DET NORSKE VERITAS

TECHNICAL REPORT

BUREAU OF OCEAN ENERGY MANAGEMENT,
REGULATION, AND ENFORCEMENT

FINAL REPORT

ON

COMPARISON OF API, ISO, AND NORSOK OFFSHORE STRUCTURAL STANDARDS

TA&R NO. 677

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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.

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Nominclature

C_a, C_v site coefficients

C_{SF} safety factor

d brace outside diameter

D chord outside diameter

D_e equivalent quasi-static action representing dynamic response defined in 9.8.1

D_o equivalent quasi-static action representing dynamic corresponding to E_o

E_e extreme environmental quasi-static action

E_o environmental action or loading

F_d action effect

F_d design action

G Ratio of effective horizontal ground acceleration to gravitational acceleration

G_1, G_2 permanent actions or gravity loads

G_T the action imposed either by the weight of the structure in air, or by the submerged weight of the structure in water

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.

Q_1, Q_2 variable actions or live loads

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

R_d design strength

S internal force

$S_{a,map}(0.2)$ = 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

$S_{a,map}(1.0)$ = 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

$S_{a,site}$ (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

$\beta = d/D$

$\gamma = D/2T$

$\gamma_{f,E}$, $\gamma_{f,D}$ 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

$\gamma_{f,G1}$, $\gamma_{f,G2}$, $\gamma_{f,Q1}$, $\gamma_{f,Q2}$ 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

$\tau = t/T$

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

Number	Revision	Title
API RP 2A (WSD)	21st Edition October 2007	Recommended Practice for Planning, Designing and Constructing Fixed Offshore Platforms – Working Stress Design
API RP 2T	3rd Edition July 2010	Planning, Designing, and Construction Tension Leg Platforms
API RP 2FPS	1st Edition March 2001	Recommended Practice for Planning, Designing, and Constructing Floating Production Systems
API RP 2A (LRFD)	1st Edition May 2003	Recommended Practice for Planning, Designing and Constructing Fixed Offshore Platforms – Load and Resistance Factor Design
API Bulletin 2INT-MET	May 2007	Interim Guidance on Hurricane Conditions in the Gulf of Mexico
API Bulletin 2INT-DG	May 2007	Interim Guidance for Design of Offshore Structures for Hurricane Conditions
API Bulletin 2INT-EX	May 2007	Interim Guidance for Assessment of Existing Offshore Structures for Hurricane conditions
ISO 19901-2	1st Edition November 2004	Specific requirements for offshore structures – Part 2: Seismic Design Procedures and Criteria
ISO 19901-6	1st Edition December 2009	Specific requirements for offshore structures – Part 6: Marine Operations
ISO 19902	1st Edition December 2007	Fixed Steel Offshore Structures
ISO 19904-1	1st Edition November 2006	Floating offshore structures – Part 1: Monohulls, Semi-submersibles and Spars
NORSOK Standard N-001	7th Edition June 2010	Integrity of Offshore Structures
NORSOK Standard N-003	2nd Edition September 2007	Action and Action Effects
NORSOK Standard N-004	2nd Edition October 2004	Design of Steel Structures
NORSOK Standard N-006	1st Edition March 2009	Assessment of Structure Integrity for Existing Offshore Load-bearing Structures

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.

