A_y f_c$</td>
<td>$N_{LSD} = N_{LSD,UND}$</td>
</tr>
<tr>
<td>$A_y$</td>
<td>design axial force</td>
<td></td>
</tr>
<tr>
<td>$f_c$</td>
<td>design axial tension capacity of the dented section</td>
<td></td>
</tr>
<tr>
<td>$A_{st}$</td>
<td>cross-sectional area of the undamaged section</td>
<td></td>
</tr>
</tbody>
</table>

#### Column Buckling

| $f_{cr}$ | $f_{cr} = \frac{A_y}{0.001 L (1 - 0.017 f_c L)^{1.2}}$ |

### Figure 7-1 Definition of axes for dented section

- $x$ = dented depth
- $w$ = wall thickness

---

**Note:** The table and formulae are excerpts from the document, providing a comparison between ISO 19902:2007 and NORSOK N-004 standards for damaged members. The document also includes a diagram illustrating the definition of axes for dented sections.
### ISO 19902: 2007

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Formulation</th>
</tr>
</thead>
<tbody>
<tr>
<td>13.7.2.3.2 Positive Bending</td>
<td>$\sigma_{pl,b} = \frac{M_{pl,b}}{I_{pl,b}} \leq \sigma_{pl,\text{allow}}$</td>
</tr>
<tr>
<td>$\sigma_{pl,b}$</td>
<td>the representative bending strength in stress units</td>
</tr>
<tr>
<td>$M_{pl,b}$</td>
<td>the positive bending moment due to forces from factored actions</td>
</tr>
<tr>
<td>$I_{pl,b}$</td>
<td>the partial resistance moment for bending strength</td>
</tr>
<tr>
<td>13.7.2.3.3 Negative Bending</td>
<td>$\sigma_{pl,n} = \frac{M_{pl,n}}{I_{pl,n}} \leq \sigma_{pl,\text{allow}}$</td>
</tr>
<tr>
<td>$\sigma_{pl,n}$</td>
<td>the negative bending strength</td>
</tr>
<tr>
<td>$M_{pl,n}$</td>
<td>the negative bending moment due to forces from factored actions</td>
</tr>
<tr>
<td>$I_{pl,n}$</td>
<td>the partial resistance moment for bending strength</td>
</tr>
<tr>
<td>13.7.2.4.4 Neutral Bending</td>
<td>$\sigma_{pl,n} = \frac{M_{pl,n}}{I_{pl,n}} \leq \sigma_{pl,\text{allow}}$</td>
</tr>
<tr>
<td>$\sigma_{pl,n}$</td>
<td>neutral bending stress</td>
</tr>
<tr>
<td>$M_{pl,n}$</td>
<td>neutral bending moment</td>
</tr>
<tr>
<td>$I_{pl,n}$</td>
<td>neutral bending section</td>
</tr>
</tbody>
</table>

### NORSOK N-004

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Formulation</th>
</tr>
</thead>
<tbody>
<tr>
<td>10.6.2.3 Bending</td>
<td>$M_{pl} \leq M_{pl,\text{allow}} \leq \sigma_{pl} M_{pl}$</td>
</tr>
<tr>
<td>$M_{pl}$</td>
<td>bending moment due to forces from factored actions</td>
</tr>
<tr>
<td>$M_{pl,\text{allow}}$</td>
<td>bending moment due to forces from factored actions</td>
</tr>
<tr>
<td>$\sigma_{pl}$</td>
<td>bending strength</td>
</tr>
</tbody>
</table>

13.7.3 Dented tubular members subjected to combined forces

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Formulation</th>
</tr>
</thead>
<tbody>
<tr>
<td>13.7.3.1.2 Axial Tension, Positive Bending and Neutral Bending</td>
<td>$\sigma_{pl,n} = \frac{N_{pl,n}}{A_{pl,n}} \leq \sigma_{pl,\text{allow}}$</td>
</tr>
<tr>
<td>$\sigma_{pl,n}$</td>
<td>the representative local buckling strength of the dented member</td>
</tr>
<tr>
<td>$A_{pl,n}$</td>
<td>area of the dented member</td>
</tr>
<tr>
<td>13.7.3.1.3 Axial Tension, Negative Bending and Neutral Bending</td>
<td>$\sigma_{pl,n} = \frac{N_{pl,n}}{A_{pl,n}} \leq \sigma_{pl,\text{allow}}$</td>
</tr>
<tr>
<td>$\sigma_{pl,n}$</td>
<td>the representative local buckling strength of the undamaged member</td>
</tr>
<tr>
<td>$A_{pl,n}$</td>
<td>area of the undamaged member</td>
</tr>
</tbody>
</table>

13.7.3.2 Axial Compression, positive bending and neutral bending

### Combined loading

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Formulation</th>
</tr>
</thead>
<tbody>
<tr>
<td>10.6.2.4 Combined loading</td>
<td>$N_{pl} \leq \left( \frac{N_{pl,b}}{N_{pl,b,\text{allow}}} \right) \leq 1$</td>
</tr>
<tr>
<td>$N_{pl}$</td>
<td>design axial force on the dented section</td>
</tr>
<tr>
<td>$N_{pl,b}$</td>
<td>design axial force on the undamaged section</td>
</tr>
<tr>
<td>$N_{pl,b,\text{allow}}$</td>
<td>effective length factor, as defined in Table 6-2</td>
</tr>
<tr>
<td>$\sigma_{pl,b}$</td>
<td>bending strength, as defined in Table 6-2</td>
</tr>
<tr>
<td>$\sigma_{pl,b,\text{allow}}$</td>
<td>bending moment about an axis parallel to the dent</td>
</tr>
<tr>
<td>$\sigma_{pl,b,\text{allow}}$</td>
<td>bending moment about an axis perpendicular to the dent</td>
</tr>
<tr>
<td>$\sigma_{pl,b,\text{allow}}$</td>
<td>moment of inertia of the dented cross-section, as defined in Table 6-2</td>
</tr>
<tr>
<td>$\sigma_{pl,b,\text{allow}}$</td>
<td>member out-of-straightness perpendicular to the dent</td>
</tr>
<tr>
<td>$\sigma_{pl,b,\text{allow}}$</td>
<td>moment reduction, as defined in Table 6-2</td>
</tr>
</tbody>
</table>
### ISO 19902: 2007

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Formulation</th>
<th>Limits</th>
</tr>
</thead>
<tbody>
<tr>
<td>A.13.8 Corroded tubular members</td>
<td>One approach to estimate the strength of an approximately uniformly corroded member is to assume a reduced thickness for the entire member. The reduced thickness should be consistent with the average material loss due to corrosion. The member with the reduced thickness can then be evaluated as an undamaged member. This reduced thickness approach is generally conservative. Another common case of corroded members is the presence of severe localized corrosion in the form of patches. This form of corrosion can not be approximated as uniform.</td>
<td></td>
</tr>
</tbody>
</table>

### NORSOK N-004

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Formulation</th>
<th>Limits</th>
</tr>
</thead>
<tbody>
<tr>
<td>10.6.3 Corroded members</td>
<td>In lieu of refined analyses, the strength of uniformly corroded members can be assessed by assuming a uniform thickness loss for the entire member. The reduced thickness should be consistent with the average material loss due to corrosion. The member with the reduced thickness can then be evaluated as an undamaged member. In lieu of refined analyses, the strength of members with severe with severe localized corrosion can be assessed by treating the corroded part of the cross-section as non-effective, and using the provisions given for dented tubulars, see 10.6.2.</td>
<td></td>
</tr>
</tbody>
</table>

\[ BD = 0.5(1 - \cos(A/2)) \]

\[ \beta = \text{equivalent dent depth} \]

\[ D = \text{tube diameter} \]

\[ A_{corr} = \text{corroded part of the cross-section} \]

\[ A = \text{full cross section area} \]

### 14.8 Damaged Joints

Joints in existing structures sometimes become damaged as a result of fatigue, corrosion, or overload (environmental or accidental). In such cases, the reduced joint strength shall be estimated either from simple models, e.g., based on the use of reduced area or section modulus, or else shall be based on more extensive numerical analysis using FEA models or experimental evidence.

### 10.7 Cracked members and joints

10.7.2 Partially cracked tubular members

Partially cracked members with the cracked area loaded in compression can be treated in a similar manner to the one described for dented tubulars.

\[ BD = 0.5(1 - \cos(A/2)) \]

where

\[ \beta = \text{equivalent dent depth} \]

\[ D = \text{tube diameter} \]

\[ A_{corr} = \text{crack area} \]

\[ A = \text{full cross section area} \]

Partially cracked members with the cracked area loaded in tension should be subject to a fracture mechanics assessment considering tearing mode of fracture and ductile crack growth.

For fatigue sensitive conditions, a fatigue evaluation of the cracked member should also be considered.

10.7.3 Tubular joints with cracks

The elastic strength of a cracked tubular joint can be calculated by reducing the joint resistances for the corresponding un-cracked geometry taken from 6.4.3, with an appropriate reduction factor accounting for the reduced ligament area. The reduced strength is given by:

\[ N_{max2} = F_{max} N_{max} \]

\[ M_{max2} = F_{max} M_{max} \]

where

\[ N_{max2} = \text{axial resistance of the cracked joint} \]

\[ M_{max2} = \text{bending resistance of the cracked joint} \]

\[ N_{max} = \text{the joint design axial resistance} \]

\[ M_{max} = \text{the joint design bending moment resistance} \]

\[ F_{max} = (1 - A/40\sqrt{2}) \]

\[ A = \text{cracked area of the brace/chord intersection} \]

\[ A_{0} = \text{full area of the brace/chord intersection} \]

\[ Q_{0} = \text{tubular joint geometry factor, given in 6.4.3.3} \]

\[ q_{0} = 0 \] for part-thickness cracks
### ISO 19902: 2007

<table>
<thead>
<tr>
<th>Stress/Parameter</th>
<th>Formulation</th>
</tr>
</thead>
<tbody>
<tr>
<td>13.9 Grouted tubular members</td>
<td></td>
</tr>
<tr>
<td>Subclause 13.9 applies to both fully grouted undamaged members and fully grouted dented tubular members, with the dent depth limited to either ( h = 0.020 ) or ( h \leq 1.0 ).</td>
<td></td>
</tr>
<tr>
<td>13.9.2.2 Axial tension</td>
<td></td>
</tr>
<tr>
<td>( \sigma_{ag} = \frac{1}{3} \sigma_{ug} ) where ( \sigma_{ag} ) is the axial stress in the steel tubular member due to forces from factored actions, neglecting the grout</td>
<td></td>
</tr>
<tr>
<td>( \sigma_{ug} ) is the representative axial tensile strength of the steel, ( f_{y} ),</td>
<td></td>
</tr>
<tr>
<td>( \sigma_{ug} = \frac{1}{3} \sigma_{ug} ) for ( \lambda_{ug} \leq 1.34 )</td>
<td></td>
</tr>
<tr>
<td>( \sigma_{ug} = \frac{1}{3} \sigma_{ug} ) for ( \lambda_{ug} &gt; 1.34 )</td>
<td></td>
</tr>
<tr>
<td>( \lambda_{ug} = \frac{\sigma_{ug}}{f_{y}} ) the column slenderness parameter of the grouted member</td>
<td></td>
</tr>
<tr>
<td>( \sigma_{ug} = \frac{1}{3} \sigma_{ug} ) for ( \lambda_{ug} \leq 1.34 )</td>
<td></td>
</tr>
<tr>
<td>( \lambda_{ug} = \frac{\sigma_{ug}}{f_{y}} ) for ( \lambda_{ug} &gt; 1.34 )</td>
<td></td>
</tr>
</tbody>
</table>

### NORSOK N-004

<table>
<thead>
<tr>
<th>Stress/Parameter</th>
<th>Formulation</th>
</tr>
</thead>
<tbody>
<tr>
<td>16.8 Repaired and strengthened members and joints</td>
<td></td>
</tr>
<tr>
<td>16.8.2 Grouted tubular members</td>
<td></td>
</tr>
<tr>
<td>16.8.2.1 Axial tension</td>
<td></td>
</tr>
<tr>
<td>( \sigma_{ag} = \frac{1}{3} \sigma_{ug} ) where ( \sigma_{ag} ) is the axial force on the grouted section</td>
<td></td>
</tr>
<tr>
<td>( \sigma_{ug} = \frac{1}{3} \sigma_{ug} ) for ( \lambda_{ug} \leq 1.34 )</td>
<td></td>
</tr>
<tr>
<td>( \lambda_{ug} = \frac{\sigma_{ug}}{f_{y}} ) the column slenderness parameter of the grouted member</td>
<td></td>
</tr>
<tr>
<td>( \sigma_{ag} = \frac{1}{3} \sigma_{ug} ) for ( \lambda_{ug} \leq 1.34 )</td>
<td></td>
</tr>
<tr>
<td>( \lambda_{ug} = \frac{\sigma_{ug}}{f_{y}} ) for ( \lambda_{ug} &gt; 1.34 )</td>
<td></td>
</tr>
</tbody>
</table>

\( f_{y} \) is the yield stress of the steel, \( f_{y} \).
13.9.2.4 Bending

\[ \kappa_0 = \frac{M_0}{W_0} \]

where

\[ \kappa_0 = \text{design bending moment for the grouted section} \]
\[ M_0 = \text{design bending moment in the grouted member} \]
\[ W_0 = \text{elastic section modulus of the transformed, composite section} \]

\[ M_0 = \frac{2EI}{d} \left(1 + 0.01k\right) \]
\[ I = \frac{2EI}{d} \left(1 + 0.01k\right) \]

\[ E = \text{elastic modulus of the material} \]
\[ I = \text{moment of inertia} \]

\[ k = \frac{1}{100} \]

13.9.3.2 Axial compression and bending

\[ T_{1} = \frac{4K_{0}K_{1}}{1 - K_{0}K_{1} - T_{2}} \]

\[ K_{1} = \frac{0.09d_{2}}{0.09d_{2}} \]

for \( \lambda_{0} \leq 1.34 \)

\[ 0.9d_{2} \]

for \( \lambda_{0} > 1.34 \)

\[ K_{0} = \frac{K_{0}K_{1}[1+30d_{2}/(2-1)]}{1-30d_{2}/(2-1)} \]

\[ \lambda_{0} \leq K_{0} \]

\[ K_{2} = \frac{K_{0}K_{1}(1+30d_{2}/(2-1))}{1-30d_{2}/(2-1)} \]

\[ \lambda_{0} > K_{0} \]

13.9.3.2 Combined axial compression and bending

\[ \frac{N_{0}M_{0}}{M_{0}N_{0}} + T_{1}M_{0}M_{0}N_{0} + T_{2}(M_{0}M_{0}N_{0})^{2} \leq 1 \]

\[ N_{0}M_{0}N_{0} = 1 \]

\[ N_{0}M_{0}N_{0} \leq K_{0} \]

\[ M_{0}M_{0}N_{0} \leq K_{0} \]

\[ T_{1} = \frac{4K_{0}K_{1}}{1 - K_{0}K_{1} - T_{2}} \]

\[ K_{1} = \frac{0.09d_{2}}{0.09d_{2}} \]

for \( \lambda_{0} \leq 1.34 \)

\[ 0.9d_{2} \]

for \( \lambda_{0} > 1.34 \)

\[ K_{0} = \frac{K_{0}K_{1}[1+30d_{2}/(2-1)]}{1-30d_{2}/(2-1)} \]

\[ \lambda_{0} \leq K_{0} \]

\[ K_{2} = \frac{K_{0}K_{1}(1+30d_{2}/(2-1))}{1-30d_{2}/(2-1)} \]

\[ \lambda_{0} > K_{0} \]
8 FIRE, BLAST, AND ACCIDENTAL LOADINGS

8.1 General

Side-by-side comparison of the API, ISO, and NORSOK requirements for fire, blast, and accidental loading is given in Table 8-2. The table presents the assessment process, ship collision criteria, dropped objects, fire and blast requirements as specified in the API RP 2A, ISO 19902, and NORSOK N-004.

Section 18 of API RP 2A and its commentary cover the design criteria of fire, blast and accidental loading. The probability of an event leading to a partial or total platform collapse occurring and consequence resulting from such an event varies with platform types. In API RP 2A, implementing preventive measures is considered as the most effective approach in minimizing the probability of occurrence of an event and the resultant consequences of the event. API RP 2A also states that consideration of preventive measures coupled with established infrastructure, open facilities and relatively benign environment have resulted in a good safety history and detailed structural assessment should therefore not be necessary for typical U.S. Gulf of Mexico-type structures and environment.

The design criteria under accidental situations are included in Clause 10 of ISO19902. In this standard, only designing for hazards for structures of exposure level L1 is qualified; specification of relevant design situations and criteria for exposure levels L2 and L3 is intended to be included in a future edition. ISO 19902 states that designers can choose between avoiding a hazard (e.g. by taking special preventive measures such as operational restrictions), minimizing the consequences of the considered hazard or designing for resistance of the hazard.

Design guidance against accidental actions is included in Annex A of NORSOK N-004. It states that the overall goal of the design against accidental actions is to achieve a system whose main functions of the installation are not impaired. The main functions include usability of escape-ways, integrity of shelter areas and global load bearing capacity.

8.2 Assessment Process

8.2.1 API RP 2A

API states that the assessment process is intended to be a series of evaluations of specific events that could occur for the selected platform over its intended service life and service functions.

Figure 8-1 is copied from API RP 2A Fig. 18.2-1. It charts the assessment process in the form of six main tasks and three risk levels utilizing the ALARP principle and assessing the consequences in a structured manner. The necessity of further study or analysis is based on the appropriate risk level for the selected platform with assigned exposure category and event with certain probability of occurrence. To determine the risk level (1, 2, or 3), a 3x3 risk matrix is defined using the platform
exposure categories L-1, L-2, and L-3 on one axis and the high, medium, and low probability of occurrence on the other axis.

Figure 8-1 Assessment Process (API RP 2A)
8.2.2 ISO 19902

In general, ISO Clause 10 defines a hazard as the potential for human injury, damage to the environment, damage to property or a combination of these. In this standard, the hazards are grouped into three main groups according to a probability of occurring or return period of being exceeded:

- **Group 1** – hazards with return periods of the order of 100 years
- **Group 2** - hazards with return periods of the order of 1000 to 10000 years
- **Group 3** - hazards with return periods well in excess of 10000 years

Designing for hazards of group 1 is normally treated by the regular design process. Other hazards belonging to group 1 and not treated by the regular design process along with hazards belonging to group 2 are specially addressed by ALS requirements. Hazards falling into group 3 are considered as residual accidentals and may normally be ignored for design.

As indicated in ISO 19900, the accidental situations are related to two types of hazards:

- Hazards associated with specially identified accidental events, such as vessel collisions, dropped objects and fires and explosions.
- Hazards associated with abnormal environmental actions including abnormal earthquake. Abnormal design situation may be based on a return period of 10000 years for an exposure level L1 platform.

When checking accidental limit states (ALS) for accidental or abnormal events, all partially action and resistance factors are to be taken as 1.0.

8.2.3 NORSOK

NORSOK N001, N-004 and N-006 state that the structure shall be checked for all ALSs for the design accidental actions defined in the risk analysis recommended in the standards. The material factor is taken as 1.0 in the ALS check.

According to NORSOK N-001, the structure is to be checked in two steps:

- **Step 1**: Resistance of the structure against design accidental actions – the structure is to maintain the prescribed load carrying capacity for the defined accidental loads
- **Step 2**: Post-accident resistance of the structure against environmental actions – If local damage occurred from step 1, the facility shall continue to resist defined environmental conditions without suffering extensive failure, free drifting, capsizing and sinking etc.
Typical accidental actions include ship collisions, dropped objects and fire and explosion. NORSOK N-004, Annex A gives the design recommendations for these actions.

### 8.3 Ship Collisions

All three codes provide similar impact energy calculation formula. The formula in API and ISO are the same. NORSOK gives three formulas for fixed installations including jacket structures, compliant installations including semi-submersibles, TLPs and production vessels, and articulated columns. Jack-ups may be classified as fixed or compliant structures depending on mode of operation.

In API, an 1100 short-ton (1,000 metric ton) vessel with impact velocity of 1.64 ft/s (0.5 m/s) is set as minimum collision requirement for application in the GOM. No guidance is provided for other areas.

ISO 19902 recommends the following minimum impacting ship displacement requirements for different geographic locations:

- Northern North Sea: 8000 metric tons
- Southern North Sea: 2500 metric tons
- GOM: 1000 metric ton (55m to 60 m)

The impact velocity is given for two energy levels in ISO:

- **a)** Low energy impact: 0.5 m/s; representing a minor accidental “bump” during normal maneuvering of the vessel
- **b)** High energy impact: 2 m/s; representing a vessel drifting out-of-control in a sea state with significant wave height of 4 m.

In API and ISO, the added mass is introduced as an added mass factor (1.4 for broadside collision, 1.1 for bow/stern collision). ISO indicates that these added mass coefficients are typical for large (5000 t displacement) supply vessels and a slightly higher value, e.g. 1.6 should be applied for a typical 2500 t supply vessel. Accordingly, it seems that the added mass factor in API for a 1000 metric ton vessel should probably be increased. For small supply vessels, the impact energy calculated using ISO added mass factor is larger compared to that predicted using API added mass coefficients.

NORSOK N-003 states that for collision energy the mass of the supply ship should normally not be less than 5000 tons and the speed not less than 0.5 m/s and 2 m/s for ULS and ALS design checks, respectively. This recommendation is consistent with ISO requirements.
All three codes require that the platform survives the initial collision and that the residual strength requirements are complied with.

API requires that the platform survives the initial impact and retain sufficient residual strength after impact to withstand the one-year environmental storm loads in addition to normal operating conditions.

ISO states that impact energy level a) (defined above) represents a serviceability limit state and that the owner can set his own requirements based on practical and economic considerations; and level b) represents an ultimate limit state in which the structure is damaged but progressive collapse shall not occur.

NORSOK requires two steps of ALS check: remain intact with the damage imposed by the ship collisions and meet residual strength requirements under undamaged condition.

In NORSOK, force-deformation relationships for a large column impact, tubular and beam type, are provided for supply vessels with displacement of 5000 tons which is commonly used in the North Sea. The detailed resistance for different types of members is also given. Compared to API and ISO, the designer may find more guidance in NORSOK to determine appropriate boat impact forces.

### 8.4 Dropped Objects

API recommends that the safe handling practice and preventive operational procedures can reduce the risk of dropped objects. The platform should survive the initial impact and meet the post-impact criteria as defined for vessel impact.

ISO suggest that a rigorous impact analysis be evaluated depending on the consequences with regard to the integrity of the structure. Indirect means should be incorporated into design, such as, avoiding weak elements in the structure (particularly at joints), selecting materials with sufficient toughness, and endurance and ensuring that critical components are not placed in vulnerable locations. No guidance is provided for design check methodology.

Compared to API and ISO, NORSOK gives considerably more guidance (see Table 8-2) for evaluating the effect of dropped objects. Energy considerations for the dropped objects combined with simple elastic-plastic methods are given in NORSOK. It is noted that dropped objects are rarely critical to the global integrity of the installation and will mostly cause local damage. The major threat to global integrity is probably puncturing of buoyance tanks, which could impair the hydrodynamic stability of a floating installation.
8.5 Fire and Blast

Commentaries 18.7, 18.8 and 18.9 of API RP 2A provide the design guidelines for fire, blast and interaction between fire and blast. Both fire and blast assessment need to demonstrate that the escape routes and safe areas will survive.

Three methods are given in API RP 2A:

1. Zone method: it is based on the assumption that a member utilization ratio calculated using basic allowable stress will remain unchanged for the fire load condition if the allowable stress is increased to yield, but the yield stress itself is subject to a reduction factor of 0.6

2. Linear elastic method (for example, a working stress code): a maximum allowable temperature in a steel member is assigned based on the stress level in the member prior to the fire and the member utilization ratio remains below 1.00

3. Elastic-plastic method (for example, a progressive collapse analysis): a maximum allowable temperature in a steel member is assigned based on the stress level in the member prior to the fire and the member utilization ratio may go above 1.00

API summarizes the factors influencing the magnitude of the loading generated by a blast as follows:

- the type and volume of hydrocarbon released
- the amount congestion in a module
- the amount of confinement,
- the amount of venting available
- the amount of module congestion caused by equipment blockage
- mitigation efforts such as water spray

A blast can cause two types of loading: overpressure and drag loading; Overpressure is likely to govern the design of structures such as blast walls and floor/roof systems. API states that the blast overpressures in a platform can vary from zero on a small, open platform to more than 2 bars (1 bar = 14.7 psi) in an enclosed or congested installation. Drag loading is caused by blast-generated wind.

Critical piping, equipment, and other items exposed to the blast wind should be designed to resist the drag loading. Static or dynamic analysis can be performed based on the duration of the blast loading relative to the natural period of the structure.
The following acceptable criteria are given in API:

1. Strength limit: API RP 2A is a working stress design. The allowable stresses can be increased so that the safety factor is 1.0.

2. Deformation limit: the API recommendations are given in Table 8-1.

<table>
<thead>
<tr>
<th>Type of Loading</th>
<th>Strain Limit</th>
</tr>
</thead>
<tbody>
<tr>
<td>Tension</td>
<td>5%</td>
</tr>
<tr>
<td>Bending or compression</td>
<td></td>
</tr>
<tr>
<td>Plastic sections</td>
<td>5%</td>
</tr>
<tr>
<td>Compact sections</td>
<td>3%</td>
</tr>
<tr>
<td>Semi-compact sections</td>
<td>1%</td>
</tr>
<tr>
<td>Other sections</td>
<td>&lt; yield strain</td>
</tr>
</tbody>
</table>

The determination of the yield point is essential to blast analysis. API states that actual yield stress should be used in the analysis and strain rates and strain hardening effects should be included in determining the yield stress and general material behavior.

API 2A suggests that fire and blast assessments should be performed together and the effects of one on the other are carefully analyzed.

The API RP 2FB 1st Ed. /8/ published in 2006 contains significantly more comprehensive treatment of the fire and blast design than previous included in API RP 2A. The document covers the required risk analyses and design methodologies against fire and blast on GOM offshore structures. As an example, the recommended structural fire and blast assessment procedures are depicted in Figure 8-2.

NORSOK N-004 refers to Norwegian Standard NS-ENV 1993-1 for fire load effect assessment. NORSOK states that the response to explosion loads may either be determined by non-linear dynamic finite element analysis or by simple calculation models based on SDOF analogies and elastic-plastic methods of analysis. Details for both methods are given in Annex A.6 of N-004 issued in 2004 prior to publication of the API RP 2FB.
8.6 Summary of Accidental Loadings Comparison

For design against fire and blast, API RP 2A charts the assessment process in the form of six main tasks and three risk levels utilizing the ALARP principle and assessing the consequences in a structured manner. In ISO 19902 hazards are grouped into three main groups according to a probability of occurrence or return period of being exceeded. NORSOK N001, N-004 and N-006 state that the structure shall be checked for all ALSs for the design accidental actions defined in the risk analysis recommended in the standards. With regards to ship collision ALS design, NORSOK gives the most comprehensive guidance of the three codes.
Table 8-2 Fire, Blast, and Accidental Loading Comparison

<table>
<thead>
<tr>
<th>API</th>
<th>AISC</th>
<th>Det norske Veritas</th>
<th>DNV</th>
<th>Det norske Veritas</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Assessment Procedure</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1. Identify the structures that could be at risk.</td>
<td>Not required</td>
<td>Structured assessment</td>
<td>Not required</td>
<td>Structured assessment</td>
</tr>
<tr>
<td>2. Evaluate the structural performance of the structures considered to be at risk.</td>
<td>Not required</td>
<td>Structured assessment</td>
<td>Not required</td>
<td>Structured assessment</td>
</tr>
<tr>
<td><strong>Assessment Process</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1. Identify the sources of risk.</td>
<td>Required</td>
<td>Source assessment</td>
<td>Required</td>
<td>Source assessment</td>
</tr>
<tr>
<td>2. Evaluate the likelihood of occurrence of each source of risk.</td>
<td>Required</td>
<td>Source assessment</td>
<td>Required</td>
<td>Source assessment</td>
</tr>
<tr>
<td>3. Evaluate the consequence of each source of risk.</td>
<td>Required</td>
<td>Source assessment</td>
<td>Required</td>
<td>Source assessment</td>
</tr>
<tr>
<td><strong>Design Considerations</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1. The platform should have the initial collision and the post-impact actions.</td>
<td>Required</td>
<td>Design consideration</td>
<td>Required</td>
<td>Design consideration</td>
</tr>
<tr>
<td>2. All exposed elements at risk in the collision zone of an installation should be assessed for accidental vessel impact during normal operations.</td>
<td>Required</td>
<td>Design consideration</td>
<td>Required</td>
<td>Design consideration</td>
</tr>
<tr>
<td>3. The vertical height of the collision zone should be determined from the considerations of vessel draft, operational water depth, and flood elevation.</td>
<td>Required</td>
<td>Design consideration</td>
<td>Required</td>
<td>Design consideration</td>
</tr>
</tbody>
</table>

### Fire, Blast, and Accidental Loading

| **Vehicle Impact** | | | | |
| --- | --- | --- | --- |
| A vehicle impact shall be addressed for the structures with exposure levels 1 and 2. | | | |
| Two energy levels shall be considered: | | | |
| 1. Low energy level, represented by the type of vessel that would not cause damage to a vessel in the platform area, | | | |
| 2. High energy level, representing a rare condition, and based on the type of vessel that would cause significant damage to the platform area. | | | |

| **Energy Absorption** | | | | |
| --- | --- | --- | --- |
| The energy of a vehicle can be expressed from the following: | | | |
| 1. Localised plastic deformation of the hull | | | |
| 2. Plastic deformation of the members | | | |
| 3. Impact energy absorption and survival of platform | | | |

| **Primary Substructure** | | | | |
| --- | --- | --- | --- |
| The primary substructure should be designed to absorb the energy during impact and to control the consequences of the impact. | | | |

For platforms in mild environments and reasonably close to their basic load of the following: the substructure should be limited, unless other criteria can be demonstrated.