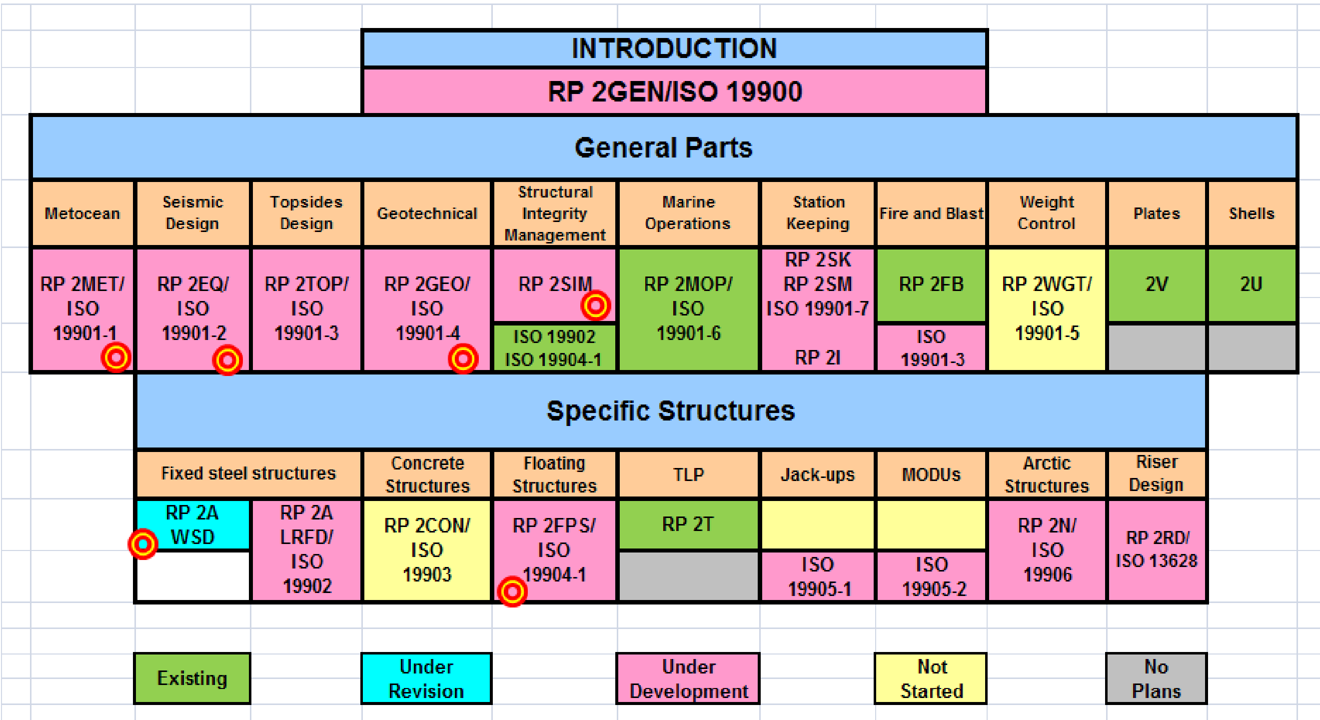


Figure 1-1 ISO API Standards Harmonization

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 vise 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.