### Vehicle Impact

| **Vehicle Impact** | | | | |
| --- | --- | --- | --- |
| The platform should have sufficient structural strength after impact to withstand the one-year environmental storm load in addition to normal operating loads. | | | |
| Special attention should be given to the representation of actual actions of damaged members or joints in the post-impact assessment. | | | |

### Energy Absorption

| **Energy Absorption** | | | | |
| --- | --- | --- | --- |
| The energy of a vehicle can be expressed from the following: | | | |
| 1. Localised plastic deformation of the hull | | | |
| 2. Plastic deformation of the members | | | |
| 3. Impact energy absorption and survival of platform | | | |

For platforms in mild environments and reasonably close to their basic load of the following: the substructure should be limited, unless other criteria can be demonstrated.
When evaluating the impact risk from dropped objects, the nature of all crane operations in the platform vicinity must be taken into account. Depending on the consequences for the structural integrity of the structure, the need for a rigorous impact analysis shall be determined.

- Theulleness or location of dropped objects should be incorporated into the design by indirect means such as:
  - Adopting weak elements in the structure (particularly at joints)
  - Selecting materials with sufficient toughness
  - Ensuring that critical components are not placed in vulnerable locations

- Dropped objects are highly critical to the global integrity of the installation and will most likely cause local damages. The major threat to global integrity is probably puncturing of buoyancy tanks, which could impair the hydrostatic stability of floating structures.

- The structural effects may be determined by non-linear dynamic finite element analysis or by energy considerations combined with simple yield limit or elastic methods.

### Table 4.5.1

<table>
<thead>
<tr>
<th>Object</th>
<th>Weight [kN]</th>
<th>Terminal velocity [m/s]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Derrick</td>
<td>30</td>
<td>23 to 28</td>
</tr>
<tr>
<td>Crane</td>
<td>220</td>
<td></td>
</tr>
<tr>
<td>Lift pump</td>
<td>900</td>
<td></td>
</tr>
<tr>
<td>Mosquito net</td>
<td>80</td>
<td>18</td>
</tr>
<tr>
<td>Total</td>
<td>1110</td>
<td>7</td>
</tr>
</tbody>
</table>

### Deflection-Energy dissipator

1. **Ipsilateral impact**

   The energy dissipated in an object subjected to deflected impact is given by:

   \[ E = \frac{1}{2} m v^2 (1 + \frac{c^2}{2}) \]

   where:
   - \( E \) = energy dissipated by impact
   - \( m \) = mass of impact object
   - \( v \) = impact velocity
   - \( c \) = constant

2. **Limit for energy dissipation**

   a. **Pipe or pipe support**

   The maximum stress for plugging of pipes due to deflected impact may be taken as:

   \[ \sigma_n = \frac{1}{2} \left( \frac{d + e + f}{2} \right) \]

   where:
   - \( d \) = diameter of pipe
   - \( e \) = thickness of pipe
   - \( f \) = distance from point of impact to the pipe's boundary

   b. **Blunt objects**

   For stability of cross-sections and tensile fracture, see A.3.10
The assessment process identified that a significant risk of blast exists. The blast should be considered as a load condition. The structural assessment must demonstrate that the escape routes and safe areas are maintained to allow sufficient time for platform evacuation and emergency response procedures to be implemented.

The fire and blast analyses should be performed together and the effects of one on the other studied. The fire as a load condition requires that the following be defined:

- Fire scenario: type, location, geometry, and intensity.
- Heat flow characteristics from the fire to structures and protected steel members - to determine the temperature of the member as a function of time.
- The amount of radiant heat arriving at the surface of a member is determined using a geometrical configuration or 'fire' factor. For engulfed members, a configuration factor of 1.0 is used.
- Properties of steel at elevated temperatures and where applicable - thermal properties - required for the calculation of the steel temperature.
- Measured properties - used to verify that original design still meets the strength and servitude requirements.
- Properties of the protection systems active and passive.
- The load is assumed to occur at a designated temperature and time.
- The design period of protection is based on the fire expected duration and the required evacuation period.

Design for Fire

- Zone method:
- The zone method of design assigns a maximum allowable stress that can develop in a steel member without reference to the stress level prior to the fire.
- The assumption of the method is that a member's ultimate ratio calculated using the allowable stress will remain unchanged for the fire load condition. If the allowable stress is increased to yield, the yield stress itself is subject to a reduction factor of 0.5. This reduction factor is defined such that the yield strength reduction factor is matched by the reduction in Young's modulus (by a factor of 0.2).
- Yield strength and Young's modulus, the governing design condition may be affected, thus, the zone method may not be applicable.
- Linear elastic method (e.g., a working stress code).
- A maximum allowable temperature in a steel member is assigned based on the stress level at the member to the fire, such that as the temperature increases, the member utilization (UR) remains below 1.0 (the member continues to behave elastically).
- With an ultimate reduction in both yield strength and Young's modulus, the governing design condition may be affected, thus, the linear elastic method may not be applicable.
- Elastic-plastic method (e.g., a progressive collapse analysis):
- A maximum allowable temperature in a steel member is assigned based on the stress level at the member to the fire. As the temperature increases, the member utilization (UR) may go above 1.0 (the member behavior is elastic-plastic).
- A nonlinear analysis to be performed to verify that the structure will not collapse and still meet the survivability criteria.

Notes:
1. Regardless of the design method, the limitation of the nonlinear stress-strain relationship of steel at elevated temperatures can be achieved by the selection of a representative value of strain.
2. A value of 0.2% is commonly used and has the benefit of giving a modified reduction in yield strength and Young's modulus, but has the disadvantage of limiting the allowable temperature of the steel to 400°C.
Due to the complexity in predicting blast loads, the pressure-time curves should be generated by an expert in this field.

A blast can cause two types of loading:

1. Overpressure - results from increases in pressure due to expanding combustion products
2. Drag loading - caused by blast-generated wind

Critical piping, equipment, and other items exposed to the blast wind should be designed to withstand the predicted drag loads.

Environmental loads can be neglected in a blast analysis.

Structural Resistance

1. Strength Limit
2. Deformation Limit

If the structural response exceeds the critical operational limits, appropriate measures must be taken to ensure the integrity and maintain the structural response to subsequent loads.

4. Structural Design

Blast load levels for different types of loading are as follows:

<table>
<thead>
<tr>
<th>Type of Loading</th>
<th>Stress Levels</th>
</tr>
</thead>
<tbody>
<tr>
<td>Impact load</td>
<td>75%</td>
</tr>
<tr>
<td>Blast wave</td>
<td>50%</td>
</tr>
<tr>
<td>Explosion</td>
<td>25%</td>
</tr>
</tbody>
</table>

The strain limits are determined for different load cases and the effect on the structural response is evaluated. This process allows the engineer to design and construct safe structures.

The industry associations have produced their own more detailed guidance applicable to particular types of operation and circumstances.

API, which can be used for Gulf of Mexico type platforms,

- UL501/509, which are suited to larger platforms operated in a safety case regime.
- NORSOK, which contains wound analytical requirements.

ISO 13702 contains requirements and recommendations for fires and explosions.

The response to explosion loads can be determined by non-linear dynamic finite element models.
9 INSTALLATION AND TEMPORARY CONDITIONS

9.1 General

Side-by-side comparison of the API, ISO, and NORSOK requirements for installation and temporary conditions is given in Table 9-2. The specific document and location within is given in the table header row.

API 2A WSD states that basic allowable stresses for member design may be increased by 1/3 for installation forces including environmental effects during transportation and launch. The details of the requirements for temporary conditions are given in Sections 2.4 and 12 of the API RP 2A.

Clauses 8 and 22 of ISO 19902 provide the LSD methods for temporary condition design.

NORSOK N-004 Clause K.4.4.6 states that transportation and installation design and operation shall comply with the requirements given in NORSOK J-003. It is noted that NORSOK J-003 (1997) requirements have been completely incorporated in the more recent ISO 19901-6 “Marine Operations” issued in 2009. Therefore, the comparison made here is actually a comparison between API and ISO.

9.2 Lifting

9.2.1 Dynamic Effects

API gives the dynamic load effects for the following conditions:

1. At open, exposed sea: padeyes and other internal members including both connections framing into the joint where the padeye is attached and transmitting lifting forces within the structure should be designed for a minimum load factor of 2.0 applied to calculated static loads. All other structural members transmitting lifting forces should be designed using a minimum load factor of 1.35.

2. For other marine situations, the selection of load factors should meet the expected local conditions but should not be less than a minimum 1.5 and 1.15 for the two conditions as listed above.

3. For land-based lifting, dynamic load factors are not required.

Dynamic amplification factors are given in ISO 19902 Clause 8 and more details are included in ISO 19901-6 Clause 18, see Table 9-1. The maximum DAF in ISO is 1.3 compared to API’s 1.35. Also ISO DAF is >1.0 on land when moving elements are involved whereas API allows no DAF (i.e., DAF=1.0). Also the ISO reduces the DAF with the increase in the weight lifted which is a logical process not yet adopted by API RP 2A.
For onshore lifts, where the crane can move horizontally, the “moving” column in Table 9-1 shall apply. ISO 19901-6 also states that the DAF values in Table 9-1 shall be multiplied by a further factor of 1.1 for offshore lifts by cranes on two or more similar vessels.

Compared to API, ISO recommends DAF that includes the crane number effects (rigging factors) and local factor except lifting conditions.

**9.2.2 Effect of Fabrication Tolerance**

The dynamic load factors are affected by fabrication tolerance and sling length tolerance which are addressed in both API and ISO.

API requires that the fabrication tolerances do not exceed the requirements of Section 11.5.1 of API RP 2A and the variation in length of slings does not exceed ±0.25% of nominal sling length, or 1.5 inches. The total variation from the longest to the shortest sling should not be greater than 0.5% of the sling length or 3 inches. If the tolerances exceed these limits, a detailed analysis including these tolerances should be performed.

ISO’s requirements are intended to apply to the situations where fabrication misalignments are consistent with Annex G of ISO 19902 and where the variance on the length of slings does not exceed the greater of 0.25% of the nominal sling length or 40 mm, which is close to API requirements.

**9.2.3 Allowable Stresses and Action Factors**

API does not allow the increase of allowable stresses in lifting design due to short-term loads. It requires that all critical structural connections and primary members should be designed to have adequate reserve strength to ensure structural integrity during lifting.

In API, the lifting eyes and the connections to the supporting structural members should be designed for a horizontal force of 5% of the static sling load, applied simultaneously.

---

**Table 9-1 DAF for a single crane on a vessel**

<table>
<thead>
<tr>
<th>Mass of lifted object (tonnes)</th>
<th>Gross weight, W (kN)</th>
<th>k_DAF in air</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>offshore</td>
</tr>
<tr>
<td>≤ 100</td>
<td>W ≤ 1 000</td>
<td>1.30</td>
</tr>
<tr>
<td>from 100 to 1 000</td>
<td>1 000 &lt; W ≤ 10 000</td>
<td>1.20</td>
</tr>
<tr>
<td>from 1 000 to 2 500</td>
<td>10 000 &lt; W ≤ 25 000</td>
<td>1.15</td>
</tr>
<tr>
<td>≥ 2 500</td>
<td>25 000 &lt; W</td>
<td>1.10</td>
</tr>
</tbody>
</table>

Notes:

- This column is included to facilitate comparison with weight reporting.
- Lifts by land-based cranes involved with marine operations such as loadouts.
with the static sling load. This horizontal force should be applied perpendicular to the padeye at the center of the pinhole. This is not required by ISO.

In ISO, member and joint strengths should be checked using one of the following formulae (Equation 9.1 and 9.2):

\[ F_d = k_{DAF} \gamma_{f,dl} \gamma_{f,lf} \frac{G_T}{T} \left( \gamma_{f,sun} S_{sun} \right) \]  
\[ S = k_{DAF} \gamma_{f,dl} \gamma_{f,lf} \gamma_{f,sun} S_{sun} \]  
\[ (9.1) \]
\[ (9.2) \]

\( F_d \) = design action

\( S \) = internal force

\( k_{DAF} \) = dynamic amplification factor; 1.10 for heavy lift by semi-submersible crane vessel for in air offshore lifts or in air onshore or in sheltered waters; 1.30 in other cases for offshore in air.

\( \gamma_{f,dl} \) = the rigging factor, 1.10 for a dual lift; 1.00 for single crane

\( \gamma_{f,lf} \) = local factor, for lifting attachments, spreader beams, and internal members attached to lifting point: 1.25 (for a lift in open waters), 1.15 (for a lift on shore or in shelter waters); 1.00 for other structures;

\( \gamma_{f,sun} \) = partial factor, 1.30

\( G_T \) = the action imposed either by the weight of the structure in air, or by the submerged weight of the structure in water

\( Q_T \) = the action imposed by the weight of the temporary equipment or other objects, including any rigging installed or carried by the structure

\( T \) = the lifting actions and hydrostatic pressure on the structure

### 9.2.4 Slings, Shackles and Fittings

Both API and ISO require that slings should have a total resistance factor of 4.0 on the manufacturer’s rated minimum breaking strength of the cable compared to the calculated sling force. The total resistance factor may be reduced to a minimum of 3.0 for carefully controlled conditions.

ISO and API also have the same requirements for shackles and fittings. Shackles and fittings should be selected so that the manufacturer’s rated working load is greater than or equal to the calculated sling force, provided the manufacturer’s specifications include a minimum resistance factor of 3.0 on minimum breaking strength.
In addition, ISO recommends that the slings should be assumed to carry the lift point force in a 45:55% split of the lift point force between the two slings, where two slings are connected to one padeye, or where a split of the lift point force between the two slings. API doesn’t require it.

9.3 Loadout

API gives short descriptions of two scenarios of loadout: direct lift and horizontal movement onto barge. If the lifting arrangement by a direct lift is different with that to be used in the offshore installation, the lifting forces should be evaluated. Since the lifting in open sea will impose more severe conditions, it is sufficient to check the latter case. During the horizontal movement onto barge, impact need not be considered since the movement is normally slow.

ISO gives the same recommendations to direct lift and horizontal movement onto barge. In addition, it also gives guidelines to self-floating structures. Actions should be evaluated for the full travel of the structure down the ways. ISO clearly states that the guideline for self-floating structures does not apply to self-floating structures built in dry dock and floated by flooding the dock.

9.4 Transportation

The basic guidelines in API and ISO are the same, including environmental criteria, determination of forces and special considerations (slamming, VIV, fatigue etc.). These guidelines are summarized in Table 9-2.

Compared to API, ISO suggest that the environmental conditions used to determine the tow motions should be established by the owner. It also gives the following guidelines:

- For long ocean tows where the structure and barge are unmanned, the extreme environmental conditions are typically selected to have a probability of exceedance during the tow duration in the range of 1% to 10%. The specific value will depend on an evaluation of acceptable risks and consequences.

- For short duration tows, the environmental conditions should generally have a return period of not less than 1 year for the season in which the tow takes place.

9.5 Launching and Uprighting Forces

ISO requires that a structure shall not be launched from the barge if the significant wave height exceeds 2.0 m or if it is expected to exceed 2.0 m before sufficient on-bottom stability is achieved. The rest of guidance in both API and ISO is identical.
9.6 On-bottom Stability

On-bottom stability requirements are given in Section 12.4.5 of API RP 2A and Clause 8.7.6 of ISO 19902. The on-bottom stability check is to ensure that the structure will remain at planned elevation, location and attitude until the piles can be installed. Both codes require that the mudmats or footings have adequate capacity against sliding and bearing failure and structural members supporting these have adequate strength to avoid being damaged.

ISO only provides general considerations on on-bottom stability check. No detailed design requirements are given. In contrast to ISO, the following detailed requirements are given in API:

- The factors of safety against bearing capacity failure recommended are 2.0 for on bottom gravity loads alone and 1.5 for including the design environmental condition applicable for the installation period.
- At the operator’s discretion, with supporting analyses, an alternative of limiting penetration criteria may be used.
- Allowable steel stresses may be increased by one-third when wave loading is included.

9.7 Summary of Installation and Temporary Conditions Comparison

The details of the requirements for temporary conditions in API RP 2A are given in Sections 2.4 and 12. Clauses 8 and 22 of ISO 19902 provide the LSD methods for temporary condition design. NORSOK N-004 Clause K.4.4.6 states that transportation and installation design and operation shall comply with the requirements given in NORSOK J-003. It is noted that NORSOK J-003 (1997) requirements have been completely incorporated in the more recent ISO 19901-6 “Marine Operations” issued in 2009. Therefore, the comparison made here is actually a comparison between API and ISO and it demonstrated that they are similar with different level of guidance and some minor quantitative differences.
### Table 9-2 Comparison of Installation and Temporary Conditions

<table>
<thead>
<tr>
<th>API Section 15</th>
<th>ISO 19902 Clause 8 and 9</th>
<th>NORSOK M-601 Clause 8 (ISO 19902)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>- NORSOK M-601 Clause 8 of installation analysis states that transport and installation design and operation shall comply with the requirements given in NORSOK-M-601. NORSOK-M-601 states that transportation and installation shall comply with the requirements given in ISO 19902.</td>
</tr>
</tbody>
</table>

#### Design Efforts (8.3.3)

- **Dynamic Efforts**:
  - For offshore structures, as in Chapter 8 of API 15A, dynamic effort is accounted for by the effective force of the vessel taking-up the load and for movement of the deck or the lift structure, shall be derived from the following:
    - For offshore vessels as in Chapter 8 of API 15A:
      - The effective force shall be the sum of the forces due to:
        - The static lift-off force.
        - The inertial forces due to the acceleration of the vessel and the structure.
    - For other marine conditions, as in Chapter 8 of API 15A, all other marine conditions may be accounted for as in static lift-off.

#### Initial Forces due to Inertial Forces (8.2.4.2)

- The initial forces due to inertial forces shall be calculated by the following expression:
  - \[ F = \frac{Q}{C} \times F_t \]
  - Where:
    - \( F \) = initial force (kN)
    - \( Q \) = design load (kN)
    - \( C \) = design factor (1.2)
    - \( F_t \) = time factor (0.7) for the time period considered.

#### Table 9.1 - Partial action factors for calculating inertial forces

<table>
<thead>
<tr>
<th>Design Load Factor</th>
<th>Partial Inertial Factor</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>1</td>
</tr>
<tr>
<td>1.2</td>
<td>1.2</td>
</tr>
<tr>
<td>1.5</td>
<td>1.5</td>
</tr>
<tr>
<td>2</td>
<td>2</td>
</tr>
</tbody>
</table>

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**Revision No.: 1**

**Date: 2012-01-12**

**Page 109**
1. Analysis with one pair of opposite strings assumed to carry 70% and the other pair of 30% of the load, and vice versa for equal inclination angles of 10%.

2. For long spans where the structure and cable are stabilized, the following conditions are typically assumed to have a probability of exceedance during the period of 10% to 15%.

- The specified values are dependent on an evaluation of acceptable risks and regulations.
- For short time spans, the environmental conditions should generally have a higher risk.
## Cross-Sectional Analysis

1. **Structural Analysis:**
   - Stability and safety of the jacket are verified using finite element analysis.
   - The jacket is designed to withstand the anticipated loads and environmental conditions.
   - The design criteria for the jacket are in accordance with API and ISO standards.

2. **Environmental Conditions:**
   - The jacket is designed to withstand the anticipated loads and environmental conditions.
   - The design criteria for the jacket are in accordance with API and ISO standards.

3. **Material Selection:**
   - The jacket is designed to withstand the anticipated loads and environmental conditions.
   - The design criteria for the jacket are in accordance with API and ISO standards.

## Conclusion

The jacket design satisfies all the required specifications and can be constructed under the given conditions. The jacket is designed to be safe, stable, and capable of withstanding the anticipated loads and environmental conditions.

---

**NORSOK M-004, K-064**

- The jacket foundation system shall be designed to withstand the anticipated loads and environmental conditions.
- The jacket foundation system shall be designed to withstand the anticipated loads and environmental conditions.

---

**ISO 19901-1, Clause 3.3.2 and Clause 17.6**

- The jacket foundation system shall be designed to withstand the anticipated loads and environmental conditions.
- The jacket foundation system shall be designed to withstand the anticipated loads and environmental conditions.
10 SEISMIC DESIGN GUIDELINES

10.1 General

The requirements in API, ISO, and NORSOK relating to seismic design criteria are compared in Table 10-12 located at end of this section. Both Clause 11 of ISO 19902 (Fixed steel offshore structures) and ISO 19901-2 (Seismic design procedures and criteria) give the guidelines with regard to seismic design and analyses of offshore platforms.

The earthquake design guidelines are included in Section 2.3.6 of API RP 2A. NORSOK seismic design guidelines are briefly given in Clause 6.5 of NORSOK N-003 and Annex K.4.4.5 of NORSOK N-004. Seismicity is not normally a design issue in the North Sea. Therefore, Seismic analysis comparisons are herein mainly focused on ISO and API requirements.

10.2 Design Guidelines Comparison

Summary of API and ISO design guidelines comparison are included in Table 10-12 and discussed below.

10.2.1 Terminology

The terms SLE (Strength Level Earthquake) and DLE (Ductility Level Earthquake) as used in API have been denoted ELE (Extreme Level Earthquake) and ALE (Abnormal Level Earthquake) in ISO.

10.2.2 Seismic Risk Maps

The API RP 2A seismic risk map (Figure C2.3.6-1 in API) provides the effective ground acceleration for seismic active zones in the offshore US. It is intended to be used for SLE design with 200-year return period earthquake and can be used for preliminary design or feasibility studies.

ISO has provided the generic 5% damped spectral accelerations for bedrock outcrop for a 1.0s oscillator period and for a 0.2s oscillator period respectively for worldwide seismic active offshore locations. These accelerations have average return period of 1000 years.

10.2.3 Seismic Zones

ISO provides five seismic site zones as presented in Table 10-1.

<table>
<thead>
<tr>
<th>Seismic zone</th>
<th>Sa, map (1.0)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>&lt; 0.03g</td>
</tr>
<tr>
<td></td>
<td>0</td>
</tr>
</tbody>
</table>
$S_{a,\text{map}}(1.0)$ is the $1.0s$ horizontal accelerations

Based on it, the site seismic zones can be determined from worldwide seismic maps.

Six seismic zones are defined in API as shown in Table 10-2 below. The table is based on 200 year return period earthquake.

<table>
<thead>
<tr>
<th>Table 10-2 Seismic Zone In API</th>
</tr>
</thead>
<tbody>
<tr>
<td>Z</td>
</tr>
<tr>
<td>G</td>
</tr>
</tbody>
</table>

Where $Z =$ Zone or relative seismicity factor given in Figure C2.3.6-1.

\[ G = \text{Ratio of effective horizontal ground acceleration to gravitational acceleration} \]

### 10.2.4 Foundation Soil Types

In ISO, the site soil classifications have been expanded to include A/B, C, D, E and F in contrast to the soil types of A, B and C used in API. The details are included in Table 10-3.

<table>
<thead>
<tr>
<th>Table 10-3 Site Class</th>
</tr>
</thead>
<tbody>
<tr>
<td>Site class (ISO)</td>
</tr>
<tr>
<td>A/B</td>
</tr>
<tr>
<td>C</td>
</tr>
<tr>
<td>D</td>
</tr>
<tr>
<td>E</td>
</tr>
<tr>
<td>F</td>
</tr>
</tbody>
</table>
10.2.5 Earthquake Response Spectrum

The API RP 2A response spectrum is defined as follows:

\[ \begin{align*}
T < 0.05 \text{ s} & \quad S_a/G = 1.0 \\
0.05 \text{ sec} < T < 0.125 \text{ s} & \quad S_a/G = 20T \\
\text{API soil type A:} & \\
0.125 \text{ sec} < T < 0.32 \text{ s} & \quad S_a/G = 2.5 \\
T > 0.32 \text{ s} & \quad S_a/G = 0.8/T \\
\text{API soil type B:} & \\
0.125 \text{ sec} < T < 0.48 \text{ s} & \quad S_a/G = 2.5 \\
T > 0.48 \text{ s} & \quad S_a/G = 1.2/T \\
\text{API soil type C:} & \\
0.125 \text{ sec} < T < 0.72 \text{ s} & \quad S_a/G = 2.5 \\
T > 0.72 \text{ s} & \quad S_a/G = 1.8/T \\
\end{align*} \]

where \( G = \) effective horizontal ground acceleration

The response spectrum defined in ISO 19901-2 is:

\[ \begin{align*}
S_{a,\text{site}}(T) &= (3T+0.4)(C_a)S_{a,\text{map}}(0.2) \quad \text{for } T \leq 0.2\text{s} \\
S_{a,\text{site}}(T) &= C_vS_{a,\text{map}}(1.0)/T \quad \text{for } T > 0.2\text{s} \\
\text{except that} & \\
S_{a,\text{site}}(T) &\leq C_aS_{a,\text{map}}(0.2) \\
S_{a,\text{site}}(T) &= 4C_vS_{a,\text{map}}(1.0)/T^2 \quad \text{for } T > 4\text{s} \\
\end{align*} \]

Where

\( T = \) natural period of a simple, single degree of freedom oscillator

\( C_a, C_v = \) site coefficients

\( S_{a,\text{site}}(T) = \) site spectral acceleration corresponding to a return period of 1000 years and a single degree of freedom oscillator period \( T \)

\( S_{a,\text{map}}(0.2) = \) 1000 year rock outcrop spectral acceleration obtained from maps in Annex 2 of ISO 19901-2 associated with a single degree of freedom oscillator period 0.2 s

\( S_{a,\text{map}}(1.0) = \) 1000 year rock outcrop spectral acceleration obtained from maps in Annex 2 of ISO 19901-2 associated with a single degree of freedom oscillator period 1.0 s

10.2.6 Earthquake Directional Loads

Both API and ISO suggest that design spectrum should be applied equally (1:1) in both horizontal directions and one-half of that applied in the vertical direction simultaneously, when the response spectrum method is used.

NORSOK also suggest the two horizontal directions and one vertical direction combination. One of the horizontal excitations should be parallel to a main structural axis, with the major
component directed to obtain the maximum value for the response quantity considered. The orthogonal horizontal component may be set equal to 2/3 of the major component and the vertical equal to 2/3 of the major component.

10.2.7 Earthquake Directional Combination

The square root of the sum of the squares (SRSS) is recommended to be used for combining the directional responses in both API and ISO.

ISO also states that the three directional responses may be combined linearly assuming that one component is at its maximum while the other two components are at 40% of their respective maximum values.

10.2.8 Time History Analysis

When a non-linear time history analysis is used, ISO requires that global structural survival shall be demonstrated in half or more of the time history analyses if seven or more time-history records are used. If fewer than seven time-history analyses are used, global survival shall be demonstrated in at least four time-history analyses.

API requires that at least three sets of representative earthquake ground motion time histories should be analyzed.

NORSOK suggests that the load effect should be calculated for at least three sets of time histories.

10.2.9 Structural Components - Tubular D/t Ratio

API suggests that the slenderness ratio (kl/r) of the primary diagonal bracing in vertical frames is limited to 80 and their ratio of diameter to thickness (D/t) is limited to 1900/F_y (F_y is the yield strength in ksi) or 13100/F_y (F_y in MPa).

In ISO, the slenderness ratio (kl/r) of primary bracing in vertical frames shall be limited to no more than 80 and F_y D/E.t ≤ 0.069 or 13800/F_y (F_y in MPa).

10.2.10 Pile Axial Capacity Requirements

API RP 2A requires a safety factor of pile penetration of 1.50 under the extreme condition and 2.0 under the operating condition.

ISO requires a partial resistance factor for pile axial capacity of 1.25 for extreme condition and a partial resistance factor for the p-y curves of 1.0 is used to determine the lateral pile performance. The partial resistance factors for axial capacity and lateral pile performance under ALE conditions shall be 1.0.
10.3 Seismic Design Comparison

10.3.1 Two Level check

The structure is designed for two levels of earthquakes in API and ISO requirements:

- **Strength Level Earthquake (Extreme Level Earthquake):** 100 – 200 year return period; Structural stress should not exceed yield. Under SLE (ELE), structure should sustain little or no damage.

- **Ductility Level Earthquake (Abnormal Level Earthquake):** 1000-5000 year return. Structural stress may exceed yield but should not collapse.

In NORSOK, earthquake design includes ULS (Ultimate Limit State) check of components based on earthquakes with an annual probability of occurrence of $10^{-2}$ and appropriate action and material factors; as well as an ALS (Abnormal Limit State) check of the overall structure to prevent its collapse during earthquakes with an annual probability of exceedance of $10^{-4}$ with appropriate action and material factors.

10.3.2 Action Combinations

**ELE Requirements**

API states that earthquake loading should be combined with other simultaneous loadings such as gravity, buoyancy and hydrostatic pressure. Gravity loading should include the platform dead weight, actual live loads and 75% of the maximum supply and storage loads. In computing the dynamic characteristics of braced, pile supported steel structures, uniform modal damping ratio of 5% critical should be used. API also states that the basic AISC allowable stresses and those presented in Section 3.2 (Allowable Stresses for Cylindrical Members) may be increased by 70% for strength requirement.

ISO requires that the all members, joints and pile components shall be checked for strength for using internal force resulting from the design action calculated by the following equations:

\[
F_d = 1.1G_1 + 1.1G_2 + 1.1Q_1 + 0.9E \quad (10.1a)
\]

or

\[
F_d = 0.9G_1 + 0.9G_2 + 0.8Q_1 + 0.9E \quad (10.1b)
\]

Where

- $E$ = the inertia action induced by ELE ground motion, which depends on the exposure level and the expected intensity of seismic events
- $G_1$ = self-weight of the structure with associated equipment and other objects
- $G_2$ = self-weight of equipment and other objects that remain constant for long periods of time, but can change during a mode of operation
- $Q_1$ = the weight of consumable supplies and fluids in pipe, tanks and storage, etc.

A modal damping ratio of up to 5% of critical is the same as the requirement in API. The inertia action ($E$) induced by ELE (SLE) ground motion can be determined by dynamic analysis procedures such as response spectrum analysis or time history analysis.
NORSOK N-001 states that earthquake shall be handled as environmental action within the limit state design for ULS and ALS. It can be interpreted into the following equations for ELE:

ULS (a): \(1.3G + 1.3Q + 0.7E\)
ULS (b): \(1.0G + 1.0Q + 1.3E\)
ALS (Abnormal effect): \(1.0G + 1.0Q + 1.0E\)

Where: \(G\) = permanent actions
\(Q\) = Variable actions
\(E\) = Earthquake action

A modal damping ratio of up to 5% of critical is the same as the requirement in API and ISO.

**ALE Requirements**

NORSOK also gives the guideline for action combination for ALS:

ALS (Abnormal effect): \(1.0G + 1.0Q + 1.0E\)

**10.3.3 Seismic Design Procedures**

API gives the basic guidelines for seismic analysis, but there is not straightforward procedure can be followed. Compared to API, ISO gives the detailed procedures which are easy for the designers to follow. The summary of the procedures in ISO is included below.

Two alternative procedures for seismic design are provided in ISO, one is “simplified method” and another is “detailed method”. A simplified method may be used where seismic considerations are unlikely to govern the design of a structure, while the detailed method shall be used where seismic considerations have a significant impact on the design. The selection of the appropriate procedure depends on the exposure level of the structure and the expected intensity and characteristics of seismic events. Simple method allows using the generic seismic maps provided in ISO, while the detailed procedure requires a site-specific seismic study. Figure 10-1 presents a flowchart of the selection procedures and the steps associated with both procedures, which are given in ISO 19901-2. ISO also summarizes the seismic design requirements in Table 10-4.
Figure 10-1 Seismic design procedures in ISO 19901-2 (Figure 1of ISO 19901-2)
The design requirements in Table 10-4 is based on the SRC determination given below.

**SRC Determination**

The complexity of a seismic action evaluation and the associated design procedure depends on the structure’s seismic risk category (SRC). ISO recommends that the following steps shall be followed to determine the SRC.

1. Determine the site seismic zone from the worldwide seismic maps in ISO, see Table 10-1.
2. Determine the structure’s exposure level. The simplified seismic action procedure has been given in Table 10-5.

<table>
<thead>
<tr>
<th>Exposure Level</th>
<th>p_f</th>
</tr>
</thead>
<tbody>
<tr>
<td>L1</td>
<td>4 x 10^{-4} = 1/2500</td>
</tr>
<tr>
<td>L2</td>
<td>1 x 10^{-3} = 1/1000</td>
</tr>
<tr>
<td>L3</td>
<td>2.5 x 10^{-3} = 1/400</td>
</tr>
</tbody>
</table>

3. Determine the structure’s seismic risk category, SRC, based on the exposure level and the site seismic zone the SCR is determined from Table 10-6.
Table 10-6 Seismic risk category, SRC

<table>
<thead>
<tr>
<th>Site seismic zone</th>
<th>Exposure level</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>L3</td>
</tr>
<tr>
<td>0</td>
<td>SRC1</td>
</tr>
<tr>
<td>1</td>
<td>SRC2</td>
</tr>
<tr>
<td>2</td>
<td>SRC2</td>
</tr>
<tr>
<td>3</td>
<td>SRC2</td>
</tr>
<tr>
<td>4</td>
<td>SRC3</td>
</tr>
</tbody>
</table>

For platforms classified as SRC1, no seismic design or analysis is required.
For platforms classified as SRC2, the simplified method can be used for seismic design and analysis. ISO maps or regional maps can be used for evaluation of seismic activity.
For platforms classified as SRC3, either simplified or detailed method can be used for seismic design and analysis. Site specific, ISO maps or regional maps can be used for the evaluation of seismic activity.
For platforms classified as SRC4, the detailed method shall be used for seismic design and analysis. A site-specific study shall be performed for evaluation of seismic activity. Only platforms classified as SRC4, non-linear ALE analysis is required.

Simplified Method
The simplified method includes the following steps:

1) Soil classification and spectral shape
   a) Determine site soil classification (Table 10-3)
   b) Determine site coefficients (C_a, C_v)

C_a and C_v depend on the site class and either the mapped 0.2 sec. or 0.1 sec spectral accelerations for shallow foundations, see Table 10-7 and Table 10-8.

Table 10-7 C_a for shallow foundations and 0.2 s period spectral acceleration (ISO 19901-2 Table 6)
Table 10-8 \( C_v \) for shallow foundations and 0.2 s period spectral acceleration (ISO 19901-2 Table 7)

<table>
<thead>
<tr>
<th>Site class</th>
<th>( S_{a,neg}(1,0) ) 0.1 g</th>
<th>( S_{a,neg}(1,0) ) 0.2 g</th>
<th>( S_{a,neg}(1,0) ) 0.3 g</th>
<th>( S_{a,neg}(1,0) ) 0.4 g</th>
<th>( S_{a,neg}(1,0) ) 0.5 g</th>
</tr>
</thead>
<tbody>
<tr>
<td>A/B</td>
<td>1.0</td>
<td>1.0</td>
<td>1.0</td>
<td>1.0</td>
<td>1.0</td>
</tr>
<tr>
<td>C</td>
<td>1.7</td>
<td>1.6</td>
<td>1.5</td>
<td>1.4</td>
<td>1.3</td>
</tr>
<tr>
<td>D</td>
<td>2.4</td>
<td>2.0</td>
<td>1.8</td>
<td>1.6</td>
<td>1.6</td>
</tr>
<tr>
<td>E</td>
<td>3.5</td>
<td>3.2</td>
<td>2.8</td>
<td>2.4</td>
<td>2.4</td>
</tr>
<tr>
<td>F</td>
<td>*</td>
<td>*</td>
<td>*</td>
<td>*</td>
<td>*</td>
</tr>
</tbody>
</table>

For deep foundations, the coefficients \( C_a \) and \( C_v \) depend on site class only, see Table 10-9 below.

Table 10-9 Values of \( C_a \) and \( C_v \) for deep pile foundation

<table>
<thead>
<tr>
<th>Site class</th>
<th>( C_a )</th>
<th>( C_v )</th>
</tr>
</thead>
<tbody>
<tr>
<td>A/B</td>
<td>1.0</td>
<td>0.8</td>
</tr>
<tr>
<td>C</td>
<td>1.0</td>
<td>1.0</td>
</tr>
<tr>
<td>D</td>
<td>1.0</td>
<td>1.2</td>
</tr>
<tr>
<td>E</td>
<td>1.0</td>
<td>1.6</td>
</tr>
<tr>
<td>F</td>
<td>*</td>
<td>*</td>
</tr>
</tbody>
</table>

2) Seismic action procedure
The ALE horizontal and vertical spectral accelerations are obtained from the site 1000-year spectral acceleration multiplied by a scale factor of NALE (Table 10-10), which depends on the structure exposure level.

\[
S_{a,ALE}(T) = \text{NALE} \times S_{a,site}(T) \quad (10.2)
\]
Table 10-10 Scale factors for ALE spectra

<table>
<thead>
<tr>
<th>Exposure Level</th>
<th>ALE scale factor</th>
</tr>
</thead>
<tbody>
<tr>
<td>L3</td>
<td>0.85</td>
</tr>
<tr>
<td>L2</td>
<td>1.15</td>
</tr>
<tr>
<td>L1</td>
<td>1.60</td>
</tr>
</tbody>
</table>

The ELE horizontal and vertical spectral acceleration at oscillator period $T$:

$$S_{a,ELE}(T) = S_{a,ALE}(T)/C_r$$  (10.3)

$C_r$ is platform reserve capacity factor, which is dependent on the platform ductility.

To avoid return periods for the ELE that are too short, $C_r$ values shall not exceed 2.8 for L1 structures; 2.4 for L2 structures; and 2.0 for L3 structures.

**Detailed Method**

Detailed method is required for the platforms categorized as SRC 3 and 4.

1) Site-specific Study

This study is normally performed by specialists using probabilistic seismic hazard analysis (PSHA) and/or with deterministic seismic hazard analysis (DSHA) as a complement to PSHA. As a result of PSHA, a set of “hazard curves” will be generated in terms of probability of exceedance versus ground motion or response of single degree of freedom oscillator. Each curve represents a spectral response to a specific natural period of the oscillator.

2) Seismic action procedure

This procedure is based on PSHA results. The following steps shall be followed to define the ALE and ELE spectral accelerations:

a) Plot the site-specific hazard curve for $T = T_{dom}$ on a log$_{10}$-log$_{10}$ basis

b) Choose the target annual probability of failure, $P_f$ (Table 10-5 Target annual probability of failure, $p_f$), and determine the site-specific spectral acceleration at $P_f$, $S_{a,pf}(T_{dom})$.

c) Determine the slope of the seismic hazard curve ($\alpha_R$) in the region close to $P_f$ by drawing a tangent line to the seismic hazard curve at $P_f$. The slope $\alpha_R$ is defined as ratio of the spectral accelerations corresponding to two probability values, at the neighbourhood of $P_f$. One is larger than $P_f$ and another is less than $P_f$.

d) The correction factor $C_c$ is used to capture the uncertainties not reflected in the seismic hazard curve.
Table 10-11 Correction factor $C_c$

<table>
<thead>
<tr>
<th>$\alpha_R$</th>
<th>1.75</th>
<th>2.0</th>
<th>2.5</th>
<th>3.0</th>
<th>3.5</th>
</tr>
</thead>
<tbody>
<tr>
<td>$C_c$</td>
<td>1.20</td>
<td>1.15</td>
<td>1.12</td>
<td>1.10</td>
<td>1.10</td>
</tr>
</tbody>
</table>

e) Determine the ALE spectral acceleration by applying the correction factor $C_c$ to $S_{a,pf}(T_{dom})$

$$S_{a,ALE}(T_{dom}) = C_c S_{a,pf}(T_{dom})$$ (10.4)

The annual probability of exceedance ($P_{ALE}$) for ALE event can be directly read from the seismic hazard curve.

$$T_{return} = 1/P_{ALE} \text{ (in years)}$$

f) Once the ALE spectral acceleration $S_{a,pf}(T_{dom})$ is determined, the ELE spectral acceleration can be obtained.

$$S_{a,ELE}(T_{dom}) = S_{a,ALE}(T_{dom})/C_r$$ (10.5)

The annual probability of exceedance ($P_{ELE}$) for ELE event can be directly read from the seismic hazard curve.

$$T_{return} = 1/P_{ELE} \text{ (in years)}$$ (10.6)

10.3.4 Seismic Analysis Methods

Several analysis methods are discussed in these design codes and summarized as follows:

- Linear methods
  - Response spectrum analysis
  - Time history method (modal analysis method, or direct time integration numerical analysis method)

- Non-linear methods
  - Static pushover or extreme displacement method
    This method is mentioned in both API and ISO. Only ISO gives the procedure to be followed.

In ISO, the objective of the static pushover analysis is to verify that the seismic reserve capacity factor, $C_r$, of the structure is greater than that initially estimated for design. $C_r$ is defined as:

$$C_r = C_{sr} C_{dr}$$ (10.7)

Where $C_{sr} = \Delta u/\Delta ELE$, is a factor corresponding to the strengthening region of the action-deformation.

$C_{dr}$ is a factor corresponding to the degrading region of the action-deformation curve. It is measure of energy dissipation capacity of the structure beyond the ultimate seismic action and the corresponding deformation.
\[ C_{dr} = \sqrt{1 + \frac{A_d}{F_u \Delta_u}} \]  \hspace{1cm} (10.8)

Where \( A_d \) is the area under the action-deformation curve starting from \( \Delta_u \) and ending with \( \Delta_{CAP} \), the deformation capacity of the structure. \( \Delta_{CAP} \) corresponds to 60\% of \( F_u \).

ii) Non-linear time history analysis method
Its objective is to demonstrate that the structure can expected to sustain the ALE seismic event without collapse and without major topsides failure.

The linear methods can be used for ELE (or SLE) design and analysis, while the non-linear methods can be used for ALE (or DLE) design and analysis. Response spectrum analysis method is a relatively simple and cost effective method.

### 10.4 Summary of Seismic Design Guideline Comparison

Seismicity is not normally a design issue in the North Sea. Therefore, Seismic analysis comparisons are herein mainly focused on ISO and API requirements. The structure is designed for two levels of earthquakes in API and ISO requirements:

a) Strength Level Earthquake (Extreme Level Earthquake): 100 – 200 year return period; Structural stress should not exceed yield. Under SLE (ELE), structure should sustain little or no damage

b) Ductility Level Earthquake (Abnormal Level Earthquake): 1000-5000 year return. Structural stress may exceed yield but should not collapse. Compared to API, ISO 19901-2 gives the clear requirements for ductility level earthquake analysis and it is easier for the designers to follow.
Table 10-12 Seismic Criteria Comparison

<table>
<thead>
<tr>
<th>Earthquake Design</th>
<th>SLE (Strength Level Earthquake)</th>
<th>ELE (Extreme Level Earthquake)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>DLE (Ductility Level Earthquake)</td>
<td>ALE (Abnormal Level Earthquake)</td>
</tr>
<tr>
<td>Elastic Risk Map</td>
<td>Figure C2.3.6.1 - Seismic Risk of United States Continental Shelves</td>
<td>Worldwide Seismic Maps (Appendix A)</td>
</tr>
<tr>
<td></td>
<td>Sa max (1.0)</td>
<td>Sa max (1.0)</td>
</tr>
<tr>
<td></td>
<td>0.25g</td>
<td>0.25g</td>
</tr>
<tr>
<td></td>
<td>1.0g</td>
<td>1.0g</td>
</tr>
<tr>
<td></td>
<td>2.0g</td>
<td>2.0g</td>
</tr>
<tr>
<td></td>
<td>3.0g</td>
<td>3.0g</td>
</tr>
<tr>
<td></td>
<td>4.0g</td>
<td>4.0g</td>
</tr>
<tr>
<td></td>
<td>6.0g</td>
<td>6.0g</td>
</tr>
<tr>
<td>Based on 50-year earthquake</td>
<td>Source (1.0) horizontal acceleration corresponding to 1000-year earthquake</td>
<td></td>
</tr>
</tbody>
</table>

Foundation Soil

<table>
<thead>
<tr>
<th>Soil Class</th>
<th>Soil Profile</th>
<th>Soil shear wave velocity, g/sec</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>Rock, crystalline, conglomerate, or shale-like material</td>
<td>&gt; 3000 g/sec (914 m/sec)</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>B</td>
<td>Shallow strong alluvium - component sands, silts and stiff clays with shear strengths in excess of about 1500 psf (70 kPa), limited to depths of less than about 200 ft (61m), and overlying rock-like materials</td>
<td></td>
</tr>
<tr>
<td>C</td>
<td>Shallow strong alluvium - component sands, silts and stiff clays with shear strengths in excess of about 2000 ft (61m), and overlying rock-like materials</td>
<td></td>
</tr>
</tbody>
</table>

Soil class | Soil profile name | Soil shear wave velocity, g/sec |
<table>
<thead>
<tr>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>Hard rock/Rock</td>
<td>u_s &gt; 750</td>
</tr>
<tr>
<td>A</td>
<td>Hard rock/fine sediments &lt; 6 m</td>
<td></td>
</tr>
<tr>
<td>C</td>
<td>Very dense hard soil and soft rock</td>
<td>356 &lt; v_s &lt; 750</td>
</tr>
<tr>
<td>D</td>
<td>Soft to very stiff soil</td>
<td>180 &lt; v_s &lt; 350</td>
</tr>
<tr>
<td>E</td>
<td>Soft to firm soil</td>
<td>120 &lt; v_s &lt; 180</td>
</tr>
</tbody>
</table>

Earthquake Loads (Actions)

<table>
<thead>
<tr>
<th>Earthquake Directional Loads (Actions)</th>
<th>1.0:1.0 (two horizontal orthogonal (dr.) and 0.5 vertical, added simultaneously)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Earthquake Directional Combinations</td>
<td>out of the sum of the squares method(BRSS) for 1 component and 100% and 40% of its maximum values in other 2 components combined separately</td>
</tr>
</tbody>
</table>

Time History Analysis

<table>
<thead>
<tr>
<th>Minimum 3 sets of time history records</th>
<th>Minimum 4 sets of time history records</th>
</tr>
</thead>
<tbody>
<tr>
<td>Response Spectrum Shape</td>
<td>T=4.0 seconds, S(t): proportional to 1/T</td>
</tr>
<tr>
<td>Structural Stiffness (SLE or ALE)</td>
<td>T=4.0 seconds, S(t): proportional to 1/T</td>
</tr>
<tr>
<td>Transient Orifice (SLE or ALE)</td>
<td>T=4.0 seconds, S(t): proportional to 1/T</td>
</tr>
<tr>
<td>File-Soil Performance for ELE</td>
<td>Partial resistance factor - 1.25 (axial)</td>
</tr>
<tr>
<td>File-Soil Performance for ALE</td>
<td>Partial resistance factor - 1.00 (axial)</td>
</tr>
<tr>
<td>File Axial Capacity Requirements (General)</td>
<td>Partial resistance factor - 1.25 (extreme conditions)</td>
</tr>
<tr>
<td>API, LFD (axial) (extreme conditions)</td>
<td>Partial resistance factor - 1.25 (extreme conditions)</td>
</tr>
<tr>
<td>API, LFD (axial) (operating conditions)</td>
<td>Partial resistance factor - 1.00 (operating conditions)</td>
</tr>
<tr>
<td>API, WGD (axial) (operating conditions)</td>
<td>Partial resistance factor - 1.00 (operating conditions)</td>
</tr>
<tr>
<td>Factor of Safety - 1.50 (extreme conditions)</td>
<td>Partial resistance factor - 1.50 (operating conditions)</td>
</tr>
<tr>
<td>Factor of Safety - 2.00 (operating conditions)</td>
<td>Partial resistance factor - 2.00 (operating conditions)</td>
</tr>
</tbody>
</table>
### API RP 2A

**Probabilistic Purpose**

1. The seismic ULS design event is the extreme-level earthquake (ELE). The structure shall be designed such that an ELE event will cause little or no damage. The ULS requirements are intended to ensure that no significant structural damage occurs for a level of earthquake ground motion with an adequately low likelihood of being exceeded during the design service life of the structure.

2. The ALE (annual-level earthquake) requirements are intended to ensure that the structural and foundation have sufficient reserve strength, displacement and/or energy dissipation capacity to maintain the overall structural integrity and avoid structural collapse.

**Probabilistic Purpose**

1. The analysis model should include the three-dimensional contribution of platform stiffness and mass.

2. Earthquake loading should be combined with other simultaneous loading mechanisms such as gravity, buoyancy, and hydrostatic pressure.

3. Gravity loading should include the platform dead weight (computed of the weight of the structure, equipment, appurtenances), fixed live loads and 75% of the supply and storage tanks.

4. In computing the dynamic characteristics of braced, pile supported steel structures, uniform modal damping ratios of 5% critical should be used for an elastic analysis.

### ISO 19901-2

**Probabilistic Purpose**

1. **Design Action**

$$ F_k = 1.00 + 1.10q + 1.10q' + 0.16q'' $$

where $E$ = the inertia action induced by the ELE ground-motions and determined using dynamic analysis procedures such as response spectrum or time-history analysis; $G_1$, and $G_2$, permanent actions; $q$, variable action, and all actions that are likely to be present during an earthquake.

When contributions to the action effects due to weight oppose the inertia actions due to the earthquake:

- $F_k = 0.95 + 0.95q + 0.8q'$
- $75\%$ of the variable actions $q$
- the mass of waterlogged mud, and the added mass

- A modal damping ratio of $5\%$ of critical may be used in the dynamic analysis of the ELE event.

### NORSOK N-003 & N-004

**Probabilistic Purpose**

1. **U.S.**

The number of vibration modes in the analysis should represent at least $90\%$ of the total response energy of all modes.

2. **U.S.**

In absence of more accurate information, a modal damping ratio of $5\%$ of critical may be used.

3. Earthquake shall be handled as environmental action within the limit state design.

4. ULS (a): $1.00 + 0.77q$

5. ULS (b): $1.00 + 1.83q$

**Response Analysis**

1. Earthquake loads shall be described by two orthogonal horizontal motions and the vertical motion simultaneously.

2. One of the horizontal excitations should be parallel to a main structural axis, with the major component directed to obtain the maximum value for the response quantity considered. Unless more accurate calculations are performed, the orthogonal horizontal component may be set equal to $20\%$ of the major component and the vertical component equal to $25\%$ of the major component, referred to load deck.

3. Time history method - the load effect shall be calculated for at least three sets of time histories. The mean values of the calculated action effects from the time history analyses may be taken as basis for design.

**Material Factor** $M_F = 1.15$
<table>
<thead>
<tr>
<th>API RP 2A</th>
<th>ISO 19901-2</th>
<th>NORSOK N-003 &amp; N-004</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Liability section 2.1.5.6 (D)</strong></td>
<td><strong>Structural models shall include possible softness and strength degradation of elements under cyclic action reversals.</strong></td>
<td>1. The number of vibration modes in the analysis should represent at least 90% of the total response energy of all modes.</td>
</tr>
<tr>
<td>1. The intensity ratio of the rare, intense earthquake ground motions to strengthen earthquake ground motions is 2:1 or less. Systems are packet type structures with 8 or more legs.</td>
<td>2. The ALS analysis shall be based on best estimate values of modeling parameters such as material strength, soil strength and soil stiffness.</td>
<td>2. In the absence of more accurate information, a modal damping ratio of 5% of critical may be used.</td>
</tr>
<tr>
<td><strong>Design Requirements</strong></td>
<td><strong>A modal damping ratio of 5% of critical may be used in the dynamic analysis of the ALE event.</strong></td>
<td>3. Earthquake shall be handled as environmental action within the limit state design for ALS (abnormal effect).</td>
</tr>
<tr>
<td>1. Jackleg piles, including any enclosed piles, are designed to meet the requirements of 2.3.6.4, using twice the strength level seismic loads.</td>
<td>2. The following two methods of analysis are allowed for the ALE design check: a) the static pushover or extreme displacement method - to be used to determine possible and controlling global mechanisms of failure, or the global displacement of the structure (beyond the ELE) b) the non-linear time history analysis method - performing a displacement controlled structural analysis.</td>
<td>ALS (d): 1.0G + 1.0G + 1.0E</td>
</tr>
<tr>
<td>2. Diagonal bracing in the vertical frames are configured such that shear forces between horizontal frames and/or vertical runs between legs are distributed approximately equally to both tension and compression diagonal braces, and that &quot;K&quot; bracing is not used where the ability of a panel to transmit shear is lost.</td>
<td>3. A minimum of 4 sets of time history records shall be used to capture the randomness in seismic motions. If 2 or more time history records are used, global structure survival shall be demonstrated in half or more of the time history analyses. If fewer than 7 time history records are used, global survival shall be demonstrated in at least 4 time history analyses.</td>
<td>Structural System</td>
</tr>
<tr>
<td>3. The compression brace base limiters where these conditions are not met, including areas such as the portal frame between the jackleg and the deck, the structural components should be designed to meet the requirements of Section 2.3.6.4 using twice the strength level seismic loads.</td>
<td>4. The slenderness ratio (k) of primary diagonal bracing in vertical frames is limited to 50 and their ratio of diameter to thickness is limited to 130(1000 / Fy for kis (130(1000 / Fy for kis). All non-tubular members at connections in vertical frames are designed as compact sections in accordance with the AISC Specifications or designed to meet the requirements of 2.3.6.4 using twice the strength level seismic loads.</td>
<td>1. Earthquake motion can be described by two orthogonal horizontal motions and the vertical motion action simultaneously.</td>
</tr>
<tr>
<td>4. Horizontal members are provided at both adjacent legs at horizontal framing levels in vertical frames and that these members have sufficient compression capacity to support the redistribution of loads resulting from the buckling of adjacent diagonal braces.</td>
<td>5. The slenderness ratio (k) of primary diagonal bracing in vertical frames is limited to 50 and their ratio of diameter to thickness is limited to 130(1000 / Fy for kis = 130(1000 / Fy for kis). All non-tubular members at connections in vertical frames are designed as compact sections in accordance with the AISC Specifications or designed to meet the requirements of 2.3.6.4 using twice the strength level seismic loads.</td>
<td>2. One of the horizontal excitations should be parallel to a main structural axis, with the major component directed to obtain the maximum value for the response quantity computed. Unless more accurate calculations are performed, the orthogonal horizontal component may be set equal to 2/3 of the major component and vertical component equal to 1/3 of the major component, referred to the deck.</td>
</tr>
<tr>
<td>5. The slenderness ratio (k) of primary diagonal bracing in vertical frames is limited to 50 and their ratio of diameter to thickness is limited to 130(1000 / Fy for kis = 130(1000 / Fy for kis). All non-tubular members at connections in vertical frames are designed as compact sections in accordance with the AISC Specifications or designed to meet the requirements of 2.3.6.4 using twice the strength level seismic loads.</td>
<td>6. The slenderness ratio (k) of primary diagonal bracing in vertical frames is limited to 50 and their ratio of diameter to thickness is limited to 130(1000 / Fy for kis = 130(1000 / Fy for kis). All non-tubular members at connections in vertical frames are designed as compact sections in accordance with the AISC Specifications or designed to meet the requirements of 2.3.6.4 using twice the strength level seismic loads.</td>
<td>3. 3. Time history method - the load effect should be calculated for at least three sets of time histories. The mean values of the calculated action effects from the time history analyses may be taken as basis for design.</td>
</tr>
<tr>
<td><strong>Structural Analysis</strong></td>
<td><strong>1. Structural elements are allowed to exhibit plastic degrading behaviour (e.g. local bucking in steel), but catastrophic failures such as global collapse of failure of a Candidated section of the deck should be avoided.</strong></td>
<td>Material Factor r_m = 1.0</td>
</tr>
<tr>
<td>1. Structure-foundation systems which do not meet the conditions listed in 2.3.6.4 should be analyzed to demonstrate their ability to withstand the rare, intense earthquake without collapsing.</td>
<td>2. Stable plastic mechanisms in foundations are allowed, but catastrophic failure modes such as instability and collapse should be avoided.</td>
<td>2. Stable plastic mechanisms in foundations are allowed, but catastrophic failure modes such as instability and collapse should be avoided.</td>
</tr>
<tr>
<td>2. The time history method of analysis is recommended.</td>
<td>3. At least three sets of representative earthquake ground motion time histories should be analyzed.</td>
<td>3. At least three sets of representative earthquake ground motion time histories should be analyzed.</td>
</tr>
<tr>
<td>3. At least three sets of representative earthquake ground motion time histories should be analyzed.</td>
<td>4. Structural elements are allowed to exhibit plastic degrading behaviour (e.g. local bucking in steel), but catastrophic failures such as global collapse of failure of a Candidated section of the deck should be avoided.</td>
<td>4. 3. Time history method - the load effect should be calculated for at least three sets of time histories. The mean values of the calculated action effects from the time history analyses may be taken as basis for design.</td>
</tr>
<tr>
<td>4. Structural elements are allowed to exhibit plastic degrading behaviour (e.g. local bucking in steel), but catastrophic failures such as global collapse of failure of a Candidated section of the deck should be avoided.</td>
<td>5. Joints are allowed to exhibit limited plastic behaviour but should stay within their ultimate strengths. Alternatively, where large deformations in the joints are anticipated, they shall be designed to demonstrate ductility and residual strength at anticipated deformation levels.</td>
<td>5. Joints are allowed to exhibit limited plastic behaviour but should stay within their ultimate strengths. Alternatively, where large deformations in the joints are anticipated, they shall be designed to demonstrate ductility and residual strength at anticipated deformation levels.</td>
</tr>
</tbody>
</table>

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Revision No.: 1
Date: 2012-01-12

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11 CASE STUDIES

This section describes the two case studies performed within the framework of the comparison study, to demonstrate how the differences in the design codes would affect utilization of the structure. Two studies were performed, analyzing a fixed platform and a floating structure separately in order to assess the different methods and applicable standards for types of structures. It should be stated that even though the selected structures are representative, they have only been analyzed with the objective of comparing the standards and not actual design optimization or practical construction considerations.