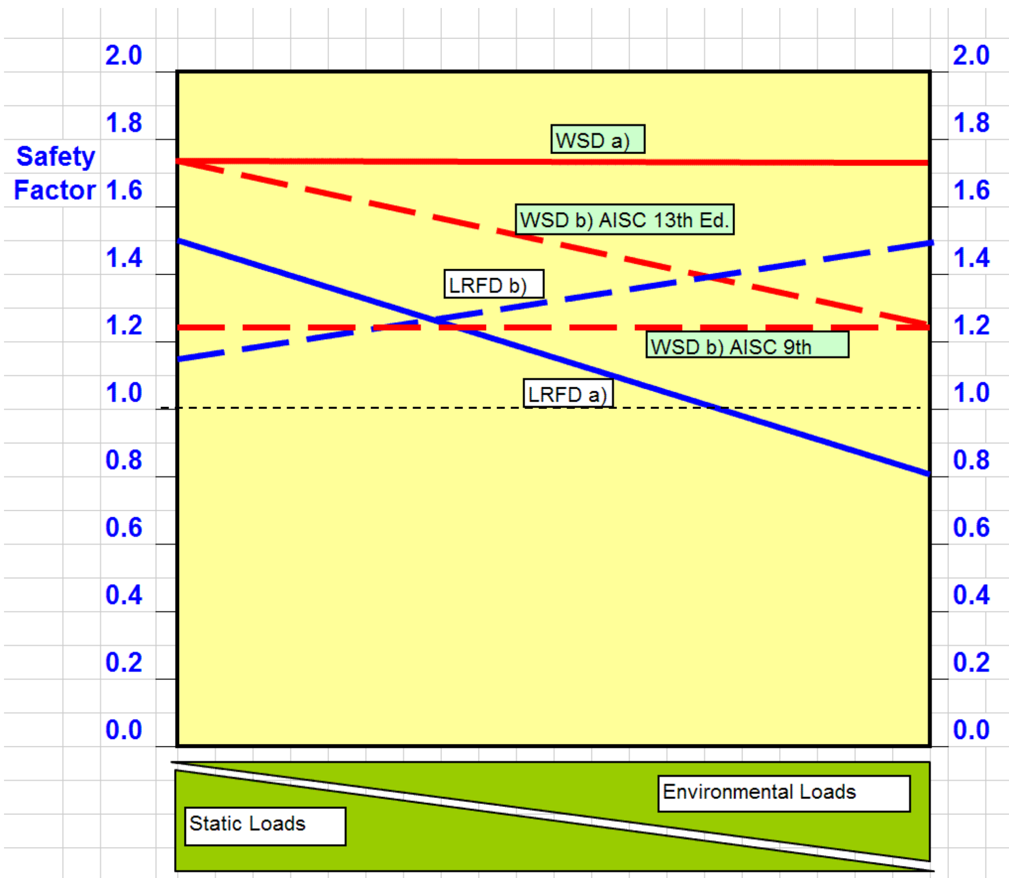


Figure 1-2 Schematic of LRFD vs. WSD Methods

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

AP 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

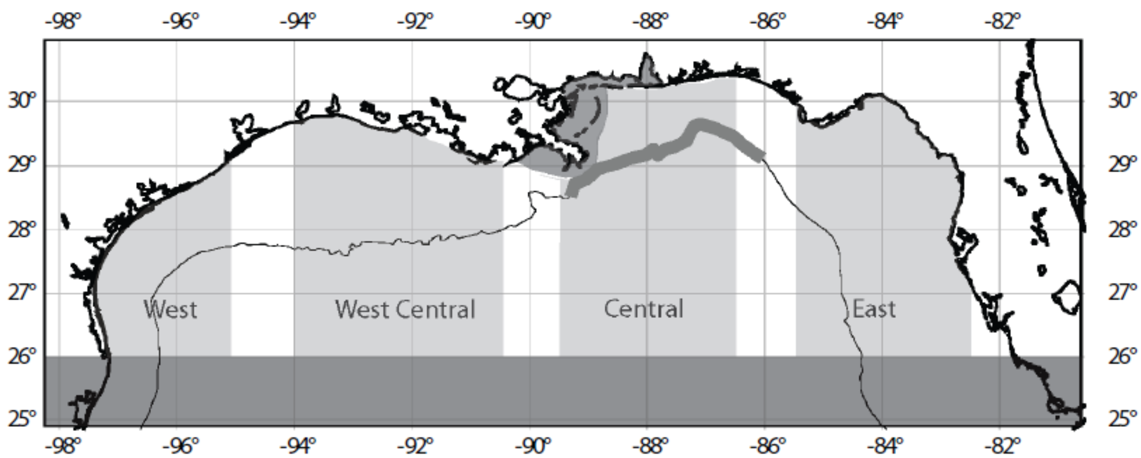


Figure 2-1 Gulf of Mexico Zones in API Bulletin 2INT-MET

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 86.5°W)

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

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.