11.1 Fixed Platform

11.1.1 Introduction

The purpose of the first case study was to analyze a fixed production platform utilizing the loads and utilization formulas described in the API, ISO, and NORSOK design codes. With BOEMRE’s agreement, DNV utilized one of the Finite Element models from its archives. The available model of the platform was representing a structure designed for more benign environment than those present in the GoM. Therefore, some modifications were necessary to assure that the platform will more realistic in withstanding the increased wave loads. The strengthening of the platform was realized by simply modifying the cross sectional properties for all structural members of the jacket by increasing the outer diameter (OD) and wall thickness. Several iterations of the FE analyses and code checks, utilizing the FE models with different OD to thickness changed ratios were performed, to assess the effect of increased loads on the structure (due to larger diameter of the members, combined with increased wave loads) on the utilization of the members. The motivation behind this initial process was the limitation of applicability of the code check formulae and the resulting adequate range of the member utilization (close to unity). The code check used in this initial screening procedure was performed according to the API RP 2A which was the original design code used in the design of the platform. Based on the results of this exercise, a model with OD and wall thickness uniformly increased for all members of the jacket structure by 40% was found to be appropriate with utilization of all elements falling within allowable limits. The diameter and wall thickness of the piles were also increased by an identical scaling factor. No modifications were deemed necessary for the topside structural members since they were outside the focus of this study. Another significant modification of the original platform model was a reduced water depth to simulate the analyzed GoM platform location. The water depth was adjusted to accommodate the increased wave height and to assure positive deck clearance in the extreme weather conditions.

11.1.2 Analysis Methodology

The modeling, load application, analysis and code checking were performed utilizing GeniE, an advanced engineering software tool for designing and analyzing offshore and maritime structures. Several DNV programs are incorporated into GeniE, providing users with ability to perform complete analyses, including pre- and post-processing, within one program.
The concept model developed in GeniE allows the user to define complex model of the structure, apply the permanent, functional and environmental loads, and define model properties used by other programs in the package. The hydrostatic and hydrodynamic forces due to waves and currents, together with wind loads are computed by WAJAC (according to Morison’s equation), and are automatically transferred for subsequent structural analyses. The non-linear soil-pile analysis is performed in SPLICE, in combination with SESTRA. SPLICE solves the displacements at the pile-structure interface points for a linear-elastic superstructure modeled with non-linear pile foundations. Finally, the FE analysis is performed in SESTRA, the DNV solver for linear structural FE analysis, and the results are imported into GeniE for further post-processing. The element forces calculated by SESTRA are mapped to the capacity model created within GeniE. The final step of the analysis was performing the unity checks, using the code check formulations which are already implemented in the software.

A two parts comparison was performed within the scope of this case study. The first part focused on the global loads comparison, while the second part compared actual utilization formulae specified in the three codes.

11.1.2.1 Comparison of the Global Loads
The global loads comparison was performed using the output files from the wave analysis in WAJAC. This exercise was performed for the extreme condition, 100-yr hurricane for Central GoM. The purpose of this study was to compare the global loads on the structure generated according to the environmental load recipes and combinations, formulated based on the three standards: API RP 2A with API Bulletin 2INT-MET, ISO 19902 and NORSOK N-003.

The wave loads were computed using each standard for 8 wave headings and 36 wave steps and reported in the listing files; see Appendix B. WAJAC creates two separate load cases based on the calculated maximum base shear force and maximum overturning moment which occur at two different phase angles (see Table 11-3).

11.1.2.2 Comparison of Member and Joint Utilizations
The main focus of this part of the comparison study was the member utilization formulas. To assure that the results obtained from the analyses are comparable, the permanent, variable and environmental loads were kept identical. This assumption was made to isolate from the results an impact of differences in the requirements regarding the environmental loads on final member utilizations. For the LRFD method (ISO and NORSOK), action factors were applied. After the FE Analysis was completed, computed element forces were mapped to the capacity model, and member and joint utilizations were calculated for the jacket structure. All parameters utilized in the code check (e.g. member buckling lengths, moment reduction factors) were applied according to the Design Standards requirements. Only one case for each of the code checks was analyzed – peak wave case for the 100-yr hurricane for Central GoM.

Figure 11-1 shows the capacity model defined in GeniE. The code check was performed on the main structural members of the jacket structure. Utilization of the deck structure was not evaluated since it is not as significantly influenced by the environmental loads. The GeniE software
recognizes the joint type (Y, X, or K) based on the geometry of joints and the load path and categorizes the members (chord or brace) intersecting at the considered joint.

Figure 11-1 Capacity Model in GeniE

11.1.3 Boundary conditions
In order to define identical boundary conditions, the same pile-soil model properties were defined for all cases analyzed within the case study. The composition and properties of the soil simulated in the analyses represent soil, which can be found in the GoM. Four (4) groups of three (3) piles, with approximate penetration of 110 m provide the foundation for the jacket structure. The pile-soil model consists of 8 (eight) soil layers, with properties as presented in Table 11-1. API methods
were used to generate the soil property curves: the lateral soil resistance P-Y (API-87), the axial pile load transfer-displacement T-Z (API-93), and the pile tip load-displacement Q-Z (API-93).

### Table 11-1 Properties of Soil Layers

<table>
<thead>
<tr>
<th>Layer</th>
<th>Soil type</th>
<th>From</th>
<th>To</th>
<th>Submerged Unit Weight</th>
<th>Undrained Shear Strength</th>
<th>Angle of Internal Friction</th>
<th>API-J Factor</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>clay</td>
<td>0</td>
<td>5.0</td>
<td>8</td>
<td>5 to 20</td>
<td>-</td>
<td>0.25</td>
</tr>
<tr>
<td>2</td>
<td>clay</td>
<td>5.0</td>
<td>22.0</td>
<td>10</td>
<td>80</td>
<td>-</td>
<td>0.25</td>
</tr>
<tr>
<td>3</td>
<td>sand</td>
<td>22.0</td>
<td>27.5</td>
<td>9.5</td>
<td>30</td>
<td>-</td>
<td>0.25</td>
</tr>
<tr>
<td>4</td>
<td>clay</td>
<td>27.5</td>
<td>36.0</td>
<td>10</td>
<td>100</td>
<td>-</td>
<td>0.25</td>
</tr>
<tr>
<td>5</td>
<td>sand</td>
<td>36.0</td>
<td>54.0</td>
<td>9.5</td>
<td>35</td>
<td>-</td>
<td>0.25</td>
</tr>
<tr>
<td>6</td>
<td>sand</td>
<td>54.0</td>
<td>100.9</td>
<td>10</td>
<td>37</td>
<td>-</td>
<td>0.25</td>
</tr>
<tr>
<td>7</td>
<td>clay</td>
<td>100.9</td>
<td>110.5</td>
<td>11</td>
<td>290</td>
<td>-</td>
<td>0.25</td>
</tr>
<tr>
<td>8</td>
<td>sand</td>
<td>110.5</td>
<td>140.0</td>
<td>10.5</td>
<td>35</td>
<td>-</td>
<td>0.25</td>
</tr>
</tbody>
</table>

The pile capacity evaluation was excluded from the scope of work. Therefore, the pile-soil model was utilized to only formulate realistic boundary conditions for the structural model.

### 11.1.4 Loads and Load Combinations

Only the extreme 100-year hurricane environmental scenario was analyzed – corresponding to the peak wave case (ULS-b in LRFD). A total of 16 load combinations were analyzed in each analysis. For simplicity, the number of load combinations was reduced from that of the original model, analyzing only one position of the drilling module. All permanent and variable loads, as per original design report, were applied to the model.

Wave loads were calculated by WAIJAC and applied to the structure, eight (8) wave headings with 45 deg increments were analyzed (see Figure 11-2). Single design wave approach was employed. Only the peak wave case for extreme condition was analyzed. The conductor shielding factor defined in the program was calculated based on transverse and longitudinal distance between the conductors. The wave kinematics factor of 0.88 was used. Uniform marine growth with 38.1 mm thickness was modeled for 60 m below waterline. Current blockage factor, varying between analyzed wave headings, was used in the analysis (0.70 for end-on, 0.85 for diagonal, and 0.80 for broadside heading). The “Wheeler” current stretching was used in the analyses.
The environmental loads analyzed in the global loads comparison study, are presented in Table 11-2. It should be highlighted, that the NORSOK standard does not include any direct guidance regarding the environmental loads for hurricane condition in the GoM, therefore the wave, current and wind loads model has been specified based on API 2INT-MET requirements, and load combinations were created based on the NORSOK N-003 standard (ref. Section 2.4.1 for details).

In all runs analyzed in the member utilization comparison study, the environmental loads were computed according to requirements of API 2INT-MET. This decision was based on the fact, that all considered design codes allow using site-specific Metocean data in the design. It is also believed that the Metocean data for the GoM included in the API standard has most recently been updated (in 2007) and is therefore adopted herein.
### Table 11-2 Wave Load Analysis Input Summary

<table>
<thead>
<tr>
<th></th>
<th>API</th>
<th>ISO</th>
<th>NORSOK</th>
</tr>
</thead>
<tbody>
<tr>
<td>Water Depth (Including Surge)</td>
<td>m</td>
<td>110</td>
<td>110</td>
</tr>
<tr>
<td>Max Wave Height</td>
<td>m</td>
<td>26.0</td>
<td>24.3</td>
</tr>
<tr>
<td>Max Wave Period</td>
<td>s</td>
<td>13.9</td>
<td>13.2</td>
</tr>
<tr>
<td>Wind Direction (Relative to Wave)</td>
<td>deg</td>
<td>-15</td>
<td>0</td>
</tr>
<tr>
<td>Avg. Wind Speed</td>
<td>m/s</td>
<td>45.6</td>
<td>46.1</td>
</tr>
<tr>
<td>Current Direction (Relative to Wave)</td>
<td>deg</td>
<td>15</td>
<td>0</td>
</tr>
<tr>
<td>Current Speed @ Surface</td>
<td>m/s</td>
<td>1.68</td>
<td>2.1</td>
</tr>
<tr>
<td>Current Speed @ Middle of Profile (Elev.)</td>
<td>m/s</td>
<td>1.46 (32)</td>
<td>1.76 (35)</td>
</tr>
<tr>
<td>Current Speed @ Bottom of Profile (Elev.)</td>
<td>m/s</td>
<td>0.00 (64)</td>
<td>0.09 (70)</td>
</tr>
<tr>
<td>Current Speed @ Mudline</td>
<td>m/s</td>
<td>0.00</td>
<td>0.09</td>
</tr>
</tbody>
</table>

1) Adjusted to the water depth, as per Standards requirements

The environmental loads presented above illustrate different philosophies behind creating the load combinations for FE analyses. Input to the wave analysis for API and NORSOK runs was assumed to be identical; however the guidance for creating the load combinations (only peak wave case was considered) differs between standards. Table 5-1 in API 2INT-MET /11/ provides factors for combining independent extremes into load cases (i.e. for 100-yr hurricane wind speed is reduced by 0.95, and current speed by 0.75), whereas NORSOK N-003 recommends combining the 10-yr current with 100-yr wind and 100-yr wave actions. ISO follows similar philosophy to API factoring the loads with the same return period, however the adjustment is limited to the current speed only (factor of 0.90 is recommended in the ISO 19901-1, Table C.21).

#### 11.1.5 Results

This section presents the results summary only. The detail results can be found in Appendix B.

#### 11.1.5.1 Comparison of the Global Loads

Table 11-3 presents summary of the results from the comparison of the global loads, induced on the structure by the environment. It can be seen; that the loads and load combinations formulated as per API Standards requirements resulted in largest magnitude of the calculated Base Shear, while the maximum calculated Overturning Moment was observed in the ISO run. The loads computed by software show only minor differences between the runs with smallest and largest loads (~1% difference for both Base Shear and Overturning Moment).
Table 11-3 Results of the Global Loads Comparison

<table>
<thead>
<tr>
<th>Heading</th>
<th>Max Base Shear</th>
<th>Max Overturning Mom.¹</th>
</tr>
</thead>
<tbody>
<tr>
<td>deg</td>
<td>MN</td>
<td>Phase</td>
</tr>
<tr>
<td>0</td>
<td>70.5</td>
<td>340</td>
</tr>
<tr>
<td>45</td>
<td>67.7</td>
<td>350</td>
</tr>
<tr>
<td>90</td>
<td>71.2</td>
<td>350</td>
</tr>
<tr>
<td>135</td>
<td>67.8</td>
<td>350</td>
</tr>
<tr>
<td>180</td>
<td>67.8</td>
<td>0</td>
</tr>
<tr>
<td>225</td>
<td>66.8</td>
<td>350</td>
</tr>
<tr>
<td>270</td>
<td>71.4</td>
<td>350</td>
</tr>
<tr>
<td>315</td>
<td>70.0</td>
<td>350</td>
</tr>
<tr>
<td>MAX</td>
<td>71.4</td>
<td>-</td>
</tr>
<tr>
<td>0</td>
<td>68.0</td>
<td>340</td>
</tr>
<tr>
<td>45</td>
<td>67.3</td>
<td>350</td>
</tr>
<tr>
<td>90</td>
<td>70.7</td>
<td>350</td>
</tr>
<tr>
<td>135</td>
<td>66.3</td>
<td>350</td>
</tr>
<tr>
<td>180</td>
<td>65.2</td>
<td>0</td>
</tr>
<tr>
<td>225</td>
<td>66.5</td>
<td>350</td>
</tr>
<tr>
<td>270</td>
<td>70.8</td>
<td>350</td>
</tr>
<tr>
<td>315</td>
<td>67.5</td>
<td>350</td>
</tr>
<tr>
<td>MAX</td>
<td>70.8</td>
<td>-</td>
</tr>
<tr>
<td>0</td>
<td>69.0</td>
<td>340</td>
</tr>
<tr>
<td>45</td>
<td>66.6</td>
<td>350</td>
</tr>
<tr>
<td>90</td>
<td>70.7</td>
<td>350</td>
</tr>
<tr>
<td>135</td>
<td>65.7</td>
<td>350</td>
</tr>
<tr>
<td>180</td>
<td>66.3</td>
<td>350</td>
</tr>
<tr>
<td>225</td>
<td>65.8</td>
<td>350</td>
</tr>
<tr>
<td>270</td>
<td>70.8</td>
<td>350</td>
</tr>
<tr>
<td>315</td>
<td>66.8</td>
<td>350</td>
</tr>
<tr>
<td>MAX</td>
<td>70.8</td>
<td>-</td>
</tr>
</tbody>
</table>

¹) Reference point [0,0,0] – CL of the platform, at the Mudline elevation

The results shown above represent the global loads induced on the structure, calculated by WAJAC. These results however do not include applicable load/action factors for LRFD methods (ISO and NORSOK), which normally would be considered during the structural analyses (not performed at this stage of the study). It is believed that for the analyzed ULS-b limit state, considering applicable factors (1.35 for the ISO, and 1.3 for NORSOK), the largest factored global loads would be calculated for the ISO run.
11.1.5.2 Comparison of the Members and Joints Utilization

This section presents the result summary for the comparison of the structure utilization. These results were divided into two separate sub-sections, where results for members and joints are summarized separately.

a) Member Results

Member utilization results are presented in Tables 11-4 through 11-6.

Table 11-4 presents the results for the base case (API), with corresponding utilization for the remaining Code Check runs (ISO and NORSOK). Utilization Factors (UF) of fifteen (15) highest utilized members from the API Code Check were presented side-by-side with UF's calculated for the corresponding members, for the same Load Cases, from the ISO and NORSOK runs. The maximum UF reported for each member is compared, without considering the position along the member, where it was calculated.

<table>
<thead>
<tr>
<th>Member</th>
<th>API</th>
<th>Corresponding Utilization</th>
<th>Ratio of Total Utilization</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

1) 33% increase of the allowable stresses included (only extreme load case was analyzed)

Results presented in

Table 11-4 for the base case show that the UF calculated according to the API code check formulae are consistently higher than results for remaining codes by up to 39% for ISO and 53% for NORSOK for those 15 members. The lowest utilization was calculated for members according to the NORSOK code check. For the base case, no member was found to fail the code checks (i.e.; there was no overstressed elements).
The results for the highest utilized members for the ISO and NORSOK Code Check runs are presented in Table 11-5. The results show good correlation in the order in which members are listed, showing only two member differences (last two in the table).

Table 11-5 Maximum Member Utilization Results – ISO and NORSOK

<table>
<thead>
<tr>
<th>Member</th>
<th>LC</th>
<th>ISO</th>
<th>LC</th>
<th>NORSOK</th>
<th>Corresponding Utilization (API)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td></td>
<td>740</td>
<td>13</td>
<td>740</td>
<td>1.36 6.71 0.62 API Cone</td>
</tr>
<tr>
<td>2</td>
<td></td>
<td>462</td>
<td>13</td>
<td>462</td>
<td>1.31 6.71 0.60 API Cone</td>
</tr>
<tr>
<td>3</td>
<td></td>
<td>1690</td>
<td>5</td>
<td>1690</td>
<td>1.26 6.71 0.58 API Cone</td>
</tr>
<tr>
<td>4</td>
<td></td>
<td>461</td>
<td>5</td>
<td>461</td>
<td>1.21 6.71 0.56 API Cone</td>
</tr>
<tr>
<td>5</td>
<td></td>
<td>41</td>
<td>5</td>
<td>41</td>
<td>1.16 6.71 0.61 API Cone</td>
</tr>
<tr>
<td>6</td>
<td></td>
<td>36</td>
<td>13</td>
<td>36</td>
<td>1.06 6.71 0.56 API Cone</td>
</tr>
<tr>
<td>7</td>
<td></td>
<td>31</td>
<td>7</td>
<td>31</td>
<td>1.04 6.71 0.46 API Cone</td>
</tr>
<tr>
<td>8</td>
<td></td>
<td>749</td>
<td>15</td>
<td>749</td>
<td>0.96 6.71 0.61 API Cone</td>
</tr>
<tr>
<td>9</td>
<td></td>
<td>10</td>
<td>5</td>
<td>10</td>
<td>0.93 6.71 0.47 API Cone</td>
</tr>
<tr>
<td>10</td>
<td></td>
<td>21</td>
<td>3</td>
<td>21</td>
<td>0.93 6.71 0.38 API Cone</td>
</tr>
<tr>
<td>11</td>
<td></td>
<td>647</td>
<td>3</td>
<td>647</td>
<td>0.89 6.71 0.56 API Cone</td>
</tr>
<tr>
<td>12</td>
<td></td>
<td>646</td>
<td>8</td>
<td>646</td>
<td>0.88 6.71 0.57 API Cone</td>
</tr>
<tr>
<td>13</td>
<td></td>
<td>2</td>
<td>13</td>
<td>2</td>
<td>0.86 6.71 0.43 API Cone</td>
</tr>
<tr>
<td>14</td>
<td></td>
<td>505</td>
<td>1/12</td>
<td>748</td>
<td>0.85 6.71 0.99/0.59 3.3.4-3/Cone</td>
</tr>
<tr>
<td>15</td>
<td></td>
<td>513</td>
<td>1/13</td>
<td>1647</td>
<td>0.83 6.71 0.98/0.44 3.3.4-3/Cone</td>
</tr>
</tbody>
</table>

1) 33% increase of the allowable stresses included
2) corresponding to ISO / NORSOK

It should be highlighted that the formulae giving the maximum utilization factors for the members presented in Table 11-5 describes the utilization of the conical transitions (except Members 505 and 513 in ISO run). There is 10 (ten) members failing the ISO-, and 7 (seven) failing the NORSOK code check. It can also be seen that the difference between the results for the ISO and NORSOK code checks is rather consistent and the ISO calculated utilization is about 10% higher. Corresponding utilization for the API Code Check for these members were found to be significantly lower (about 50%) with no member has failing the unity check.

In order to complete the case study summary, results for the 6 (six) chosen members for all Code Check runs are presented and compared in Table 11-6 and Table 11-7. The member selection was based on the type and location (one brace and one leg member from the top, middle and bottom sections of the jacket), to capture an impact of the ratio of the dynamic to total load on the utilization. The total and partial utilisations (due to axial force and bending moment) are reported.
Table 11-6 Member Utilization Results – Chosen Members

<table>
<thead>
<tr>
<th>Member</th>
<th>Utilization Factor</th>
<th>Utilization Factor</th>
<th>Utilization Factor</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>UF&lt;sub&gt;API&lt;/sub&gt;</td>
<td>UF&lt;sub&gt;ISO&lt;/sub&gt;</td>
<td>UF&lt;sub&gt;NORSOK&lt;/sub&gt;</td>
</tr>
<tr>
<td>ID</td>
<td>Location</td>
<td>Type</td>
<td>TOTAL</td>
</tr>
<tr>
<td>1674</td>
<td>Top</td>
<td>Leg</td>
<td>0.29</td>
</tr>
<tr>
<td>1678</td>
<td></td>
<td>Brace</td>
<td>0.56</td>
</tr>
<tr>
<td>43</td>
<td>Middle</td>
<td>Leg</td>
<td>0.47</td>
</tr>
<tr>
<td>662</td>
<td></td>
<td>Brace</td>
<td>0.64</td>
</tr>
<tr>
<td>1286</td>
<td>Bottom</td>
<td>Leg</td>
<td>0.49</td>
</tr>
<tr>
<td>442</td>
<td></td>
<td>Brace</td>
<td>0.41</td>
</tr>
</tbody>
</table>

Table 11-7 Ratio of Total Utilization – Chosen Members

<table>
<thead>
<tr>
<th>Member</th>
<th>Ratio of Total Utilization</th>
<th>Ratio of Total Utilization</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>UF&lt;sub&gt;API&lt;/sub&gt;/UF&lt;sub&gt;ISO&lt;/sub&gt;</td>
<td>UF&lt;sub&gt;API&lt;/sub&gt;/UF&lt;sub&gt;NORSOK&lt;/sub&gt;</td>
</tr>
<tr>
<td>ID</td>
<td>Location</td>
<td>Type</td>
</tr>
<tr>
<td>1674</td>
<td>Top</td>
<td>Leg</td>
</tr>
<tr>
<td>1678</td>
<td></td>
<td>Brace</td>
</tr>
<tr>
<td>43</td>
<td>Middle</td>
<td>Leg</td>
</tr>
<tr>
<td>662</td>
<td></td>
<td>Brace</td>
</tr>
<tr>
<td>1286</td>
<td>Bottom</td>
<td>Leg</td>
</tr>
<tr>
<td>442</td>
<td></td>
<td>Brace</td>
</tr>
</tbody>
</table>

Results presented in Table 11-6 and Table 11-7 show that member results for the API code check are less conservative than for remaining codes. However, it is an expected difference for the analyzed case (extreme condition for the API run and ULS-b for the ISO and NORSOK), due to a significant increase of the environmental loads for the LRFD method. The difference between the API code check and remaining results is less significant in the middle section of the jacket, where the effect of the environmental load is expected to be lower than at the top and the bottom of the structure.

It can also be seen, that the utilization calculated for the ISO code check is roughly 5% higher than for the NORSOK. The code check formulae are very similar or identical for most of the failure modes in these two codes (ref. Section 3 for details). The difference in calculated utilizations can be caused by different load/action factors applied to the load cases with permanent and variable loads in ISO and NORSOK runs (1.1 vs. 1.0, respectively).
b) Joint Results

The original structure of the platform was designed without overlapping joints. However, due to resizing of the jacket members (described in Section 11.1.1), several members originally designed as non-overlapping were overlapping, causing the joints to fail. The results for these joints were not reported. This was not deemed significant for this study which is not concerned with design optimization but rather only code comparison.

Tables 11-8 through 11-10 present the results summary for the joint utilization. Similar to the member results, the results are presented in three separate tables. Table 11-8 presents the maximum calculated utilization for the base case – the API Code Check run. Only maximum utilizations calculated for the 15 (fifteen) highest utilized joints are reported. Unlike for the member check comparison, the results for the maximum utilized joints for the ISO and NORSOK runs are not presented separately, due to good correlation and similar order of results for all of the runs. More detailed results can be found in Appendix B.

<table>
<thead>
<tr>
<th>Joint</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
</tr>
<tr>
<td>2</td>
</tr>
<tr>
<td>3</td>
</tr>
<tr>
<td>4</td>
</tr>
<tr>
<td>5</td>
</tr>
<tr>
<td>6</td>
</tr>
<tr>
<td>7</td>
</tr>
<tr>
<td>8</td>
</tr>
<tr>
<td>9</td>
</tr>
<tr>
<td>10</td>
</tr>
<tr>
<td>11</td>
</tr>
<tr>
<td>12</td>
</tr>
<tr>
<td>13</td>
</tr>
<tr>
<td>14</td>
</tr>
<tr>
<td>15</td>
</tr>
</tbody>
</table>

Table 11-8 Maximum Joints Utilization Results – Base Case (API)

<table>
<thead>
<tr>
<th>Joint</th>
<th>API</th>
<th>Corresponding Utilization</th>
<th>Ratio of Total Utilization</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>UF&lt;sub&gt;API&lt;/sub&gt;</td>
<td>Corresponding Utilization</td>
<td>Ratio of Total Utilization</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1</td>
<td>33</td>
<td>2.48</td>
<td>13</td>
</tr>
<tr>
<td>2</td>
<td>321</td>
<td>1.88</td>
<td>10</td>
</tr>
<tr>
<td>3</td>
<td>373</td>
<td>1.37</td>
<td>2</td>
</tr>
<tr>
<td>4</td>
<td>335</td>
<td>1.00</td>
<td>15</td>
</tr>
<tr>
<td>5</td>
<td>341</td>
<td>0.91</td>
<td>13</td>
</tr>
<tr>
<td>6</td>
<td>324</td>
<td>0.88</td>
<td>10</td>
</tr>
<tr>
<td>7</td>
<td>339</td>
<td>0.83</td>
<td>11</td>
</tr>
<tr>
<td>8</td>
<td>35</td>
<td>0.68</td>
<td>5</td>
</tr>
<tr>
<td>9</td>
<td>320</td>
<td>0.68</td>
<td>2</td>
</tr>
<tr>
<td>10</td>
<td>16</td>
<td>0.63</td>
<td>13</td>
</tr>
<tr>
<td>11</td>
<td>298</td>
<td>0.61</td>
<td>15</td>
</tr>
<tr>
<td>12</td>
<td>319</td>
<td>0.58</td>
<td>2</td>
</tr>
<tr>
<td>13</td>
<td>366</td>
<td>0.56</td>
<td>3</td>
</tr>
<tr>
<td>14</td>
<td>188</td>
<td>0.56</td>
<td>1</td>
</tr>
<tr>
<td>15</td>
<td>117</td>
<td>0.56</td>
<td>15</td>
</tr>
</tbody>
</table>

1) 33% increase of the allowable stresses included
Table 11-9 Joint Utilization Results – Chosen Joints

<table>
<thead>
<tr>
<th>Joint</th>
<th>Utilization Factor</th>
<th>UF API</th>
<th>UF ISO</th>
<th>UF NORSOK</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>ID/Elev [m]</td>
<td>TOTAL</td>
<td>Axial</td>
<td>Bending</td>
</tr>
<tr>
<td>22 @ Mem1629</td>
<td>Top 111</td>
<td>0.29</td>
<td>0.07</td>
<td>0.22</td>
</tr>
<tr>
<td>195 @ Mem728</td>
<td>Middle 72.3</td>
<td>0.46</td>
<td>0.44</td>
<td>0.02</td>
</tr>
<tr>
<td>298 @ Mem432</td>
<td>Bottom 2.7</td>
<td>0.61</td>
<td>0.55</td>
<td>0.07</td>
</tr>
</tbody>
</table>

Table 11-10 Ratio of Total Utilization – Chosen Joints

<table>
<thead>
<tr>
<th>Member</th>
<th>Ratio of Total Utilization</th>
<th>UF API/UF ISO</th>
<th>UF API/UF NORSOK</th>
</tr>
</thead>
<tbody>
<tr>
<td>22 @ Mem1629</td>
<td>Top</td>
<td>1.02</td>
<td>0.98</td>
</tr>
<tr>
<td>195 @ Mem728</td>
<td>Middle</td>
<td>0.90</td>
<td>0.85</td>
</tr>
<tr>
<td>298 @ Mem432</td>
<td>Bottom</td>
<td>0.75</td>
<td>0.73</td>
</tr>
</tbody>
</table>

These results indicate that for this particular platform, the API calculated joint utilizations are lower than those predicted by the ISO and NORSOK codes by about 10% for the highest loaded joints (UF greater than approximately 0.7). Also the joint utilizations for near top section of platform appear to be better than that for middle and bottom section joints.
11.2 Floater - SPAR

11.2.1 Introduction

The main purpose of this case study is to compare the strength utilizations of the structure of the floater calculated in accordance with the different design codes. The Client had requested DNV to use a SPAR model which is a floater type commonly used by operators in the Gulf of Mexico. A functional FE model of an existing spar platform installed in the Gulf of Mexico waters was made available for use in this study through BOEMRE with the full cooperation and assistance from both the Operator and the Designer. The main focus of the work was directed towards the global strength analysis (yield and buckling checks) of the hard tank structure. Other components of the structure such as the truss, soft tank and topsides and aspects of the structural design related to local design and fatigue strength were not included in the Scope of Work.

It should be noted that although the model was developed by the Designer in the FEED phase of the project and may not represent the final configuration of the structure, it was deemed to be satisfactory for the purpose of this code comparison study. It is realized that due to modeling simplifications, the calculated utilizations for several elements of the structure exceeded allowable limits. Such results were reported herein ‘as is’ without any special consideration or further modification of the model (e.g., by local reinforcement of the critical connections). These aspects were addressed as usual in the detail design stage.

The following standards were used as basis for this comparison:

- API 2FPS and API 2T
- ISO 19904-1
- NORSOK N-004/N-001

The main difference between the API and the other two standards is the design format. API is employing the WSD methodology, whereas ISO and NORSOK adopt the limit state design format. Due to this basic difference, the analysis setup (load combination) and the post-processing needed to be performed separately. The NORSOK N-004 (Design of steel structures) does not include special design provisions for SPARs, as it does for other types of floaters. However, the generic action and material factors recommended for steel structures in the N-001 are identical with those given in the ISO requirements. Therefore the results for these two codes will be identical for the ULS assessment. Furthermore, the ISO 19904-1 provides guidance for a WSD based analyses as an alternative to the LRFD design format. In such a case ISO 19904-1 recommends the use of RCS (Recognized Classification Society) allowable utilization factors. Considering the basic usage factor for the extreme loading conditions recommended in the DNV OS C-201, the allowable stress limit is identical to API WSD requirements.
11.2.2 Analysis Methodology

The global performance and strength analysis of a SPAR platform is a complex task, with requirements to analyze several loading conditions representing the most unfavorable realistic load combinations. In the current analysis several simplifications were made in order to limit the Scope of Work to a manageable level within available resources. Therefore, only one extreme loading condition was analyzed.

The global response of the SPAR composed of two evenly important parts. These are caused by the wave frequency and the low frequency loading conditions. In the original design analysis the Designer computed the response of the platform in two separate steps:

- a frequency domain analysis in WADAM for the wave frequency part, and
- a time domain analysis in MULTISIM for the low frequency forces

In the current study, DNV had undertaken a simplified approach where the results for the low frequency part reported by the Designer (i.e. pitch angle due to combined current and wind action) were combined with the results of the independent wave frequency analysis performed by DNV. To verify this approach the results of DNV analysis were compared with the results reported by the Designer. The comparison yielded a very close correlation between the results of these two analyses. Figure 11-3 presents side-by-side graphic comparison of the calculated global von Mises stress for one combined (static + dynamic) result case.

![Figure 11-3 Comparison of the Designer’s (left) and DNV’s (right) Global Stress Results, (ksi, Nodal Von Mises Stress, 100 year Hurricane)](image-url)
a) Hydrodynamic Analysis

The hydrodynamic wave load analysis was carried out using the 3D potential theory program SESAM WADAM, which calculates RAO’s for motions and loads in long crested regular waves. WADAM is a general purpose hydrodynamic analysis program for calculation of wave loading and wave induced responses of fixed and floating marine structures with zero or low forward speed. WADAM computations take place in the frequency domain.

Two types of calculations were carried out using WADAM:

- Hydrostatic calculations, in which the hydrostatic and inertia properties of the structure are calculated, together with the loading from weight and buoyancy. This loading is important for equilibrium checking, and the static load must also be included in the subsequent structural analysis.
- Load calculations, in which the detailed pressure distribution on an element level is calculated. These pressures are transferred to the structural FEM model for subsequent quasi-static structural analysis. Mapping of the hydrostatic and dynamic pressure on the structural model is shown in Figure 11-4.

In this study only one environmental scenario, the extreme 100-yr hurricane condition (ULS-b for the LRFD), and one mass distribution were analyzed. The draft for this condition (provided by the Designer) was 153.9 m.

The panel and mass models provided by the Designer were re-used in the analysis. In the WADAM analysis the SPAR was analyzed as a free floating body, without considering the coupling effect of the mooring lines and risers on the spar motion response. It is assumed that this simplification leads to more conservative results; however this effect is not expected to be significant.

It should be noted that DNV did not perform a global performance analysis as a part of this study. Instead, the design wave selection and headings from the original Designer’s analysis were used. Table 11-11 presents the design wave selection for analyzed cases. The analyzed wave headings with reference to the platform coordinate system are shown in Figure 11-1.

<table>
<thead>
<tr>
<th>Design Wave Case</th>
<th>Wave Amplitude [m]</th>
<th>Wave Period [s]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Max Shear</td>
<td>10.2</td>
<td>12</td>
</tr>
<tr>
<td>Max Moment</td>
<td>11.1</td>
<td>14</td>
</tr>
<tr>
<td>Max Axial</td>
<td>4.3</td>
<td>20</td>
</tr>
</tbody>
</table>
Figure 11-4 Static and Dynamic Pressure Distribution on the Structure

Figure 11-5 Analyzed Wave Headings
b) Structural Analysis

Structural analysis limited in this comparison study to the hard tank structure was performed to verify the global strength of the SPAR. In the analyzed configuration the SPAR model consists of the hard and soft tanks and the truss section with two (2) heave plates. The 3-level topside structure is supported on the hard tank by four (4) jacket legs. Structural integrity was checked with respect to yield and buckling. Yield checks are performed based on membrane von Mises stress (element average), checked against allowable stress limits specified in the compared standards. The standards allow using the rules of RCS for the buckling calculations; therefore for simplicity the buckling checks were performed in accordance with DNV RP-C201. In the current study no consideration was given to the code checking of the beam members in the truss section since beam utilization formulae were compared in the fixed structure case study.

The global structural model was generated using the pre-processor Patran PRE. The super-element technique was utilized; five super-elements were assembled using PRESEL (see Figure 11-6). Decks, bulkheads and web frames were modelled using 4-node shell elements. Flanges of web frames were modelled using 2-node beam elements. Stiffeners were also modelled with 2-node beam elements with eccentricity. Mesh size was based on the stiffener spacing (one element between stiffeners).

A steel density of 7.85 t/m³ with a Poisson’s ratio of 0.3 and a Young’s modulus of 2.1x10⁵ MPa was applied to the structural model. The yield strength of the structural steel is 345 MPa.

The rigid body motions of the model were restrained by means of fixing translations in the x, y and z directions (pinned) for the nodes at the fairlead locations (9 nodes around perimeter of the hard tank, at elevation 110m ABL). The sum of the reaction forces was checked and confirmed to be similar to the mooring line forces reported by the Designer.

Following loads were applied to the structural model:

12. Gravity acceleration
13. Static pressure (on the outer shell and the moon pool bulkheads)
14. Riser loads (SCR and TTR)
15. Tilted gravitational acceleration (corresponding to 6.6° low frequency pitch angle)
16. Dynamic wave loads (pressures and inertia loads from WADAM analysis)

The local effect of the wind and current loads on the hard tank structure is considered negligible compared to extreme wave loading, therefore these loads were not applied to the model directly. The global effect of the current and wind loads on the behaviour of the platform (i.e. inclination) was implemented in the analysis by applying tilted gravitational acceleration.

The action factors presented in Table 2-1 were applied to the ISO/Norsok (LRFD) runs. The utilization of the structure was calculated based on the recommended for the ULS limit state resistance (material) factor $\gamma_M=1.15$.
Table 11-12 Action Factors Applied to the ULS-b Limit State

<table>
<thead>
<tr>
<th>Action Category</th>
<th>Permanent (G)</th>
<th>Variable (Q)</th>
<th>Environmental (E)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>1.0</td>
<td>1.0</td>
<td>1.3</td>
</tr>
</tbody>
</table>

Table 11-13 below presents API and ISO recommended usage factors applicable to the extreme loading condition.

Table 11-13 Usage Factors Applied to Extreme 100 yr Hurricane Loading Condition

<table>
<thead>
<tr>
<th>Usage Factor</th>
<th>API</th>
<th>ISO</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>$\frac{1}{1.67} \cdot 1.33$</td>
<td>0.8*</td>
</tr>
</tbody>
</table>

*) Based on DNV OS-C201
Figure 11-6 Structural Model Overview
11.2.3 Results

a) Yield Check

A yield check was performed based on membrane von Mises stresses (element average). Stresses induced by each complex wave load case were combined with the static load cases. These combined cases were scanned in order to find the maximum von Mises stress for each element along all wave cases. The results are presented for three (3) panels for each of the four (4) regions of the hard tank (defined by the distance between watertight decks): outer shell, radial bulkhead and center well (moonpool) bulkhead.

Two sets of the results are reported – for the elements with maximum von Mises stress and for the elements in the centre of each of the panels. The maximum stresses indicate the local stress concentrations (caused by the modelling simplifications or lack of the local reinforcements added to the structure in the detail engineering phase of the project) and may not represent the stress level in the actual structure. Simultaneously, the stress level for the elements closer to the geometric centre of the panel are governed by the global and local loads on the structure, and are not significantly impacted by the local modelling approximations.

Table 11-14 and Table 11-15 present result of the comparison of the von Mises stresses. It can be seen, that the utilization calculated based on the maximum observed stress is slightly higher for the LRFD runs (about 10%). Comparison of the utilizations calculated based on the stresses in the middle of the panel yielded close correlation between the results for WSD and LRFD utilizations.

It is worth mentioning, that the comparison is based on one loading condition only – extreme 100-yr hurricane for the GOM (ULS-b for the LRFD method) and is only reflective of the magnitudes of the static and dynamic loadings on this structure.
## Table 11-14 Maximum Von Mises Stresses Reported for Analyzed Panels

<table>
<thead>
<tr>
<th>Region</th>
<th>Panel</th>
<th>WSD (API &amp; ISO)</th>
<th>LRFD – ULS-b (ISO &amp; Norsok)</th>
<th>Ratio of Utilization</th>
<th>Stress Location</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Stress [Mpa]</td>
<td>Stress [Mpa]</td>
<td>UF&lt;sub&gt;WSD&lt;/sub&gt;</td>
<td>UF&lt;sub&gt;LRFD&lt;/sub&gt;</td>
<td>Elev. (ABL) [m]</td>
</tr>
<tr>
<td></td>
<td></td>
<td>UF&lt;sub&gt;WSD&lt;/sub&gt;[</td>
<td>UF&lt;sub&gt;LRFD&lt;/sub&gt;</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Deck</td>
<td>Radial Bhd</td>
<td>251</td>
<td>260</td>
<td>1.05</td>
<td>93.7</td>
<td>Intersection with OS</td>
</tr>
<tr>
<td>3-4</td>
<td>Moonpool Bhd</td>
<td>205</td>
<td>212</td>
<td>1.04</td>
<td>88.7</td>
<td>Radial bulkhead @ Deck 4</td>
</tr>
<tr>
<td></td>
<td>OS</td>
<td>577</td>
<td>683</td>
<td>0.92</td>
<td>108.0</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Radial Bhd</td>
<td>380</td>
<td>442</td>
<td>0.94</td>
<td>108.7</td>
<td>Intersection with OS</td>
</tr>
<tr>
<td>Deck</td>
<td>Moonpool Bhd</td>
<td>127</td>
<td>147</td>
<td>0.94</td>
<td>110.9</td>
<td>Moonpool corner</td>
</tr>
<tr>
<td>4-5</td>
<td>OS</td>
<td>409</td>
<td>486</td>
<td>0.91</td>
<td>108.9</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Radial Bhd</td>
<td>161</td>
<td>201</td>
<td>0.87</td>
<td>128.5</td>
<td>Intersection with OS</td>
</tr>
<tr>
<td></td>
<td>Moonpool Bhd</td>
<td>88</td>
<td>101</td>
<td>0.94</td>
<td>138.4</td>
<td>Moonpool corner</td>
</tr>
<tr>
<td></td>
<td>OS</td>
<td>170</td>
<td>209</td>
<td>0.89</td>
<td>128.5</td>
<td></td>
</tr>
<tr>
<td>Deck</td>
<td>Radial Bhd</td>
<td>411</td>
<td>485</td>
<td>0.92</td>
<td>154.7</td>
<td>Intersection with OS</td>
</tr>
<tr>
<td>5-6</td>
<td>Moonpool Bhd</td>
<td>105</td>
<td>118</td>
<td>0.97</td>
<td>168.7</td>
<td>Moonpool CL</td>
</tr>
<tr>
<td></td>
<td>OS</td>
<td>408</td>
<td>477</td>
<td>0.93</td>
<td>154.7</td>
<td>Radial bulkhead</td>
</tr>
<tr>
<td>Deck</td>
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<td>155.1</td>
<td>164</td>
<td>1.02</td>
<td>101.0</td>
<td></td>
</tr>
<tr>
<td>6-7</td>
<td>Moonpool Bhd</td>
<td>114.4</td>
<td>119</td>
<td>1.03</td>
<td>118.8</td>
<td></td>
</tr>
<tr>
<td></td>
<td>OS</td>
<td>126.2</td>
<td>133.9</td>
<td>1.02</td>
<td>138.5</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Radial Bhd</td>
<td>107.1</td>
<td>126.9</td>
<td>0.93</td>
<td>161.2</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Moonpool Bhd</td>
<td>77.3</td>
<td>82.7</td>
<td>1.00</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>OS</td>
<td>140.1</td>
<td>155.7</td>
<td>0.98</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>Radial Bhd</td>
<td>101.1</td>
<td>120.6</td>
<td>0.93</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>Moonpool Bhd</td>
<td>74.5</td>
<td>85</td>
<td>0.96</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>OS</td>
<td>103.4</td>
<td>119</td>
<td>0.93</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>Radial Bhd</td>
<td>102.7</td>
<td>110.8</td>
<td>1.00</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>Moonpool Bhd</td>
<td>71.4</td>
<td>90.1</td>
<td>0.87</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>OS</td>
<td>53.6</td>
<td>69</td>
<td>0.83</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

## Table 11-15 Von Mises Stress for the Centre of Analyzed Panels

<table>
<thead>
<tr>
<th>Region</th>
<th>Panel</th>
<th>WSD (API &amp; ISO)</th>
<th>LRFD – ULS-b (ISO &amp; Norsok)</th>
<th>Ratio of Utilization</th>
<th>Elev. (ABL)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Stress [Mpa]</td>
<td>Stress [Mpa]</td>
<td>UF&lt;sub&gt;WSD&lt;/sub&gt;</td>
<td>UF&lt;sub&gt;LRFD&lt;/sub&gt;</td>
</tr>
<tr>
<td></td>
<td></td>
<td>UF&lt;sub&gt;WSD&lt;/sub&gt;[</td>
<td>UF&lt;sub&gt;LRFD&lt;/sub&gt;</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Deck</td>
<td>Radial Bhd</td>
<td>155.1</td>
<td>164</td>
<td>1.02</td>
<td>101.0</td>
</tr>
<tr>
<td>3-4</td>
<td>Moonpool Bhd</td>
<td>114.4</td>
<td>119</td>
<td>1.03</td>
<td>118.8</td>
</tr>
<tr>
<td></td>
<td>OS</td>
<td>126.2</td>
<td>133.9</td>
<td>1.02</td>
<td>138.5</td>
</tr>
<tr>
<td>Deck</td>
<td>Radial Bhd</td>
<td>107.1</td>
<td>126.9</td>
<td>0.93</td>
<td>161.2</td>
</tr>
<tr>
<td>4-5</td>
<td>Moonpool Bhd</td>
<td>77.3</td>
<td>82.7</td>
<td>1.00</td>
<td></td>
</tr>
<tr>
<td></td>
<td>OS</td>
<td>140.1</td>
<td>155.7</td>
<td>0.98</td>
<td></td>
</tr>
<tr>
<td>Deck</td>
<td>Radial Bhd</td>
<td>101.1</td>
<td>120.6</td>
<td>0.93</td>
<td></td>
</tr>
<tr>
<td>5-6</td>
<td>Moonpool Bhd</td>
<td>74.5</td>
<td>85</td>
<td>0.96</td>
<td></td>
</tr>
<tr>
<td></td>
<td>OS</td>
<td>103.4</td>
<td>119</td>
<td>0.93</td>
<td></td>
</tr>
<tr>
<td>Deck</td>
<td>Radial Bhd</td>
<td>102.7</td>
<td>110.8</td>
<td>1.00</td>
<td></td>
</tr>
<tr>
<td>6-7</td>
<td>Moonpool Bhd</td>
<td>71.4</td>
<td>90.1</td>
<td>0.87</td>
<td></td>
</tr>
<tr>
<td></td>
<td>OS</td>
<td>53.6</td>
<td>69</td>
<td>0.83</td>
<td></td>
</tr>
</tbody>
</table>
Where:

\[ UF_{WSD} = \frac{\sigma_{Y-M}}{\left(\frac{\sigma_Y}{1.67}\right) \cdot 1.33} \quad \text{(11.1a)} \]

\[ UF_{LRFD} = \frac{\sigma_{Y-M}}{\left(\frac{\sigma_Y}{1.15}\right)} \quad \text{(11.1b)} \]

b) Buckling Check

A buckling check was performed based on the membrane component stresses (element average). The DNV RP C-201 /-/ was used to calculate the buckling utilization of the panels. The flat panel formulations were used for all panels, including outer shell plate, which is acceptable considering large D/t ratio for the curved panels (D/t > 1400). The comparison focuses on the stiffened panel buckling, exclusive of the girder checks.

The results are presented in Table 11-16 and Table 11-17. It can be seen, that the buckling checks for the plate for the WSD runs indicate higher utilization of the panels. The results for the stiffeners are very similar for both runs.
### Table 11-16 Results of the Buckling Check – API WSD Run

<table>
<thead>
<tr>
<th>Region</th>
<th>Panel</th>
<th>Oct. 2010</th>
<th>Breadth S (m)</th>
<th>Length L (m)</th>
<th>Thickness T (mm)</th>
<th>Type</th>
<th>Spacing</th>
<th>X (MPa)</th>
<th>Y (MPa)</th>
<th>Shear (MPa)</th>
<th>Pressure Utilization</th>
<th>Stiffness Utilization</th>
</tr>
</thead>
<tbody>
<tr>
<td>Deck 3-4</td>
<td>Radial blk</td>
<td>8.05</td>
<td>1.98</td>
<td>21</td>
<td>HP370x13</td>
<td>0.75</td>
<td>-55</td>
<td>-122</td>
<td>65</td>
<td>0.000</td>
<td>0.56</td>
<td>0.63</td>
</tr>
<tr>
<td></td>
<td>Moonpool blk</td>
<td>14.02</td>
<td>1.98</td>
<td>24</td>
<td>HP340x12</td>
<td>0.79</td>
<td>-40</td>
<td>-115</td>
<td>30</td>
<td>0.504</td>
<td>0.45</td>
<td>0.60</td>
</tr>
<tr>
<td></td>
<td>Outer shell</td>
<td>28.21</td>
<td>1.98</td>
<td>23</td>
<td>HP400x14</td>
<td>0.76</td>
<td>-55</td>
<td>-98</td>
<td>70</td>
<td>0.543</td>
<td>0.54</td>
<td>0.64</td>
</tr>
<tr>
<td>Deck 4-5</td>
<td>Radial blk</td>
<td>8.05</td>
<td>1.98</td>
<td>20</td>
<td>HP370x13</td>
<td>0.75</td>
<td>-48</td>
<td>-115</td>
<td>51</td>
<td>0.000</td>
<td>0.48</td>
<td>0.56</td>
</tr>
<tr>
<td></td>
<td>Moonpool blk</td>
<td>14.02</td>
<td>1.98</td>
<td>21</td>
<td>HP340x12</td>
<td>0.79</td>
<td>-37</td>
<td>-92</td>
<td>22</td>
<td>0.321</td>
<td>0.37</td>
<td>0.48</td>
</tr>
<tr>
<td></td>
<td>Outer shell</td>
<td>28.21</td>
<td>1.98</td>
<td>21</td>
<td>HP340x12</td>
<td>0.76</td>
<td>-60</td>
<td>-142</td>
<td>28</td>
<td>0.379</td>
<td>0.48</td>
<td>0.71</td>
</tr>
<tr>
<td>Deck 5-6</td>
<td>Radial blk</td>
<td>8.05</td>
<td>1.98</td>
<td>15</td>
<td>HP280x12</td>
<td>0.75</td>
<td>-39</td>
<td>-99</td>
<td>39</td>
<td>0.000</td>
<td>0.40</td>
<td>0.55</td>
</tr>
<tr>
<td></td>
<td>Moonpool blk</td>
<td>14.02</td>
<td>1.98</td>
<td>16</td>
<td>HP260x10</td>
<td>0.79</td>
<td>-30</td>
<td>-79</td>
<td>19</td>
<td>0.119</td>
<td>0.28</td>
<td>0.48</td>
</tr>
<tr>
<td></td>
<td>Outer shell</td>
<td>28.21</td>
<td>1.98</td>
<td>20</td>
<td>HP300x12</td>
<td>0.76</td>
<td>-48</td>
<td>-99</td>
<td>16</td>
<td>0.185</td>
<td>0.33</td>
<td>0.50</td>
</tr>
<tr>
<td>Deck 6-7</td>
<td>Radial blk</td>
<td>8.05</td>
<td>2.13</td>
<td>15</td>
<td>HP280x12</td>
<td>0.75</td>
<td>-14</td>
<td>-75</td>
<td>55</td>
<td>0.000</td>
<td>0.43</td>
<td>0.46</td>
</tr>
<tr>
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<td>2.13</td>
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<td>HP260x10</td>
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<td>-26</td>
<td>23</td>
<td>0.000</td>
<td>0.17</td>
<td>0.16</td>
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<tr>
<td></td>
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<td>28.2</td>
<td>2.13</td>
<td>20</td>
<td>HP280x12</td>
<td>0.76</td>
<td>-15</td>
<td>-75</td>
<td>67</td>
<td>0.000</td>
<td>0.49</td>
<td>0.48</td>
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</table>
### Table 11-17 Results of the Buckling Check – ISO/NORSOK LRFD Run

<table>
<thead>
<tr>
<th>Region</th>
<th>Panel</th>
<th>Oct. 2010</th>
<th>Plate Type</th>
<th>Stiffener Spacing</th>
<th>Stresses</th>
<th>Unity Check</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
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<td></td>
<td></td>
<td></td>
<td>s, σx, σy, τxy</td>
<td>P_HYD</td>
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<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>η_Plate, η_Stiff</td>
</tr>
<tr>
<td>Deck 3-4</td>
<td>Radial blk</td>
<td>8.05</td>
<td>1.98</td>
<td>21 HP370x13</td>
<td>0.75</td>
<td>-62 -141 73</td>
</tr>
<tr>
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<td>Moonpool blk</td>
<td>14.02</td>
<td>1.98</td>
<td>24 HP340x12</td>
<td>0.79</td>
<td>-48 -123 31</td>
</tr>
<tr>
<td></td>
<td>Outer shell</td>
<td>28.21</td>
<td>1.98</td>
<td>23 HP400x14</td>
<td>0.76</td>
<td>-59 -129 68</td>
</tr>
<tr>
<td>Deck 4-5</td>
<td>Radial blk</td>
<td>8.05</td>
<td>1.98</td>
<td>20 HP370x13</td>
<td>0.75</td>
<td>-54 -128 45</td>
</tr>
<tr>
<td></td>
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<td>1.98</td>
<td>21 HP340x12</td>
<td>0.79</td>
<td>-44 -96 24</td>
</tr>
<tr>
<td></td>
<td>Outer shell</td>
<td>28.21</td>
<td>1.98</td>
<td>21 HP340x12</td>
<td>0.76</td>
<td>-62 -157 30</td>
</tr>
<tr>
<td>Deck 5-6</td>
<td>Radial blk</td>
<td>8.05</td>
<td>1.98</td>
<td>15 HP280x12</td>
<td>0.75</td>
<td>-45 -106 50</td>
</tr>
<tr>
<td></td>
<td>Moonpool blk</td>
<td>14.02</td>
<td>1.98</td>
<td>16 HP260x10</td>
<td>0.79</td>
<td>-35 -86 26</td>
</tr>
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<td></td>
<td>Outer shell</td>
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<td>1.98</td>
<td>20 HP300x12</td>
<td>0.76</td>
<td>-63 -24 17</td>
</tr>
<tr>
<td>Deck 6-7</td>
<td>Radial blk</td>
<td>8.05</td>
<td>2.13</td>
<td>15 HP280x12</td>
<td>0.75</td>
<td>-16 -59 64</td>
</tr>
<tr>
<td></td>
<td>Moonpool blk</td>
<td>14.02</td>
<td>2.13</td>
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<td>2.13</td>
<td>20 HP280x12</td>
<td>0.76</td>
<td>-8 -74 50</td>
</tr>
</tbody>
</table>

### 11.3 Summary

API joint check utilizations for the fixed platform case study are about 10% lower than ISO results and 18% lower than NORSOK values. Therefore the NORSOK joints would be significantly more conservative in comparison with ISO and API. This is a conclusion relevant to the case study that may not be generalized without further evaluations.

The Spar case study indicates that the yield check utilization calculated based on the maximum observed hull stress is slightly higher for the ISO/NORSOK LRFD (about 10%) compared to the API WSD results. The buckling checks for the plate for the WSD runs indicate higher utilizations (about 10%) of the panels while the results for the stiffeners show that the LRFD gives higher utilizations (also about 10%) than the WSD.
12 CONCLUSIONS AND RECOMMENDATIONS

This study covers an extensive scope of work comparing API, ISO, and NORSOK structural standards currently in use for design, construction, installation, and in-service inspection of fixed and floating offshore structures with emphasis on application to the US Gulf of Mexico and West Coast. The following salient conclusions are made from the work:

1. The design environmental loads such as wind, wave, and current depend on geographical locations. In absence of site-specific data, regional information is defined in all three codes that give minimum requirements of the extreme environmental conditions. ISO 19902 adopts environmental criteria proposed by API for the Gulf of Mexico and by NORSOK for the North Sea.

2. For snow and ice loading, NORSOK N-003 and ISO 19901-1 provide more specific information compared to API RP 2A. For earthquake; ISO 19902 and ISO 19901-2 give more comprehensive design guidelines when compared with API or NORSOK standards.

3. API RP 2T is more comprehensive than RP 2A in defining the loads and load combinations due to the sensitivity of the Tendon Leg Platform with regards to its payload.

4. Both API RP 2A and 2T utilize WSD approach for the design of the structure. Notably, the RP 2T 3rd Edition (latest) specifies the limit states design approach for the tendon design which was not the case in the previous editions of the document.

5. The API RP 2FPS 1st Edition issued in 2001 for GOM floating production systems refers to API RP’s 2A and 2T for the definition of the environmental criteria and guidance on load conditions.

6. Wave kinematics factor is similar in the three standards varying, e.g.; from 0.85 to 0.95 for tropical storms. Marine growth is dependent on the regional conditions with about double the marine growth required in the North Sea compared to the GOM. The same drag and inertia coefficients are specified across the three codes. The wind spatial coherence is the same in API and ISO but is more severe in NORSOK.

7. With regards to deck clearance requirements, it is noted that all three codes require 1.5m (5 ft) air gap above the 100-year wave crest elevation. The ISO 19902 gives more details on how to calculate the deck elevation and has an additional criterion of 30% of wave crest elevation as governing clearance if greater than the 1.5m. The NORSOK N-003 and N-004 require a positive air gap for the 10,000 year wave crest in addition to the 1.5m above the 100 year wave crest requirement.

8. The load/action factors are similar in 2A LRFD and ISO 19902. The API RP 2A WSD, API RP 2A LRFD, ISO 19902 and NORSOK N-004 provisions for checking the adequacy of tubular members are similar in that all four codes give formulations for each load effect type acting alone and for load effects acting in combination.

9. API allows simplified fatigue calculations only for Category L-3 template type platforms that are constructed of notch-tough ductile steels, have redundant inspectable structure, and have natural period of less than 3s or for preliminary design of all structure categories in water depth up to 400 ft (122m). NORSOK refers to DNV-RP-C203, Section 5 for details of the methodology and the allowable stress range as function of the Weibull shape parameter and
applicable fatigue curve (depending on the joint detail and stress field configuration) for 20 years’ service life (10^8 cycles).

10. Detail fatigue assumptions, loading definitions, hot spot stress range calculation, stress concentration factor formulas, S-N curves for tubular joints, and required DFF values are specified in all three codes. In addition, details of the spectral analysis, utilization of fracture mechanics, and fatigue life improvement techniques are also compared. The requirements are quite similar.

11. With regards to the safety related to pile design, code requirements and recommendations are similar in the three standards, and the choice of standard will not therefore be decisive. No calibration of safety factors towards probability of failure is documented as background for the safety factors given in the standards. A small structure with few legs/piles has less redundancy than a structure with many legs and piles and correspondingly a higher probability of failure. The designer’s choice of relevant pile capacity calculation method and of related soil shear strength parameters is more important for the overall safety related to pile foundation. Effects not normally accounted for in pile design may have large influence on safety, such as ageing effects and effects of cyclic loading.

12. The API RP 2A (WSD) is the main document where specific guidance with regards to the in-service inspection scope and frequency for fixed platforms is available. In-service inspection requirements for floating production units (2FPS) and tension leg platforms (2T) are also given but at a higher level than 2A. The ISO approach to in-service inspection requirements adopts the Structural Integrity Management (SIM) methodology and also applies RBI procedures. NORSOK presents only high level requirements regarding in-service inspection program.

13. The assessment criteria in API RP 2A Section 17 allows the use of reduced environmental criteria. ISO 19902 does not have reduced criteria but allows local damage provided reserve strength is verified. NORSOK does not allow any degradation and requires existing structures to be able to resist ULS and ALS conditions at same safety levels as for new structures.

14. For design against fire and blast, API RP 2A charts the assessment process in the form of six main tasks and three risk levels utilizing the ALARP principle and assessing the consequences in a structured manner. In ISO 19902 hazards are grouped into three main groups according to a probability of occurrence or return period of being exceeded. NORSOK N-001, N-004 and N-006 state that the structure shall be ALS checked for the design accidental actions defined in the risk analysis recommended in the standards. With regards to ship collision ALS design, NORSOK gives the most comprehensive guidance of the three codes.

15. The details of the requirements for temporary conditions in API RP 2A are given in Sections 2.4 and 12. Clauses 8 and 22 of ISO 19902 provide the LSD methods for temporary condition design. NORSOK N-004 Clause K.4.4.6 states that transportation and installation design and operation shall comply with the requirements given in NORSOK J-003. It is noted that NORSOK J-003 (1997) requirements have been completely incorporated in the more recent ISO 19901-6 “Marine Operations” issued in 2009. Therefore, the comparison made here is actually also a comparison between API and ISO, and demonstrates that the three codes are similar with different level of guidance and some minor quantitative differences.
16. Seismicity is not normally a design issue in the North Sea. Therefore, Seismic analysis comparisons are herein focused mainly on ISO and API requirements. The structure is designed for two levels of earthquakes in API and ISO: a) strength Level Earthquake (SLE) or Extreme Level Earthquake (ELE) corresponding to 100 – 200 year return period with stress not exceeding yield, and b) Ductility Level Earthquake (DLE) or Abnormal Level Earthquake (ALE) with 1000-5000 year return with stress allowed to exceed yield leading to damage but without collapse.

17. The fixed offshore platform design case study showed that design environmental criteria in the three codes are based on similar reliability analyses and definition of probability of failure. However no details are given in the standards regarding the underlying assumptions employed in these analyses. Some differences exist in defining the load combinations. Applying the load recipes in the three standards to a case study structure in the GOM indicated that similar base shear and over turning moment values are predicted for the same 100-yr extreme environmental condition.

18. Member utilization comparison indicates that both ISO and NORSOK give very conservative formulation for members with cone transitions compared to API. Notwithstanding cone checks, member results for the API code check were more conservative with up to 39% higher utilization than the ISO and 53% higher than NORSOK results. One reason may be that API recommends the use of a 0.8 buckling factor brace members compared to 0.7 recommended in ISO and NORSOK. The ISO and NORSOK member utilizations are significantly higher than API values (more than double) due to cone transition formulations adopted by these two codes.

19. API joint check utilizations for the fixed platform case study are about 10% lower than ISO results and 18% lower than NORSOK values. Therefore the NORSOK joints would be significantly more conservative in comparison with ISO and API.

20. The Spar case study indicates that the yield check utilization calculated based on the maximum observed hull stress is slightly higher for the ISO/NORSOK LRFD (about 10%) compared to the API WSD results. The buckling checks for the plate for the WSD runs indicate higher utilizations (about 10%) of the panels while the results for the stiffeners show that the LRFD gives higher utilizations (also about 10%) than the WSD.

The following recommendations are also made:

1. A more comprehensive comparison of the principles and methodology employed in arriving at the action/load factors or safety factors in LRFD and WSD methodologies are recommended.

2. More case studies with more in depth specific calculations would add considerable value to the comparisons. Further analysis of the results from the case studies performed herein could yield better understanding of specific code differences.

3. Case studies for a TLP and an FPSO are commended.
4. A venue where the results from this work are presented and experts are invited to participate would be most valuable. API and ISO committees could be viable options for such a discussion. Discussion of results with operators, designers, regulators and other stakeholders is encouraged.

5. Each section of this report could be expanded into an own study. It is recommended to have similar studies carried out on each specialized topic involving industry experts.

6. More case studies for fixed and floating structures should be carried out. Also the case studies performed herein could benefit from additional parametric and sensitivity analyses.

7. Both ISO 19902 and NORSOK N-003 require very strict air gap compared to existing API requirements. This may be because the platforms in the North Sea are not evacuated during storms. However the question of the air gap still deserves further assessment for GOM structures even with the increased criteria given in the upcoming API RP 2MET 1st Edition and API RP 2A 22nd Edition.

8. The degree of conservatism or lack of same between API and ISO/NORSOK member cone unity check formulations (with API predicting much lower member utilizatons by a factor of more than 2.0) deserves further evaluation.

9. The question of reduced assessment criteria for existing platforms compared to criteria for new designs should be further examined considering risk and reliability principles. New API RP 2SIM and ongoing RBI JIP work are efforts in this direction.

10. The comparison performed herein considered only the current (at time of contract award in late 2010) editions of the standards. It is recommended to update the study by incorporating the contemplated significant updates in API, ISO and NORSOK standards.
13 REFERENCES


/6/ PAFA Consulting Engineers (2000). C031-002-R Rev2, “Implications for the Assessment of Existing Fixed Steel Structures of Proposed ISO 13819-2 Member Strength Formulations”.


/14/ ISO 19901-1 - Petroleum and natural gas industries - Specific requirements for offshore structures - Part 1: MetOcean Design and Operating Conditions, 1st edition, November 2005


NORSOK Standard N-001 - Integrity of Offshore Structures, 7th edition, June 2010


NORSOK Standard N-004 - Design of Steel Structures, 2nd edition, October 2004


http://www.texaspelagics.com/GOMocean.html


DNV-RP-C201, “Buckling Strength of Plated Structures”, October 2010


APPENDIX A: MATHCAD UNITY CHECK SHEETS
A.1 MATHCAD - API RP 2A
API RP 2A
Axial Tension, Bending and Hoop Buckling
Member No. 633, LC752_8, Pos 1.0

INPUT the following data:
Diameter, D (mm)
Wall thickness, t (mm)
Yield Strength, \( \sigma_y \) (N/mm²)
Young’s Modulus, E (N/mm²)
Allowable tension stress (N/mm²)
Allowable bending stress \( f_b \) (N/mm²)
Tension stress due to actual loads, \( f_t \) (N/mm²)
Axial compression stress due to actual loads \( f_c \) (N/mm²)
Bending stress due to actual loads about y axis, \( f_{by} \) (N/mm²)
Bending stress due to actual loads about z axis, \( f_{bz} \) (N/mm²)
Hoop stress due to actual loads, \( f_h \) (N/mm²)
Unbraced length of member for local y axis, \( L_y \) (mm)
Unbraced length of member for local z axis, \( L_z \) (mm)

INPUT DATA

\[
\begin{align*}
D & := 1066.8 & D & = 1.067 \times 10^3 \\
t & := 17.8 & t & = 17.8 & \frac{D}{t} & = 59.933 \\
\sigma_y & := 343 & \sigma_y & = 343 \\
E & := 210000 & E & = 2.1 \times 10^5 \\
\sigma_t & := 0 & \sigma_t & = 0 \\
\sigma_h & := 29 & \sigma_h & = 29 \\
f_{by} & := 3.93734375 & f_{by} & = 3.937 \\
f_{bz} & := 11.016950 & f_{bz} & = 11.017 \\
f_h & := 29.462564 & f_h & = 29.463 \\
\frac{1}{\nu} & := 16178 & 1 & = 1.618 \times 10^4 \\
\nu & := 0.8 \\
\nu & := 27922.17 & \nu & = 2.792 \times 10^4 
\end{align*}
\]
\[
\begin{align*}
I & = \pi \frac{D^4 - (D - 2t)^4}{64} \\
& = 8.071 \times 10^9 \\
\text{area} & = \frac{\pi}{4} \left[ D^2 - (D - 2t)^2 \right] \\
& = 5.866 \times 10^4 \\
r & = \frac{1}{\sqrt{\text{area}}} \\
& = 370.931 \\
\frac{k_1}{r} & = 34.892
\end{align*}
\]

3.2.1 Axial tension, \(f_t \leq F_y\)

\[
F_t := 0.60 F_y \\
F_t = 207 \\
UR_{3.2.1.1} := \frac{f_t}{F_t} \\
UR_{3.2.1.1} = 0
\]

3.2.2 Axial Compression

3.2.2.a Column Buckling

The allowable axial compressive stress, \(F_a\), should be determined from the following AISC formula for members with \(D/t\) ratio equal to or less than 60:

\[
C_c := \left( 2 \cdot \frac{2 \cdot \pi \frac{E}{F_y}}{r} \right)^{1/2} \\
C_c = 109.614
\]

\[
F_a = \left[ \frac{\left( \frac{k_1}{r} \right)^2}{1 - 2 \cdot C_c^2} \right] F_y \\
\text{if } \frac{k_1}{r} < C_c
\]

\[
\left[ \frac{5}{3} \frac{\left( \frac{k_1}{r} \right)^2}{8 \cdot C_c} + \frac{12 \cdot n^2 \frac{E}{23 \cdot \frac{k_1}{r}}} {8 \cdot C_c^3} \right] \\
\text{if } \frac{k_1}{r} \geq C_c
\]

\[
F_a = 183.794
\]
3.2.2.b Local Buckling

1. Elastic Load Buckling Stress

\[ C_{\text{cr}} := 0.3 \]

\[ F_{xx} = 2 \cdot C_{\text{cr}} \cdot E \cdot \frac{L}{D} \]  
(3.2.2 - 3)

\[ F_{xx} = 2.102 \times 10^3 \]

2. Inelastic Local Buckling Stress

\[ \Gamma_{xc} := \begin{cases} 
\frac{1}{1.64 - 0.23 \left( \frac{D}{t} \right)^{1/4}} & \text{if} \left( \frac{D}{t} \right) \leq 60 \leq \frac{10340}{F_Y} \\
F_Y & \text{equal or less than} F_{xx} 
\end{cases} \]  
(3.2.2 - 4)

\[ F_{xc} = 345 \]

3.2.3 Bending

\[ F_b := \begin{cases} 
(0.75F_Y) & \text{if} \frac{D}{t} \leq \frac{10340}{F_Y} \\
\left( 0.84 \cdot 1.74 \frac{D}{E_t} F_Y \right) & \text{if} \frac{10340}{F_Y} < \frac{D}{t} \leq \frac{20680}{F_Y} \\
\left( 0.72 \cdot 0.58 \frac{D}{E_t} F_Y \right) & \text{if} \frac{20680}{F_Y} < \frac{D}{t} \leq 300 
\end{cases} \]  
(3.2.3 - 1a)

\[ F_b = 230.694 \]

Combined bending stress is given by, \[ f_b = (f_{by}^2 + f_{bz}^2)^{0.5} \]

\[ f_b = (f_{by}^2 + f_{bz}^2)^{0.5} \]

\[ f_b = 11.699 \]

\[ UR_{3.2.3} := \frac{f_b}{F_b} \]

\[ UR_{3.2.3} = 0.051 \]
3.2.4 Shear

3.2.4.a Beam Shear

The maximum beam shear, \( f_v \), for cylindrical member is:

\[
\frac{f_v}{0.5 \cdot \text{area}} = 0.952 \\
\Rightarrow f_v = 0.952 \quad (3.2.4 - 1)
\]

The allowable beam shear stress:

\[
F_v := 0.4 \cdot F_y \\
\Rightarrow F_v = 138 \quad (3.2.4 - 2)
\]

\[
\text{UR}_{3.2.4.a} := \frac{f_v}{F_v} \\
\Rightarrow \text{UR}_{3.2.4.a} = 6.899 \times 10^{-3}
\]

3.2.4.b Torsional Shear

Input the following data:

\[
M_t := 32827.6 \\
M_k = 3.283 \times 10^4 \quad \text{Torsional Moment}
\]

\[
I_p = \frac{\pi}{2} \left[ \frac{(D)}{2} \right]^4 - \left[ \frac{(D - 2t)}{2} \right] \quad I_p = 1.614 \times 10^{10} \quad \text{polar moment of inertia}
\]

The maximum torsional shear stress, \( F_{vt} \), for cylindrical members caused by torsion is:

\[
F_{vt} = M_t \cdot \frac{D}{I_p} \\
\Rightarrow F_{vt} = M_t \cdot \frac{D}{I_p} \quad (3.2.4 - 3)
\]

The allowable torsional shear stress:

\[
F_{vt} := 0.4 \cdot F_y \\
\Rightarrow F_{vt} = 138 \quad (3.2.4 - 4)
\]

\[
\text{UR}_{3.2.4.b} := \frac{f_{vt}}{F_{vt}} \\
\Rightarrow \text{UR}_{3.2.4.b} = 7.861 \times 10^{-6}
\]

3.2.5 Hydrostatic Pressure (Stiffened and Unstiffened Cylinders)

3.2.5.a Design Hydrostatic Head

Input the following data:

\[ \rho := 1025 \quad \text{density of the sea water which may be taken as 1025 kg/m}^3 \]

\[ g := 9.810 \quad \text{the acceleration due to gravity (m/s}^2) \]
\[ \text{the still water depth to the sea floor (m)} \]
\[ d := 110.000 \]
\[ H_w := 26 \quad \text{wave height (m)} \]
\[ T_p := 13.9 \quad \text{wave period (s)} \]
\[ z := -95.38 \quad \text{the depth of the member relative to still water level (measured positive upwards) (m)} \]
\[ L_{\text{wave}} := \frac{g T_p^2}{2 \pi} \quad L_{\text{wave}} = 301.661 \]
\[ H_z := -z + \frac{H_w}{2} \left[ \cosh \left( 2 \frac{\pi}{L_{\text{wave}}} (d + z) \right) \right] \]
\[ \cosh \left( 2 \frac{\pi}{L_{\text{wave}}} d \right) \]
\[ (3.25 - 3) \]
\[ H_z = 98.105 \]

Hydrostatic Pressure (N/mm²):
\[ p_0 := \frac{\rho g H_z}{1000000} \quad p_0 = 0.986 \]
\[ f_{\text{hp}} := p_0 \frac{D}{2t} \quad f_{\text{hp}} = 29.561 \]

3.2.5.b Hoop Buckling Stress

1. Elastic Hoop Buckling Stress

Length of cylinder between stiffening rings, diaphragms, or end connections:
\[ L_{\text{stiff}} := 161.78 \quad L_{\text{stiff}} = 1.618 \times 10^4 \quad (\text{Inpu}) \]

The geometric parameter, \( M \), is defined as:
\[ M := \frac{L_{\text{stiff}}}{D} \left( \frac{1}{2 \cdot \frac{D}{t}} \right)^2 \quad M = 166.031 \]

The critical hoop buckling coefficient \( C_h \) includes the effect of initial geometric imperfections within API Spec 2B tolerance limits.
\[ C_h := \begin{cases} 
0.44 \frac{t}{D} & \text{if } M \geq 1.6 \frac{D}{t} \\
0.44 \frac{t}{D} + 0.21 \left( \frac{D}{t} \right)^3 & \text{if } 0.825 \frac{D}{t} \leq M < 1.6 \frac{D}{t} \\
[0.736 \cdot (M - 0.636)] & \text{if } 3.5 \leq M \leq 0.825 \frac{D}{t} \\
[0.755 \cdot (M - 0.559)] & \text{if } 1.5 \leq M < 3.5 \\
0.8 & \text{if } M < 1.5
\end{cases} \]

\[ C_h = 7.342 \times 10^{-3} \]

Elastic hoop buckling stress:

\[ F_{he} := 2 \cdot C_h \cdot F \frac{t}{D} \quad F_{he} = 51.449 \]

2. Critical Hoop Buckling Stress

The material yield strength relative to the elastic hoop buckling stress determines whether elastic or inelastic hoop buckling occurs and the critical hoop buckling stress, \( F_{hc} \), is defined by the appropriate formula.

Elastic buckling (3.2.5-6)

\[
F_{hc} := \begin{cases} 
F_{he} & \text{if } F_{he} \leq 0.55 F_y \\
0.45 F_y + 0.18 F_{he} & \text{if } 0.55 F_y < F_{he} \leq 1.6 F_y \\
\frac{1.31 F_y}{1.15 + \left( \frac{F_{he}}{F_y} \right)} & \text{if } 1.6 F_y < F_{he} < 6.2 F_y \\
F_y & \text{if } F_{he} \geq 6.2 F_y
\end{cases}
\]

\[ F_{hc} = 51.449 \]

\[ F_{hp} := \min(F_{hc}, F_{he}) \quad F_{hp} = 51.449 \]

\[ UR_{3.2.5} = \frac{F_{hp}}{F_{hp}} \quad UR_{3.2.5} = 0.575 \]

3.3 Combined Stresses for Cylindrical Members
Sections 3.3.1 and 3.3.2 apply to overall member behavior while Sections 3.3.3 and 3.3.4 apply to local buckling.

3.3.1 Combined Axial Compression and Bending

3.3.1.a Cylindrical Members

\[ f_a = 29 \quad \text{Compression Stress due to axial loads} \]

\[ C_{mx} := 0.85 \quad \text{based on Section 3.3.1.d} \]

\[ C_{my} := 0.85 \quad \text{based on Section 3.3.1.d} \]

\[ k = 0.8 \]

\[ l_{by} := 1 \quad \text{Actual unbraced length in the plane of bending} \]

\[ l_{bz} := 1 \quad \text{Actual unbraced length in the plane of bending} \]

\[ r_y := r \quad \text{Corresponding radius of gyration} \]

\[ r_z := r \quad \text{Corresponding radius of gyration} \]

\[ F_{ey} := \frac{12 \pi^2 E}{23 \left( \frac{k}{l_{by}} \right)^2} \quad \text{Euler stress divided by a factor of Safety} \]

\[ F_{ez} := \frac{12 \pi^2 E}{23 \left( \frac{k}{l_{bz}} \right)^2} \quad \text{Euler stress divided by a factor of Safety} \]

\[ F_{ey} = 888.237 \quad F_{ez} = 888.237 \]

\[ UR_{3.3.1a} := \left( \frac{f_a}{F_a} + \sqrt{\frac{C_{my}^2 l_{by}^2}{F_b}} \right) \quad \text{if} \quad \frac{f_a}{F_a} \leq 0.15 \quad (3.3.1 - 3) \]

\[ UR_{3.3.1a} := \left( \frac{f_a}{F_a} + \sqrt{\left( \frac{C_{mx} l_{by}}{1 - \frac{f_a}{F_{ey}}} \right)^2 + \left( \frac{C_{my} l_{bz}}{1 - \frac{f_a}{F_{ez}}} \right)^2} \right) \quad (3.3.1 - 4) \]
UR_{3.3.1a} = 0.202

3.3.2 Combined Axial Tension and Bending

\[ UR_{3.3.2} = \frac{f_a}{0.6F_y} + \sqrt{\frac{f_{by}^2 + f_{bz}^2}{F_b}} \]

UR_{3.3.2} = 0.191

3.3.3 Axial Tension and Hydrostatic Pressure

\[ SF_x = 1.67 \]  
\[ SF_h = 2 \]  
\[ f_b = 11.699 \]  
\[ f_h = 29.463 \]  
\[ \nu = 0.3 \]  
\[ AA := \frac{f_a + f_b - 0.5f_h}{F_y} SF_x \]

\[ AA = 0.126 \]

\[ BB := \frac{f_h}{F_{hc}} SF_h \]

\[ BB = 1.145 \]

\[ UR_{3.3.3.1} := \begin{cases} 0 & \text{if } f_t > 0 \\ \left( AA^2 + BB^2 + 2\nu |AA| BB \right) & \text{otherwise} \end{cases} \]  

(3.3.3 - 1)

\[ UR_{3.3.3.1} = 0 \]

3.3.4 Axial Compression and Hydrostatic Pressure

\[ f_c := f_a + f_b + 0.5f_h \]

\[ f_c = 55.431 \quad f_a = 29 \quad f_b = 11.699 \quad f_h = 29.463 \]

\[ F_{aa} := \frac{F_{xe}}{SF_x} \]

\[ F_{aa} = 1.259 \times 10^3 \]

\[ F_{ha} := \frac{F_{he}}{SF_h} \]

\[ F_{ha} = 25.724 \]
\[ SF_b := \frac{f_y}{f_b} \quad \text{SF}_b = 1.495 \]

\[ UR_{3.3.4.1} := \frac{f_a + 0.5f_h}{F_{sc}} \cdot SF_x + \frac{f_b}{f_y} \cdot SF_b \quad \text{UR}_{3.3.4.1} = 0.262 \]

\[ UR_{3.3.4.2} := \frac{SF_h f_h}{F_{hc}} \quad \text{UR}_{3.3.4.2} = 1.145 \]

\[ UR_{3.3.4.3} := \frac{f_x - 0.5F_{ha}}{F_{aa} - 0.5F_{ha}} + \left( \frac{f_h}{F_{ha}} \right)^2 \quad \text{UR}_{3.3.4.3} = 1.346 \quad \text{when} \ f_x > 0.5F_{ha} \]

If \( f_b > f_a + 0.5f_h \), both \( UR_{3.3.3.1} \) and \( UR_{3.3.4.1} \) must be less than 1.
4.3 Simple Joints

4.3.1 Validity Range

\[ 0.2 \leq \beta \leq 1.0 \]
\[ 10 \leq \gamma \leq 50 \]
\[ 30^0 \leq \theta \leq 90^0 \]
\[ F_y \leq 72\text{ksi} \]

\[ \frac{g_b}{D} > -0.6 \text{ (for K Joints)} \]

**INPUT: Basic Geometric Parameters for Simple Tubular Joints**

\[ F_y := 345 \]
\[ F_{yb} := 345 \]

\[ \theta := 49.6^0 \text{ Brace included angle} \]
\[ g_b := 1058 \text{ Gap between braces} \]
\[ t := 22.2 \text{ Brace wall thickness at intersection} \]
\[ T_c := 53.3 \text{ Chord wall thickness at intersection} \]
\[ d := 853 \text{ Brace outside diameter} \]
\[ D := 2560 \text{ Chord outside diameter} \]

\[ A_c = \frac{\pi}{4} \left[ D^2 - (D - 2T_c)^2 \right] = 4.197 \times 10^5 \text{ Chord cross sectional area} \]

\[ \beta := \frac{d}{D} \quad \beta = 0.333 \]

\[ \gamma := \frac{D}{2T_c} \quad \gamma = 24.015 \]

\[ \tau := \frac{t}{T_c} \quad \tau = 0.417 \]

\[ \phi := \frac{tF_{yb}}{T_cF_y} \quad \phi = 0.417 \]

\[ \frac{g_b}{D} = 0.413 \]
4.3.3. Strength Factor $Q_u$

K joint only with positive gap

$$Q_{g1} = \begin{cases} 1 + 0.2 \left( 1 - 2.8 \frac{g_b}{D} \right)^3 & \text{if } \frac{g_b}{D} > 0.05 = 0.999 \\ \text{"gap too small" otherwise} & \end{cases}$$

$$Q_g = \max(1, Q_{g1})$$

$$Q_g = 1$$

$Q_u$ varies with the joint and load type, as given in Table 4.3-1.

- $Q_u$ for Axial Loads

$$Q_{u\_a1} := (16 + 1.2 \gamma) \beta^{1.2} Q_g$$

$$Q_{u\_a} = \min\left( Q_{u\_a1}, 40 \beta^{1.2} Q_g \right)$$

$$\beta = 0.333$$

$$Q_{u\_a} = 10.698$$

- $Q_u$ for In-Plane Bending

$$Q_{u\_ipb} := (5 + 0.7 \gamma) \beta^{1.2}$$

$$Q_{u\_ipb} = 5.833$$

- $Q_u$ for Out-Of-Plane Bending

$$Q_{u\_opb} := 2.5 + (4.5 + 0.2 \gamma) \beta^{2.6}$$

$$Q_{u\_opb} = 3.034$$

4.3.4 Chord Load Factor $Q_i$

$Q_i$ is a factor to account for the presence of nominal loads in the chord.

$$Z_p = \frac{1}{6} \left[ D^3 - (D - 2T_c)^3 \right]$$

Plastic section modulus
\[ M_p := F_Y Z_p = 1.156 \times 10^{11} \] Plastic moment capacity of the chord

\[ P_y := F_Y A_c = 1.448 \times 10^8 \] Yield axial capacity of the chord

\[ FS := 1.6 \] Safety factor

\[ P_c := -1452340 \] Chord axial force

\[ M_{c_ipb} = 247281000 \]

\[ M_{c_opb} = -75653000 \]

\[ M_c := \sqrt{M_{c_ipb}^2 + M_{c_opb}^2} = 2.474 \times 10^9 \]

\[ AA := \left[ \left( \frac{FS \cdot P_c}{P_y} \right) + \left( \frac{FS \cdot M_c}{M_p} \right) \right]^{0.5} \quad (4.3 - 3) \]

\[ AA = 0.038 \]

**INPUT the following coefficients:**

Coefficients depending on joint and load type as given in Table 4.3-2.

\[ C_{1_a} := 0.2 \]
\[ C_{2_a} = 0.2 \]
\[ C_{3_a} = 0.3 \]

\[ Q_{c_a} = \left[ 1 + C_{1_a} \left( \frac{FS \cdot P_c}{P_y} \right) - C_{2_a} \left( \frac{FS \cdot M_{c_ipb}}{M_p} \right) - C_{3_a} AA \right] \quad (4.3 - 2) \]

\[ Q_{c_a} = 0.989 \]

\[ C_{1_b} = 0.2 \]
\[ C_{2_b} := 0 \]
\[ C_{3_b} = 0.4 \]

\[ Q_{c_ipb} = \left[ 1 + C_{1_b} \left( \frac{FS \cdot P_c}{P_y} \right) - C_{2_b} \left( \frac{FS \cdot M_{c_ipb}}{M_p} \right) - C_{3_b} AA \right] \]

\[ Q_{c_ipb} = 0.996 \]
\[ Q_{f_{opb}} = \left[ 1 + C_{1_b} \left( \frac{F_S \cdot P_c}{P_y} \right) - C_{2_b} \left( \frac{F_S \cdot M_{c_{opb}}}{M_p} \right) - C_{3_b} \cdot A \right]^2 \]

\[ Q_{f_{opb}} = 0.996 \]

### 4.3.2 Basic Capacity

Tubular joints without overlap of principal braces and having no gussets, diaphragms, grout or stiffeners should be designed using the following guidelines.

\[ P_a = Q_{u_{a_{-}}} Q_{c_{a}} \frac{F_y \cdot T_c^2}{F_S \cdot \sin(\theta)} \quad \text{allowable capacity for brace axial load} \]

\[ P_a = 8.515 \times 10^6 \]

\[ M_{a_{ipb}} = Q_{u_{ipb}} Q_{c_{ipb}} \frac{F_y \cdot T_c^2 \cdot d}{F_S \cdot \sin(\theta)} \quad \text{allowable capacity for brace BM} \]

\[ M_{a_{ipb}} = 3.987 \times 10^6 \]

\[ M_{a_{opb}} = Q_{u_{opb}} Q_{c_{opb}} \frac{F_y \cdot T_c^2 \cdot d}{F_S \cdot \sin(\theta)} \quad \text{allowable capacity for brace BM} \]

\[ M_{a_{opb}} = 2.074 \times 10^6 \]

### 4.3.6 Strength Check

\[ P = 4720645 \]

\[ M_{ipb} = 194146000 \]

\[ M_{opb} = -56799000 \]

\[ IR = \left( \frac{P}{P_a} \right)^2 + \left( \frac{M_{ipb}}{M_{a_{ipb}}} \right)^2 + \left( \frac{M_{opb}}{M_{a_{opb}}} \right) \]

IR = 0.584

\[ \left( \frac{P}{P_a} \right) = 0.554 \quad \left( \frac{M_{ipb}}{M_{a_{ipb}}} \right)^2 = 2.371 \times 10^{-3} \quad \left( \frac{M_{opb}}{M_{a_{opb}}} \right) = 0.027 \]
A.2 MATHCAD – ISO 19902
ISO19902
Axial Tension, Bending and Hoop Buckling
Member No. 533, LC752_8, Pos. 1.0

INPUT the following data:
Diameter, D (mm)
Wall thickness, t (mm)
Yield strength, σ_y (N/mm²)
Young's Modulus, E (N/mm²)
Axial tension stress due to factored loads, σ_t (N/mm²)
Axial compression stress due to factored loads, σ_c (N/mm²)
Bending stress due to factored loads about y axis, f_y (N/mm²)
Bending stress due to factored loads about z axis, f_z (N/mm²)
Hoop stress due to factored loads, f_h (N/mm²)
Unbraced length of member for local y axis, L_y (mm)
Unbraced length of member for local z axis, L_z (mm)
Radius of gyration, r (mm)

\[
\begin{align*}
D &= 1066.8 \\
t &= 17.8 \\
f_y &= 345 \\
E &= 210000 \\
\sigma_c &= 22.4 \\
\sigma_t &= 0 \\
\sigma_{by} &= 5.1 \\
\sigma_{bz} &= 11.8 \\
L_y &= 16178
\end{align*}
\]

\[
\begin{align*}
D &= 1.067 \times 10^3 \\
t &= 17.8 \\
f_y &= 345 \\
E &= 2.1 \times 10^5 \\
\sigma_c &= 22.4 \\
\sigma_t &= 0 \\
\sigma_{by} &= 5.1 \\
\sigma_{bz} &= 11.8 \\
L_y &= 1.618 \times 10^4
\end{align*}
\]
\[ L_z := 16178 \quad L_z = 1.618 \times 10^4 \text{ mm} \]

Combined bending stress is given by, \( \sigma_b = (\sigma_{by}^2 + \sigma_{bz}^2)^{0.5} \)

\[ \sigma_b := \left( \sigma_{by}^2 + \sigma_{bz}^2 \right)^{0.5} \quad \sigma_b = 12.855 \quad \frac{N}{\text{mm}^2} \]

13.2.2 Axial tension, \( \sigma_t := \frac{f_t}{Y_{Rt}} \) (13.2-1)

\[ \gamma_{Rt} := 1.05 \]

\[ f_t := f_y \]

\[ UR_{13.2.2} := \frac{\sigma_t}{f_t} \quad UR_{13.2.2} = 0 \]

13.2.3.3 Local buckling

\[ C_x := 0.3 \quad \text{elastic critical buckling coefficient - account for the effect of initial geometric imperfections within the tolerance limits given in Clause 21.} \]

\[ f_{xe} := 2 \cdot C_x \cdot E \cdot \frac{t}{D} \quad f_{xe} = 2.102 \times 10^3 \] (13.2 - 10)

\[ f_{yc} := \begin{cases} f_y & \text{if} \quad \frac{f_y}{f_{xe}} \leq 0.17 \\ \left[ (1.047 - 0.274 \cdot \frac{f_y}{f_{xe}}) \cdot f_y \right] & \text{if} \quad \frac{f_y}{f_{xe}} > 0.17 \end{cases} \] (13.2 - 9)

\[ f_{yc} = 345 \]

13.2.3.2 Column buckling

INPUT following parameter:

\[ k := 0.7 \quad \text{effective length factor, in y or z direction, see 13.5} \]

\[ L_y = 1.618 \times 10^4 \]
\[ L_z = 1.618 \times 10^4 \]

\[
I := \frac{\pi}{64} \left[ D^4 - (D - 2t)^4 \right] \quad I = 8.071 \times 10^9
\]

Area := \frac{\pi}{4} \left[ D^2 - (D - 2t)^2 \right] \quad \text{Area} = 5.866 \times 10^4

\[
r := \frac{I}{\text{Area}} \quad r = 370.931
\]

Column slenderness parameter:

\[
\lambda := \frac{kL_v}{\pi r} \sqrt{\frac{f_{yc}}{E}} \quad \lambda = 0.394 \quad 13.2 - 7
\]

\[
f_c := \begin{cases} 
(1.0 - 0.278\lambda^2) f_{yc} & \text{if } \lambda \leq 1.34 \\
0.9 f_{yc} & \text{if } \lambda > 1.34
\end{cases} \quad 13.2 - 5
\]

\[
f_c = 330.119
\]

13.2.3.1 General, \( \sigma_c \leq f_c / \gamma_{Re} \) (13.2-3)

\[
\sigma_c = 22.4
\]

\[
\gamma_{Re} = 1.18
\]

\[
\text{UR}_{13.2.4} := \frac{\sigma_c}{f_c} \quad \text{UR}_{13.2.4} = 0.08
\]

13.2.4 Bending, \( \sigma_b = M/Z_e \leq f_b / \gamma_{Rb} \) (13.2-11)

\[
Z_p := \frac{1}{6} \left[ D^3 - (D - 2t)^3 \right] \quad Z_p = 1.959 \times 10^7 \quad \text{plastic section modulus}
\]

\[
Z_e := \frac{\pi}{64} \left[ \frac{D^4 - (D - 2t)^4}{D} \right] \quad Z_e = 1.513 \times 10^7 \quad \text{elastic section modulus}
\]
\[ f_b := \begin{cases} \frac{Z_p f_y}{Z_e f_y} \text{ if } \frac{f_y D}{E t} \leq 0.0517 \\ \left[ 1.13 - 2.58 \frac{f_y D}{E t} \right] \frac{Z_p f_y}{Z_e f_y} \text{ if } 0.0517 < \frac{f_y D}{E t} \leq 0.1034 \\ \left( 0.94 - 0.76 \frac{f_y D}{E t} \right) \frac{Z_p f_y}{Z_e f_y} \text{ if } 0.1 < \frac{f_y D}{E t} \leq 120 f_y \end{cases} \] (13.2 - 13)

\[ f_b = 391.24 \]

\[ \gamma_{Rb} := 1.05 \quad \text{partial resistance factor for bending stress} \]

\[ \sigma_b = 12.855 \]

\[ UR_{13.2.11} := \frac{\sigma_b}{f_b} \]

\[ UR_{13.2.11} = 0.034 \]

13.2.5.1 Beam shear \[ \tau_b = 2 V/A \leq f_y/\gamma_{Rv} \]

\[ \gamma_{Rv} := 1.05 \]

\[ V_{\text{shear}} := 38529 \]

\[ f_v := \frac{f_y}{\sqrt{3}} \]

\[ f_v = 199.186 \]

\[ UR_{13.2.17} := \frac{2 \cdot V_{\text{shear}}}{\text{Area} / f_v} \]

\[ UR_{13.2.17} = 6.925 \times 10^{-3} \]

13.2.5.2 Torsional shear

\[ I_p = \frac{\pi}{32} \left[ D^4 - (D - 2 t)^4 \right] \]

\[ I_p = 1.614 \times 10^{10} \quad \text{polar moment of inertia} \]

\[ M_{vt} := 41438000 \quad \text{Torsional moment due to factored actions} \]
\[ UR_{13.2.19} := \frac{M_{vt}D}{2T_p} \frac{f_v}{\gamma_{Rv}} \]

\[ UR_{13.2.19} = 7.218 \times 10^{-3} \]

13.2.6 Hydrostatic pressure

\[ \gamma_{fG1} := 1.3 \quad \text{partial action factor for permanent actions 1, see Table 9.10.1} \]

\[ \rho_w := 1025 \quad \text{density of the sea water which may be taken as 1025 kg/m}^3 \]

\[ g_c := 9.810 \quad \text{the acceleration due to gravity (m/s}^2) \]

\[ d := 110.000 \quad \text{the still water depth to the sea floor (m)} \]

\[ H_w := 26 \quad \text{wave height (m)} \]

\[ T_p := 13.9 \quad \text{wave period (s)} \]

\[ z := -95.38 \quad \text{the depth of the member relative to still water level (measured positive upwards) (m)} \]

\[ L_{wave} := \frac{g_c T_p^2}{2 \cdot \pi} \quad L_{wave} = 301.661 \]

Effective hydrostatic head (m):

\[ H_z := -z + \frac{H_w}{2} \cosh \left[ \frac{2 \cdot \pi}{L_{wave}} (d + z) \right] \cosh \left( \frac{2 \pi}{L_{wave}} d \right) \]

\[ H_z = 98.105 \]

The factored hydrostatic pressure (p, N/mm\(^2\)) shall be calculated:

\[ p := \frac{\gamma_{fG1} g_c \rho_w H_z}{1000000} \]

\[ p = 1.282 \quad \frac{N}{\text{mm}^2} \quad (13.2 - 20) \]

13.2.6.2 Hoop buckling

Tubular members subjected to external pressure shall be designed to satisfy the following condition:
\[ \sigma_h = \frac{pD}{2t} \leq \frac{f_h}{\gamma_{Rh}} \quad (13.2.22) \]

\[ \gamma_{Rh} = 1.25 \]

Hoop stress due to the forces from factored hydrostatic pressure:

\[ \sigma_h = \frac{pD}{2t} \quad \sigma_h = 38.429 \quad \frac{N}{\text{mm}^2} \]

Geometric Parameter:

\[ L_z := 16178 \quad \text{the length of tubular between stiffening rings, diaphragms, or end connections} \]

\[ \mu := \frac{L_z}{D} \sqrt{\frac{2\cdot D}{t}} \]

Elastic critical hoop buckling coefficient \( C_h \):

\[ C_h = \begin{cases} 
0.44 - \frac{t}{D} & \text{if } \mu \geq 1.6 - \frac{D}{t} \\
0.44 - \frac{1}{D} + 0.21 \left( \frac{D}{t} \right)^3 \cdot \mu^4 & \text{if } 0.825 \cdot \frac{D}{t} \leq \mu < 1.6 - \frac{D}{t} \\
0.737 \cdot \mu - 0.579 & \text{if } 1.5 \leq \mu < 0.825 \cdot \frac{D}{t} \\
0.80 & \text{if } \mu < 1.5 
\end{cases} \quad (13.2.27) \]

\[ f_{he} := 2 \cdot C_h \cdot \frac{E}{D} \quad f_{he} = 51.449 \quad (13.2.26) \]

\( f_h \) the representative hoop buckling strength

For tubular members satisfying the out-of-roundness tolerance given in Annex G, \( f_h \) shall be determined from:

\[ f_h = \begin{cases} 
f_y & \text{if } f_{he} > 2.44 \cdot f_y \\
0.7 \left( \frac{f_{he}}{f_y} \right) \cdot f_y & \text{if } 0.55f_y < f_{he} \leq 2.44 \cdot f_y \\
f_{he} & \text{if } f_{he} \leq 0.55f_y 
\end{cases} \quad (13.2.23) \]

\[ (13.2.24) \]

\[ (13.2.25) \]
\[ f_h = 51.449 \]

The utilization of a member under external pressure shall be calculated:

\[ UR_{13.2.31} = \frac{\sigma_h}{f_h} = \frac{\gamma_R}{\gamma_{Rh}} \]  \hspace{1cm} (13.2 - 31)

\[ UR_{13.2.31} = 0.934 \]

13.3 Tubular members subjected to combined forces without hydrostatic pressure

13.3.2 Axial Tension and Bending

Tubular members subjected to combined axial tension and bending forces shall be designed to satisfy the following condition at all cross-sections along their length:

\[ UR_{13.3.2} = \frac{\gamma_{Rt}\cdot\sigma_t}{f_t} + \frac{\gamma_{Rb}\cdot\sqrt{\sigma_{by}^2 + \sigma_{bz}^2}}{f_b} \]  \hspace{1cm} (13.3 - 2)

\[ UR_{13.3.2} = 0.034 \]

13.3.3 Axial Compression and Bending

Tubular members subjected to combined axial compression and bending forces shall be designed to satisfy the following conditions at all cross-sections along their length:

**Euler buckling strengths corresponding to the member y- and z-axes, respectively, in stress units:**

\[ k_y = 0.7 \quad \sigma_{ny} = 0.85 \]

\[ k_z = 0.7 \quad \sigma_{nz} = 0.85 \]

\[ f_{ey} = \frac{\pi^2\cdot E}{\left(\frac{L_y}{k_y\cdot r}\right)^2} \quad f_{ey} = 2.224 \times 10^3 \]  \hspace{1cm} (13.3 - 5)

\[ f_{ez} = \frac{\pi^2\cdot E}{\left(\frac{L_z}{k_z\cdot r}\right)^2} \quad f_{ez} = 2.224 \times 10^3 \]  \hspace{1cm} (13.3 - 6)
\[ \text{UR}_{13.3.7} = \begin{cases} 0 & \text{if } p > 0 \\ \frac{\gamma R_c \sigma_c}{f_c} + \frac{\gamma R_b}{f_b} \left[ \frac{C_{my} \sigma_{by}}{1 - \frac{\sigma_c}{f_{ey}}} \right]^2 + \left( \frac{C_{mz} \sigma_{by}}{1 - \frac{\sigma_c}{f_{ez}}} \right)^2 & \text{if } p = 0 \end{cases} \]  

(13.3 - 7)

\[ \text{UR}_{13.3.8} = \begin{cases} 0 & \text{if } p > 0 \\ \frac{\gamma R_c \sigma_c}{f_c} + \frac{\gamma R_b \sqrt{\sigma_{by}^2 + \sigma_{bz}^2}}{f_b} & \text{if } p = 0 \end{cases} \]  

(13.3 - 8)

\[ U_{13.3.3} := \max(\text{UR}_{13.3.7}, \text{UR}_{13.3.8}) \quad U_{13.3.3} = 0 \]

13.4 Tubular members subjected to combined forces with hydrostatic pressure

13.4.1 General

\[ \sigma_q := 0.5 \cdot \sigma_h \]  

(13.4 - 4)  

Compressive axial stress due to the capped-end hydrostatic actions using the value of the pressure from Eq (13.2-20)

**INPUT the following parameters:**

- \( \sigma_{tc} := 0 \)  
  The axial tensile stress due to forces from factored actions

- \( \sigma_{cc} := 22.4 \)  
  The axial compressive stress due to forces from factored actions

\[ \sigma_{tc} := \begin{cases} (\sigma_q - \sigma_{tc}) & \text{if } \sigma_{tc} < \sigma_q \\ (\sigma_{cc} - \sigma_q) & \text{if } \sigma_{cc} > \sigma_q \end{cases} \]  

(13.4 - 5)

13.4.2 Axial Tension, Bending and Hydrostatic Pressure
\[ \eta = 5 - 4 \frac{f_h}{f_y} \]  \hspace{1cm} (13.4 - 11) \hspace{1cm} \eta = 4.403

\[ B = \frac{\gamma_{Rh} \sigma_h}{f_h} \]  \hspace{1cm} (13.4 - 10) \hspace{1cm} B = 0.934

The representative bending strength in the presence of external hydrostatic pressure:

\[ f_{bh} = f_y \left( \sqrt{1 + 0.09 \cdot B^2 - B^2 \eta - 0.3B} \right) \]  \hspace{1cm} (13.4 - 9) \hspace{1cm} f_{bh} = 175.796

The representative axial tensile strength in the presence of external hydrostatic pressure:

\[ f_{th} = f_y \left( \sqrt{1 + 0.09 \cdot B^2 - B^2 \eta - 0.3B} \right) \]  \hspace{1cm} (13.4 - 8) \hspace{1cm} f_{th} = 155.019

\[ UR_{13.4.12} := \frac{\gamma_{Re} \cdot \sigma_{tc}}{f_{th}} + \frac{\gamma_{Re} \cdot \sqrt{\sigma_{by}^2 + \sigma_{bz}^2}}{f_{bh}} \]  \hspace{1cm} (13.4 - 12)

\[ UR_{13.4.12} = 0.077 \]

### 13.4.3 Axial compression, bending and hydrostatic pressure

\[ \lambda = 0.394 \]

\[ \sigma_q = 19.215 \]

\[ \sigma_x := \sigma_b + \sigma_{cc} \]

\[ \sigma_x = 35.255 \] \hspace{1cm} Maximum combined compressive stress

\[ f_{xe} = 2.102 \times 10^3 \]

\[ f_{he} = 51.449 \]

If the maximum combined compressive stress, \( \sigma_x \), and the representative elastic local buckling strength, \( f_{xe} \), exceed the limits given below \( (\sigma_{13.4.17.1} > \sigma_{13.4.17.2}) \),

\[ \sigma_{13.4.17.1} := \frac{0.5 \cdot f_{he}}{\gamma_{Rh}} \]

\[ \sigma_{13.4.17.1} = 20.58 \]

\[ \sigma_{13.4.17.2} := \max \left( \sigma_x, \frac{f_{xe}}{\gamma_{Re}} \right) \]

\[ \sigma_{13.4.17.2} = 1.782 \times 10^3 \]

Then Equation (13.4-8) shall also be satisfied (less than or equal to 1)
\[
\sigma_x = \frac{0.5 f_{he}}{\gamma_{Rh}} + \left( \frac{\gamma_{Rh} \sigma_h}{f_{he}} \right)^2
\]

(13.4 - 18)

\[f_{ch} = \begin{cases} 
\frac{f_{yc}}{2} \left[ \frac{(1 - 0.278 \lambda^2)}{f_{yc}} - \frac{2 \sigma_q}{f_{yc}} \right] + \sqrt{\left(1 - 0.278 \lambda^2\right)^2 + 1.12 \frac{\lambda^2 \sigma_q}{f_{yc}}} & \text{if } \lambda \leq 1.34 \\
\frac{0.9 \lambda^2 f_{yc}}{f_{yc}} & \text{if } \lambda > 1.34 \\
\end{cases}
\]

\[f_{ch} = 311.775
\]

(13.4 - 19)

\[
\sigma_x = \frac{0.5 f_{he}}{\gamma_{Rh}} + \left( \frac{\gamma_{Rh} \sigma_h}{f_{he}} \right)^2
\]

(13.4 - 21)

\[U_{13.4.19} = 0.153 \quad U_{13.4.20} = 0.122 \quad U_{13.4.21} = 0.88 \]

\[U_m := \max(U_{13.4.19}, U_{13.4.20}, U_{13.4.21}) \quad U_m = 0.88
\]
14. Strength of tubular joints

14.3.1 General

Validity Range

\[ 0.2 \leq \beta \leq 1.0 \]

\[ 10 \leq \gamma \leq 50 \]

\[ 30^\circ \leq \theta \leq 90^\circ \]

\[ \tau \leq 1.0 \]

\[ \frac{F_y}{N} \leq 500 \text{ mm}^{-2} \]

\[ g_b T_c > -1.2 \gamma \quad \text{For K-joints} \]

INPUT: Basic Geometric Parameters for Simple Tubular Joints

\[ F_y := 345 \quad \text{Yield strength, chord} \]

\[ F_{yb} := 345 \quad \text{Yield strength, brace} \]

\[ \theta := 49.5^\circ \quad \text{Brace included angle} \]

\[ g_b := 1058 \quad \text{Gap between braces} \]

\[ t := 22.2 \quad \text{Brace wall thickness at intersection} \]

\[ T_c := 53.3 \quad \text{Chord wall thickness at intersection} \]

\[ d := 853 \quad \text{Brace outside diameter} \]

\[ D := 2560 \quad \text{Chord outside diameter} \]

\[ A_c := \frac{\pi}{4} \left[ D^2 - (D - 2T_c)^2 \right] = 4.197 \times 10^5 \quad \text{Chord cross sectional area} \]

\[ \beta := \frac{d}{D} \quad \beta = 0.333 \]

\[ \gamma := \frac{D}{2T_c} \quad \gamma = 24.015 \]

\[ \tau := \frac{t}{T_c} \quad \tau = 0.417 \]

\[ \phi := \frac{tF_{yb}}{T_cF_y} \]
14.3.3. Strength Factor $Q_u$

*Joint only with positive gap*

$$Q_{g_1} = \begin{cases} 
1.9 - 0.7\gamma - 0.5 \left( \frac{e_b}{T_c} \right)^{0.5} & \text{if } \frac{e_b}{T_c} \geq 2 \\
\text{"gap too small" otherwise} & 
\end{cases} \quad (14.3-7)$$

$$Q_g := \max(1, Q_{g_1})$$

$$Q_g = 1.264 \quad \text{Gap factor}$$

$$Q_\beta := \begin{cases} 
0.3 \frac{\beta}{(1 - 0.8333\beta)} & \text{if } \beta > 0.6 \\
1.0 & \text{otherwise} 
\end{cases} \quad (14.3-5) \quad (14.3-6)$$

$$Q_\beta = 1 \quad \text{Geometrical factor}$$

$Q_u$ varies with the joint and load type, as given in Table 14.3-1.

- **$Q_u$ for Axial Loads**
  
  $$Q_{u-a} := (1.9 + 19\beta)Q_\beta^{0.5}Q_g$$

  $$Q_{u-a} = 10.4$$

- **$Q_u$ for In-Plane Bending**
  
  $$Q_{u-ipb} := 4.5\beta \gamma^{0.5}$$

  $$Q_{u-ipb} = 7.348$$

- **$Q_u$ for Out-Of-Plane Bending**
  
  $$Q_{u-opb} := 3.2\gamma \left( \frac{e_b}{T_c} \right)^{0.5}$$

  $$Q_{u-opb} = 3.818$$
14.3.4 Chord Load Factor $Q_f$

$Q_f$ is a factor to account for the presence of nominal loads in the chord.

$$Z_p := \frac{1}{6} \left[ D^3 - (D - 2T_c)^3 \right]$$  
Plastic section modulus

$$M_p := F_y Z_p = 1.156 \times 10^{11}$$  
Plastic moment capacity of the chord

$$P_y := F_y A_c = 1.448 \times 10^8$$  
Yield axial capacity of the chord

$$P_c := -13396500$$  
Chord axial force

$$M_{c_{ipb}} := -1800220000$$  
Chord bending moment, in-plane bending

$$M_{c_{opb}} := 125749000$$  
Chord bending moment, out-of-plane bending

$$\lambda_a := 0.03$$

$$\lambda_{ipb} := 0.045$$

$$\lambda_{opb} := 0.021$$

$$\gamma_{Rq} := 1.05$$  
Partial resistance factor for yield strength

**INPUT the following coefficients:**

Coefficients depending on joint and load type as given in Table 14.3-2.

$$C_{1,a} = 14$$

$$C_{2,a} = 43$$

$$q_{A,a} := \left[ C_{1,a} \left( \frac{P_c}{P_y} \right)^2 + C_{2,a} \left( \frac{M_{c_{ipb}}}{M_p} \right)^2 + C_{2,a} \left( \frac{M_{c_{opb}}}{M_p} \right)^2 \right]^{0.5} \cdot \gamma_{Rq} \quad (14.3-10)$$

$$q_{A,a} = 0.379$$

$$Q_{f,a} := \left( 1 - \lambda_a q_{A,a}^2 \right) \quad (14.3-9)$$

$$Q_{c,a} = 0.996$$  
Chord force factor, axial force
\[ C_{1_b} = 25 \]
\[ C_{2_b} = 43 \]

\[ q_{A_b} = \left[ \frac{C_{1_b}}{P_c} \left( \frac{P_c}{P_y} \right)^2 + C_{2_b} \left( \frac{M_{c_ipb}}{M_p} \right)^2 + C_{2_b} \left( \frac{M_{c_opb}}{M_p} \right)^2 \right]^{0.5} \gamma_{Rq} \quad (14.3-10) \]

\[ q_{A_b} = 0.497 \]

\[ Q_{r_ipb} = 1.0 - \lambda_{ipb} q_{A_b}^2 \quad (14.3-9) \]

\[ Q_{r_ipb} = 0.989 \quad \text{Chord force factor, in-plane bending} \]

\[ Q_{r_opb} = 1.0 - \lambda_{opb} q_{A_b}^2 \quad (14.3-9) \]

\[ Q_{r_opb} = 0.985 \quad \text{Chord force factor, out-of-plane bending} \]

### 14.3.2 Basic joint strength

Tubular joints without overlap of principal braces and having no gussets, diaphragms, grout or stiffeners should be designed using the following guidelines.

\[ P_{uj} = \frac{F_y T_c}{\sin(\theta)} Q_{u,a} Q_{f,a} \quad \text{Representative joint axial strength} \quad (14.3-1) \]

\[ P_{uj} = 1.333 \times 10^7 \]

\[ M_{uj_ipb} = \frac{F_y T_c}{\sin(\theta)} Q_{u_ipb} Q_{r_ipb} \quad \text{Representative joint moment strength, in-plane bending} \quad (14.3-2) \]

\[ M_{uj_ipb} = 7.977 \times 10^9 \]

\[ M_{uj_opb} = \frac{F_y T_c}{\sin(\theta)} Q_{u_opb} Q_{f_opb} \quad \text{Representative joint moment strength, out-of-plane bending} \quad (14.3-2) \]

\[ M_{uj_opb} = 4.169 \times 10^9 \]
\[ \gamma_{R_j} = 1.05 \]  
Partial resistance factor for tubular joints

\[ P_d = \frac{P_{uj}}{\gamma_{R_j}} = 1.269 \times 10^7 \]  
Joint axial design strength

\[ M_{d\_ipb} = \frac{M_{uj\_ipb}}{\gamma_{R_j}} = 7.597 \times 10^9 \]  
Design joint bending moment strength, in-plane bending

\[ M_{d\_opb} = \frac{M_{uj\_opb}}{\gamma_{R_j}} = 3.971 \times 10^9 \]  
Design joint bending moment strength, out-of-plane bending

### 14.3.6 Strength Check

\[ P_B = -7979041 \]  
Axial force in brace

\[ M_{B\_ipb} = -154001000 \]  
Bending moment in brace, in-plane bending

\[ M_{B\_opb} = 99443000 \]  
Bending moment in brace, out-of-plane bending

\[ U_j = \left| \frac{P_B}{P_d} \right| + \left( \frac{M_{B\_ipb}}{M_{d\_ipb}} \right)^2 + \left| \frac{M_{B\_opb}}{M_{d\_opb}} \right| \]

\[ U_j = 0.654 \]
A.3 MATHCAD – NORSOK N004
NORSOK N-004 Design of Steel Structures

6.3 Tubular Members

INPUT the following data:

Diameter, D (mm)
Wall thickness, t (mm)
Yield strength, $\sigma_y$ (N/mm$^2$)
Young's Modulus, $E$ (N/mm$^2$)
Design Axial Tension Forces due to factored loads, $N_{sa}$ (N)
Design Axial compression Forces due to factored loads, $N_{sdc}$ (N)
Design Bending Moment due to factored loads about y axis, $M_{ysa}$ (N-mm)
Design Bending Moment due to factored loads about z axis, $M_{zs}$ (N-mm)
Design Shear Force $V_{sa}$ (N)
Design Hydrostatic Pressure, $p_{sa}$ (N/mm$^2$)
Hoop stress due to factored loads, $f_y$ (N/mm$^2$)
Unbraced length of member for local y axis, $L_y$ (mm)
Unbraced length of member for local z axis, $L_z$ (mm)
Radius of gyration, $r$ (mm)

$D := 1066.8$ mm
$t := 17.8$ mm
$f_y := 345$ N/mm$^2$
$E := 210000$ N/mm$^2$
$N_{sdc} := 2330506$ N
$N_{sdt} := 0$ N
$M_{ysa} := \frac{38764000}{N - mm}$ N - mm
\[ M_{2sd} := 230831000 \quad N - \text{mm} \]
\[ V_{2sd} = 39479 \quad N \]
\[ P_{2sd} := 0.985 \quad \frac{N}{\text{mm}^2} \]
\[ l_y := 16178 \quad \text{mm} \]
\[ l_z := 16178 \quad \text{mm} \]
\[ \frac{D}{t} = 59.933 \]

6.3.2 Axial tension

\[ \text{Area} = \frac{\pi}{4} \left[ D^2 - (D - 2t)^2 \right] \]
\[ \gamma_{Mt} := 1.15 \quad \text{Cross Section Area} \]
\[ \text{Area} = 5.866 \times 10^4 \]
\[ N_{ir} := \frac{\text{Area} f_y}{\gamma_{Mt}} \]
\[ N_{ir} = 1.76 \times 10^7 \]

\[ \mathbf{UR}_{6.1} := \frac{N_{std}}{N_{ir}} \quad (6.1) \]

\[ \mathbf{UR}_{6.1} = 0 \]

6.3.3 Axial Compression

INPUT the following parameters:

\[ k := .7 \quad \text{See Table 6-2} \]
\[ L_t := 16178 \quad \text{Length of tubular between stiffening rings, diaphragms, or end connections} \]
\[ C_e := 0.3 \quad \text{critical elastic buckling coefficient} \]
\[ I := \frac{\pi}{64} \left[ D^4 - (D - 2t)^4 \right] \quad \text{Moment of inertia} \]
\[ I = 8.071 \times 10^9 \quad \text{mm}^4 \]
\[ i := \sqrt{\frac{1}{\text{Area}}} \quad \text{radius of gyration} \]
\[ i = 370.931 \quad \text{mm} \]
\[ \varepsilon_{ele} := 2C_e E \frac{i}{D} \quad \text{characteristic elastic local buckling strength} \]
\[ f'_{\text{cle}} = 2.102 \times 10^3 \]

The characteristic local buckling strength should be determined from:

\[
f'_{\text{cl}} = \begin{cases} 
  f_y & \text{if } \frac{f_y}{f'_{\text{cle}}} \leq 0.170 \\
  \left[1.047 - 0.274 \cdot \frac{f_y}{f'_{\text{cle}}} \right] f_y & \text{if } 0.170 < \frac{f_y}{f'_{\text{cle}}} \leq 1.911 \\
  f'_{\text{cle}} & \text{if } \frac{f_y}{f'_{\text{cle}}} > 1.911 
\end{cases} \]  

(6.6)

(6.7)

(6.8)

\[ f'_{\text{cl}} = 345 \]

\[
\lambda_y := \frac{k \cdot l_y}{\pi i} \sqrt{\frac{f'_{\text{cl}}}{E}} \quad \text{column slenderness parameter} \]  

(6.5) \quad \lambda_y = 0.394

\[
\lambda_z := \frac{k \cdot l_z}{\pi i} \sqrt{\frac{f'_{\text{cl}}}{E}} \quad \text{column slenderness parameter} \]  

(6.5) \quad \lambda_z = 0.394

\[
f_{cy} := \begin{cases} 
  \left[1.0 - 0.28 \cdot \lambda_y^2 \right] \cdot f_y & \text{if } \lambda_y \leq 1.34 \\
  \left(0.9 \cdot \frac{f_y}{\lambda_y^2} \right) & \text{if } \lambda_y > 1.34 
\end{cases} \]  

(6.3)

\[
f_{cy} = 330.012 \]  

(6.4)

\[
f_{cz} := \begin{cases} 
  \left[1.0 - 0.28 \cdot \lambda_z^2 \right] \cdot f_y & \text{if } \lambda_z \leq 1.34 \\
  \left(0.9 \cdot \frac{f_y}{\lambda_z^2} \right) & \text{if } \lambda_z > 1.34 
\end{cases} \]  

(6.3)

\[
f_{cz} = 330.012 \]  

(6.4)

\[ f_c := \min(f_{cy}, f_{cz}) \quad f_c = 330.012 \]
Define the characteristic elastic hoop buckling strength (Clause 6.3.6.1):

Geometric parameter, \( \mu \):

\[
\mu := \frac{L}{D} \sqrt{\frac{2}{D}} \quad \mu = 166.031
\]

\[
C_h = \begin{cases} 
0.44 \frac{t}{D} & \text{if } \mu \geq 1.6 \frac{D}{t} \\
0.44 \frac{t}{D} + 0.21 \left( \frac{D}{t} \right)^3 & \text{if } 0.825 \frac{D}{t} \leq \mu < 1.6 \frac{D}{t} \\
0.737 & \text{if } 1.5 \leq \mu < 0.825 \frac{D}{t} \\
\mu - 0.579 & \text{if } \mu < 1.5
\end{cases}
\]

\[C_h = 7.342 \times 10^{-3}\]

Then elastic hoop buckling strength, \( f_{he} \), is determined:

\[f_{he} = 2 \cdot C_h \cdot E \cdot \frac{t}{D} \quad (6.20) \quad f_{he} = 51.449\]

Characteristic elastic buckling strength:

\[f_h := \begin{cases} 
f_y & \text{if } f_{he} > 2.44 f_y \\
0.7 f_y \left( \frac{f_{he}}{f_y} \right)^{0.4} & \text{if } 2.44 f_y \geq f_{he} > 0.55 f_y \\
f_{he} & \text{if } f_{he} \leq 0.55 f_y
\end{cases} \quad (6.17) \quad f_h = 51.449\]

Define Material Factor (Clause 6.3.7)

\[W_e := \frac{2 I}{D} \quad \text{Elastic Section Modulus} \quad W_e = 1.513 \times 10^7\]

\[N_{sd} := N_{Sdc} \quad N_{sd} = 2.331 \times 10^6 \quad N_{sd} \text{ is negative if in tension}\]

\[
\sigma_{cSd} := \frac{N_{sd}}{\text{Area}} + \sqrt{\frac{M_{ySd}^2 + M_{zSd}^2}{W_e}} \quad (6.25) \quad \sigma_{cSd} = 56.074
\]
\[
\sigma_{p sd} = \frac{P_{sd}D}{2t}\quad (6.16) \quad \sigma_{p sd} = 29.517
\]

\[
\lambda_c := \frac{f_y}{f_{cle}} \quad \text{and} \quad \lambda_h := \frac{f_y}{f_{he}} \quad (6.24) \quad \lambda_c = 0.405 \quad \lambda_h = 2.59
\]

\[
\lambda_s := \frac{\sigma_{c Sd}}{f_{cl}} \cdot \lambda_c + \left(\frac{\sigma_{p sd}}{f_{sh}}\right)^2 \cdot \lambda_h \quad (6.23) \quad \lambda_s = 0.918
\]

\[
\gamma_M := \begin{cases} 
1.15 & \text{if } \lambda_s < 0.5 \\
(0.85 + 0.60 \cdot \lambda_s) & \text{if } 0.5 \leq \lambda_s \leq 1.0 \\
1.45 & \text{if } \lambda_s > 1.0 
\end{cases} \quad (6.22)
\]

\[
N_{c Rd} = \frac{\text{Area} \cdot f_c}{\gamma_M} \quad N_{c Rd} = 1.382 \times 10^7
\]

\[
UR_{6.2} := \frac{N_{sd}}{N_{c Rd}} \quad (6.2) \quad UR_{6.2} = 0.169
\]

### 6.3.4 Bending

\[
Z := \frac{1}{6} \left[D^3 - (D - 2t)^3\right] \quad \text{plastic section modulus}
\]

\[
M_{sd} := \sqrt{M_{y Sd}^2 + M_{z Sd}^2} \quad \text{design bending moment}
\]

Characteristic bending strength:

\[
f_m := \begin{cases} 
\frac{Z}{W_e} f_y & \text{if } \frac{f_y D}{E t} \leq 0.0517 \\
\left[1.13 - 2.58 \frac{f_y D}{E t}, \frac{Z}{W_e} f_y\right] & \text{if } 0.0517 < \frac{f_y D}{E t} \leq 0.1034 \\
\left[0.94 - 0.76 \frac{f_y D}{E t}, \frac{Z}{W_e} f_y\right] & \text{if } 0.1034 < \frac{f_y D}{E t} \leq 120 \frac{f_y}{E} 
\end{cases} \quad (6.10, 6.11, 6.12)
\]
\[ f_m = 391.24 \quad \gamma_M = 1.401 \]

\[ M_{Rd} := \frac{f_m W_e}{\gamma_M} \quad M_{Rd} = 4.226 \times 10^9 \quad \text{Moment Resistance} \]

\[ UR_{6.9} := \frac{M_{sd}}{M_{Rd}} \quad UR_{6.9} = 0.019 \]

### 6.3.5 Shear

\[ V_{sd} = 3.948 \times 10^4 \]

\[ \gamma_{Ms} := 1.15 \]

\[ V_{Rd} := \frac{\text{Area} \cdot f_y}{2 \sqrt{3} \cdot \gamma_{Ms}} \quad V_{Rd} = 5.08 \times 10^6 \quad \text{Shear Resistance} \]

\[ UR_{6.13} := \frac{V_{sd}}{V_{Rd}} \quad UR_{6.13} = 7.771 \times 10^{-3} \]

\[ M_{Ts} := 4.53 \times 10000 \]

\[ I_p := 2 \]

\[ M_{TRd} := \frac{2 \cdot I_p \cdot f_y}{D \sqrt{3} \cdot \gamma_{Ms}} \quad M_{TRd} = 5.242 \times 10^9 \]

\[ UR_{6.14} := \frac{M_{Ts}}{M_{TRd}} \quad UR_{6.14} = 8.642 \times 10^{-3} \]

### 6.3.6.1 Hoop Buckling

\[ \sigma_{psd} = 29.517 \]

\[ f_h = 51.449 \]

\[ f_{hRd} := \frac{f_h}{\gamma_M} \quad \text{Hoop Stress Resistance} \]

\[ UR_{6.15} := \frac{\sigma_{psd}}{f_{hRd}} \quad UR_{6.15} = 0.804 \]
6.3.8 Tubular members subjected to combined loads without hydrostatic pressure

6.3.8.1 Axial Tension and Bending

\[ N_{Sdt} = 0 \quad M_{ySd} = 8.879 \times 10^7 \quad M_{zSd} = 2.308 \times 10^8 \]

\[ N_{tRd} = 1.76 \times 10^7 \quad M_{Rd} = 4.226 \times 10^9 \]

\[
U_{Rd}^{6.26} := \begin{cases} 
0 & \text{if } p_{Sd} > 0 \\
\left[ \frac{N_{Sdt}}{N_{tRd}} \right]^{1.75} + \frac{M_{ySd}^2 + M_{zSd}^2}{M_{Rd}} & \text{if } p_{Sd} = 0
\end{cases}
\]

\[ U_{Rd}^{6.26} = 0 \]

6.3.8.2 Axial Compression and Bending

**INPUT** the following parameters:

\[ C_{my} = 0.85 \quad C_{mz} = 0.85 \quad \text{reduction factors corresponding to the member y and z axes} \]

\[ N_{ey} := \frac{\pi^2 \cdot E \cdot \text{Area}}{\left( \frac{k \cdot l_y}{r} \right)^2} \quad N_{ey} = 1.304 \times 10^8 \]

\[ N_{ez} := \frac{\pi^2 \cdot E \cdot \text{Area}}{\left( \frac{k \cdot l_z}{r} \right)^2} \quad N_{ez} = 1.304 \times 10^8 \]

\[ N_{cIRd} := \frac{f_{c1} \cdot \text{Area}}{\gamma_M} \quad \text{Design axial local buckling resistance} \]
\[ N_{cIRd} = 1.445 \times 10^7 \]

\[
UR_{6.27} := \begin{cases} 0 & \text{if } P_{Sd} > 0 \\ \left( \frac{N_{sd}}{N_{cIRd}} + \frac{1}{M_{RD}} \left( \frac{C_{my} M_{ySd}}{N_{sd}} \right)^2 + \left( \frac{C_{mz} M_{zSd}}{N_{sd}} \right)^2 \right)^{0.5} & \text{if } P_{Sd} = 0 \\
\end{cases} \tag{6.27}
\]

\[
UR_{6.28} := \begin{cases} 0 & \text{if } P_{Sd} > 0 \\ \left( \frac{N_{sd}}{N_{cIRd}} + \left( \frac{M_{ySd}}{M_{RD}} \right)^2 + \frac{M_{zSd}}{M_{RD}} \right) & \text{if } P_{Sd} = 0 \\
\end{cases} \tag{6.28}
\]

UR_{6.27} = 0 \quad \text{and} \quad UR_{6.28} = 0

6.3.8.3 Interaction Shear and Bending Moment

\[
\frac{V_{Sd}}{V_{Rd}} = 7.771 \times 10^{-3}
\]

\[
UR_{6.3.8.3} := \begin{cases} \frac{M_{Sd}}{M_{RD}} & \text{if } \frac{V_{Sd}}{V_{Rd}} \geq 0.4 \\ \sqrt{1.4 - \left( \frac{V_{Sd}}{V_{Rd}} \right)} \frac{M_{Sd}}{M_{RD}} & \text{if } \frac{V_{Sd}}{V_{Rd}} < 0.4 \\
\end{cases} \tag{6.31}
\]

\[
UR_{6.3.8.3} = 0.059
\]

6.3.8.4 Interaction Shear, Bending Moment and Torsional Moment

\[ M_{TSd} = 4.53 \times 10^7 \]
\[ \gamma_M = 1.401 \]

\[ \text{Radius} = \frac{D}{2} \quad \text{Radius} = 533.4 \]

\[ \bar{f}_d := \frac{f_y}{\gamma_M} \quad \bar{f}_d = 246.27 \]

\[ \tau_{Ts d} := \frac{M_{Ts d}}{2 \pi \text{Radius}^2 t} \quad \tau_{Ts d} = 1.424 \]

\[ f_{mRed} := f_m \sqrt{1 - \left( \frac{\tau_{Ts d}}{\bar{f}_d} \right)^2} \quad f_{mRed} = 391.22 \]

\[ M_{RedRd} := \frac{W_e f_{mRed}}{\gamma_M} \]

\[ \text{UR}_{6.33} := \begin{cases} \frac{M_{sd}}{M_{RedRd}} & \text{if} \quad \frac{V_{sd}}{V_{Rd}} \geq 0.4 \\ \sqrt{1.4 - \frac{V_{sd}}{V_{Rd}}} & \text{if} \quad \frac{V_{sd}}{V_{Rd}} < 0.4 \\ \frac{M_{sd}}{M_{RedRd}} & \end{cases} \]

\[ \text{UR}_{6.33} = 0.059 \]

### 6.3.9 Tubular Members Subjected to Combined Loads with Hydrostatic Pressure

#### 6.3.9.1 Axial Tension, Bending, and Hydrostatic Pressure

**INPUT the following data:**

\[ \sigma_{sd} := -39.7 \quad \text{design axial stress that excludes the effect of capped-end axial compression arising from the external hydrostatic pressure (tension positive)} \]

\[ \sigma_{qd} := 0.5 \sigma_{psd} \quad \text{capped-end design axial compression stress due to external hydrostatic pressure (compression positive)} \]

\[ \sigma_{my sd} := 5.9 \quad \text{design in plane bending stress} \]
\[ \sigma_{mSd} = 15.2 \quad \text{design out-of-plane bending stress} \]
\[ \sigma_{qSd} = 14.758 \quad \sigma_{hRd} = 36.726 \quad f_m = 391.24 \]

**Method A (\( \sigma_{a,sd} \) is in Tension)**

a). For \( \sigma_{aSD} \geq \sigma_{qSD} \) (net axial tension condition)

\[ \eta = 5 - 4 \frac{f_h}{f_y} \quad \eta = 4.403 \quad (6.38) \]

\[ B := \min \left( 1, \frac{\sigma_{psd}}{f_{hRd}} \right) \quad B = 0.804 \quad (6.37) \]

Design bending resistance in the presence of external hydrostatic pressure:

\[ f_{mhRd} := \frac{f_m}{\gamma_M} \left( \sqrt{1 + 0.09 \cdot B^2 - \eta^2 \cdot B^2} - 0.3 \cdot B \right) \quad (6.36) \]

\[ f_{mhRd} = 199.394 \]

Design axial tensile resistance in the presence of external hydrostatic pressure:

\[ f_{hRd} := \frac{f_y}{\gamma_M} \left( \sqrt{1 + 0.09 \cdot B^2 - \eta^2 \cdot B^2} - 0.3 \cdot B \right) \quad (6.35) \]

\[ f_{hRd} = 172.828 \]

\[ \text{UR}_{6.34} := \begin{cases} 0 & \text{if } \sigma_{aSD} < 0 \\ \frac{\sigma_{aSD} - \sigma_{qSD}}{f_{hRd}} + \sqrt{\left( \frac{\sigma_{MySD}^2 + \sigma_{mzSD}^2}{f_{mhRd}} \right)} & \text{if } \sigma_{aSD} > 0 \end{cases} \quad (6.34) \]

\[ \text{UR}_{6.34} = 0 \]

b). For \( \sigma_{aSD} < \sigma_{qSD} \) (net axial compression condition)

\[ f_{clRd} := \frac{f_{cl}}{\gamma_M} \quad (6.40) \quad f_{clRd} = 246.27 \]

\[ \text{UR}_{6.39} := \frac{\left| \sigma_{aSD} - \sigma_{qSD} \right|}{f_{clRd}} + \sqrt{\left( \frac{\sigma_{MySD}^2 + \sigma_{mzSD}^2}{f_{mhRd}} \right)} \]
\[ UR_{5.39} = 0.303 \]

When \( \sigma_{\text{csd}} > 0.5f_{\text{he}}/\gamma_M \) and \( f_{\text{cle}} > 0.5f_{\text{he}} \) then

\[
\sigma_{\text{mSd}} := \sqrt{\frac{M_{\text{xSd}}^2 + M_{\text{ySd}}^2}{w_e}}
\]

\[ \sigma_{\text{csd}} := \sigma_{\text{mSd}} + \sigma_{\text{qSd}} - \sigma_{\text{aSd}} \quad \text{Reflect the maximum combined compressive stress} \]

\[ \sigma_{\text{csd}} = 70.803 \]

\[
UR_{6.41} = \frac{\sigma_{\text{csd}} - 0.5 \frac{f_{\text{he}}}{\gamma_M}}{\frac{f_{\text{cle}}}{\gamma_M} - 0.5 \frac{f_{\text{he}}}{\gamma_M}} + \left( \frac{\sigma_{\text{psd}}}{\frac{f_{\text{he}}}{\gamma_M}} \right)^2 \tag{6.41}
\]

\[ UR_{6.41} = 0.681 \]

**Method B (\( \sigma_{\text{acSd}} \) is in Tension)**

\[ \sigma_{\text{acSd}} := -39.7 \]

Design axial stress that includes the effect of the capped-end compression arising from the external hydrostatic pressure (tension positive)

\[
UR_{6.42} := \frac{\sigma_{\text{acSd}}}{f_{\text{thRd}}} + \frac{\sigma_{\text{mySd}}^2 + \sigma_{\text{mzSd}}^2}{f_{\text{mhRd}}} \tag{6.42}
\]

\[ UR_{6.42} = 0.308 \]

6.3.9.2 Axial Compression, Bending, and Hydrostatic Pressure

**Method A (\( \sigma_{\text{aSd}} \) is in Compression)**

\[ \lambda := \lambda_y \quad \lambda = 0.394 \]

\[ \xi := 1 - 0.28 \lambda^2 \quad \xi = 0.957 \]
\[ f_{chRd} = \begin{cases} \frac{1}{2} \frac{f_{cl}}{\gamma_M} \left( \xi - \frac{2 \sigma_{qSd}}{f_{cl}} + \frac{\sigma_{qSd}^2}{f_{cl}^2} \right) \sqrt{\left( \frac{2}{1 - \frac{2 \sigma_{qSd}}{f_{cl}}} \right)^{-1}} & \text{if } \lambda < 1.34 \\ \frac{0.9 f_{cl}}{\lambda^2 \gamma_M} \sqrt{\left( \frac{2 \sigma_{qSd}}{f_{cl}} \right)^{-1}} & \text{if } \lambda \geq 1.34 \end{cases} \] (6.47)

\[ f_{chRd} = 225.514 \] (6.48)

\[ f_{EY} := \frac{2 \cdot E}{k \cdot l_y} \] (6.45) \[ f_{EY} = 2.224 \times 10^3 \]

\[ f_{EZ} := \frac{\pi^2 \cdot E}{k \cdot l_z} \] (6.46) \[ f_{EZ} = 2.224 \times 10^3 \]

\[ UR6.43 := \frac{\sigma_{aSd}}{f_{chRd}} + \frac{1}{f_{mhrd}} \left[ \frac{C_{my} \cdot \sigma_{mySd}}{1 - \frac{\sigma_{aSd}}{f_{EY}}} + \left( \frac{C_{mz} \cdot \sigma_{mzSd}}{1 - \frac{\sigma_{aSd}}{f_{EZ}}} \right)^2 \right]^{-0.5} \] (6.43)

\[ UR6.44 := \frac{\sigma_{aSd} + \sigma_{qSd}}{f_{chRd}} + \sqrt{\frac{\sigma_{mySd}^2 - \sigma_{mzSd}^2}{f_{mhrd}}} \] (6.44)

\[ UR_6.43 = 0.247 \quad UR_6.44 = 0.303 \]

**Method B (\( \sigma_{aSd} \) is in Compression)**

\[ \sigma_{aSd} = 39.7 \]

Design axial stress that includes the effect of the capped-end compression arising from the external hydrostatic pressure (compression positive)

\[ UR6.50 := \frac{\sigma_{aSd} - \sigma_{qSd}}{f_{chRd}} + \frac{1}{f_{mhrd}} \left[ \frac{C_{my} \cdot \sigma_{mySd}}{1 - \frac{\sigma_{aSd} - \sigma_{qSd}}{f_{EY}}} + \left( \frac{C_{mz} \cdot \sigma_{mzSd}}{1 - \frac{\sigma_{aSd} - \sigma_{qSd}}{f_{EZ}}} \right)^2 \right]^{-0.5} \] (6 - 50)

\[ UR6.50 = 0.181 \]
\[ UR_{6.51} := \frac{\sigma_{accSd}}{f_{cIRd}} + \sqrt{\frac{\sigma_{mySd}^2 + \sigma_{mxSd}^2}{f_{mhrd}}} = 0.243 \quad (6 - 51) \]

\[ U_m := \begin{cases} 
\max(UR_{6.50}, UR_{6.51}) & \text{if } \sigma_{accSd} > \sigma_{qSd} \\
UR_{6.51} & \text{if } \sigma_{accSd} \leq \sigma_{qSd} 
\end{cases} \]

\[ U_m = 0.243 \]
6.4. Tubular joints

6.4.3.1 General

Validity Range

\[ 0.2 \leq \beta \leq 1.0 \]
\[ 10 \leq \gamma \leq 50 \]
\[ 30^\circ \leq \theta \leq 90^\circ \]
\[ \tau \leq 1.0 \]

\[ F_y \leq 500 \quad \frac{N}{mm^2} \]

\[ \frac{\varepsilon_b}{D} \geq -0.6 \quad \text{For K-joints} \]

INPUT: Basic Geometric Parameters for Simple Tubular Joints

\[ F_y := 345 \quad \text{Yield strength, chord} \]
\[ F_{yb} := 345 \quad \text{Yield strength, brace} \]
\[ \theta := 49.6^\circ \quad \text{Brace included angle} \]
\[ \varepsilon_b := 1058 \quad \text{Gap between braces} \]

\[ t := 22.2 \quad \text{Brace wall thickness at intersection} \]

\[ T_c := 53.3 \quad \text{Chord wall thickness at intersection} \]

\[ d := 853 \quad \text{Brace outside diameter} \]

\[ D := 2560 \quad \text{Chord outside diameter} \]

\[ A_c := \frac{\pi}{4} \left[ D^2 - (D - 2T_c)^2 \right] = 4.197 \times 10^5 \quad \text{Chord cross sectional area} \]

\[ \beta := \frac{d}{D} \quad \beta = 0.333 \]

\[ \gamma := \frac{D}{2T_c} \quad \gamma = 24.015 \]

\[ \tau := \frac{t}{T_c} \quad \tau = 0.417 \]
6.4.3.3. Strength Factor \( Q_u \)

**Joint only with positive gap**

\[
Q_{g1} = \begin{cases} 
1.9 - \left( \frac{g_b}{D} \right)^{0.5} & \text{if } \frac{g_b}{T_c} \geq 2 \\
"gap too small" & \text{otherwise}
\end{cases}
\]

\[Q_g = \max(1, Q_{g1})\]

\[Q_g = 1.257\] Gap factor

\[Q_\beta := \begin{cases} 
0.3 & \text{if } \beta > 0.6 \\
\beta (1 - 0.833\beta) & \text{otherwise}
\end{cases}\]

\[Q_\beta = 1\] Geometrical factor

\( Q_u \) varies with the joint and load type, as given in Table 6-3.

- **\( Q_u \) for Axial Loads**
  \[
  Q_{u_a} := (1.9 + 1.9\cdot\beta) \cdot Q_\beta^{0.5} \cdot Q_g
  \]
  \[Q_{u_a} = 10.347\]

- **\( Q_u \) for In-Plane Bending**
  \[
  Q_{u_{ipb}} := 4.5\cdot\beta \cdot \gamma^{0.5}
  \]
  \[Q_{u_{ipb}} = 7.348\]

- **\( Q_u \) for Out-Of-Plane Bending**
  \[
  Q_{u_{opb}} := 3.2\cdot\gamma \left(0.5\cdot\beta^2\right)
  \]
  \[Q_{u_{opb}} = 3.818\]
6.4.3.4 Chord action factor $Q_t$

$Q_t$ is a factor to account for the presence of nominal loads in the chord.

\[
Z_E := \frac{\pi}{32} \left[ \frac{D^4 - (D - 2 \cdot T_c)^4}{n} \right] \quad \text{Elastic section modulus}
\]

\[
Z_E = 2.577 \times 10^8
\]

\[
P_c := -12525200 \quad \text{Chord axial force}
\]

\[
M_{c\_ipb} := 1827400000 \quad \text{Chord bending moment, in-plane bending}
\]

\[
M_{c\_obp} := 115418000 \quad \text{Chord bending moment, out-of-plane bending}
\]

\[
\sigma_{aSd} := \frac{P_c}{A_c} = -29.84 \quad \text{Design axial stress in chord}
\]

\[
\sigma_{mySd} := \frac{M_{c\_ipb}}{Z_E} = 7.092 \quad \text{Design in-plane bending stress in chord}
\]

\[
\sigma_{mzSd} := \frac{M_{c\_obp}}{Z_E} = 0.448 \quad \text{Design out-of-plane bending stress in chord}
\]

\[
\lambda_a := 0.03
\]

\[
\lambda_{ipb} := 0.045
\]

\[
\lambda_{obp} := 0.021
\]

INPUT the following coefficients:

Coefficients depending on joint and load type as given in Table 6-4.

\[
C_{1\_a} := 20
\]

\[
C_{2\_a} := 22
\]

\[
A_{a} := C_{1\_a} \left( \frac{\sigma_{aSd}}{F_y} \right)^2 + C_{2\_a} \left( \frac{\sigma_{mySd}^2 + \sigma_{mzSd}^2}{1.62 \cdot F_y^2} \right)^{0.5}
\]  \quad (6.55)
\[ A_a = 0.394 \]

\[ Q_{L_a} := \left( 1 - \lambda_a A_a^2 \right) \]  
(6.54)

\[ Q_{L_a} = 0.995 \]
Chord action factor, axial force

\[ C_{1_b} = 25 \]
\[ C_{2_b} = 30 \]

\[ A_b := \left[ C_{1_b} \left( \frac{\sigma_{nd}}{F_y} \right)^2 + C_{2_b} \left( \frac{\sigma_{nyst}^2 + \sigma_{nmsd}^2}{1.62 F_y^2} \right) \right]^{0.5} \]
(6.55)

\[ A_b = 0.441 \]

\[ Q_{L_{ipb}} := 1.0 - \lambda_{ipb} A_b^2 \]  
(6.54)

\[ Q_{L_{ipb}} = 0.991 \]
Chord action factor, in-plane bending

\[ Q_{L_{opb}} := 1.0 - \lambda_{opb} A_b^2 \]  
(6.54)

\[ Q_{L_{opb}} = 0.986 \]
Chord action factor, out-of-plane bending

### 6.4.3.2 Basic resistance

Tubular joints without overlap of principal braces and having no gussets, diaphragms, grout or stiffeners should be designed using the following guidelines.

\[ \gamma_M := 1.15 \]
Resistance factor

\[ N_{Rd} := \frac{F_y T_e^2}{\gamma_M \sin(\theta)} Q_{u_a} Q_{f_a} \]  
(6.52)

\[ N_{Rd} = 1.153 \times 10^7 \]
Joint design axial resistance

\[ M_{Rd_{ipb}} := \frac{F_y T_e^2 d}{\gamma_M \sin(\theta)} Q_{u_{ipb}} Q_{f_{ipb}} \]  
(6.53)
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