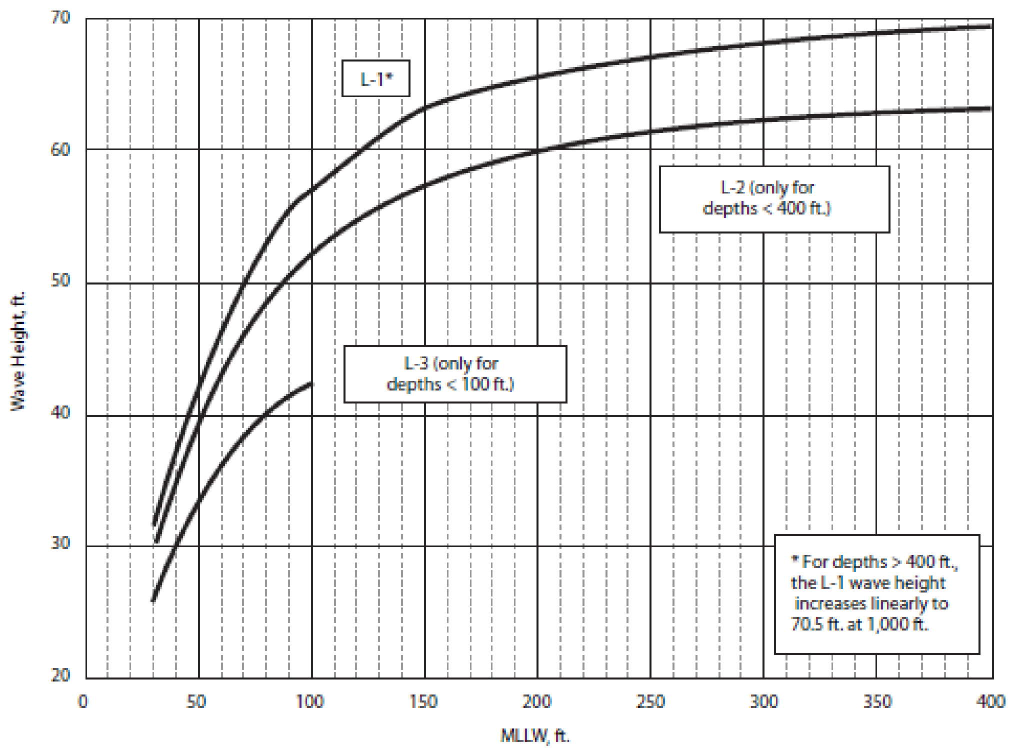


Figure 2-2 Original Extreme Wave Definition in API RP 2A 21st Edition

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 3—Loading Type Category Descriptions

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

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 2-3 API RP 2T Load Conditions

Table 1—Project Design Load Cases

Design Load Case	Safety Category	Project Phase	Platform Configuration[e]	Design Environment	Annual Probability of Exceedance
1	A	Construction	Various		
2	A	Load out	Intact	Calm	
3	B	Hull/deck mating	Intact	Site specific	
4	B	Tow/transportation	Intact/damaged	Route	Varies
5	A	Installation	Intact	Installation	Varies
6	A	In place	Intact	One-year normal	≤1
7	B	In place	Intact	100-year extreme	0.01
8	S	In place	Intact	1000-year extreme	0.001
9	B	In place	Damaged—no compensation	One-year normal	≤ 0.01 [a]
10	S [b c]	In place	Damaged—no compensation	10-year reduced extreme	≤ 0.001 [a]
11	B	In place	Damaged—compensation	10-year reduced extreme	≤ 0.01 [a]
12	S [b c]	In place	Damaged—compensation	100-year extreme	≤ 0.001 [a]
13	B	In place	Tendon removed (planned)	10-year reduced extreme	≤ 0.01 [a]
14	S [b c]	In place	Tendon removed (planned)	100-year extreme	≤ 0.001 [a]
15	C	In place	Intact	Annual scatter diagram	1
16	SLE [d]	In place	Intact	SLE seismic	Varies
17	DLE [d]	In place	Intact	DLE seismic	Varies

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 exceedence 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.

Table 2-4 API RP 2T Environnemental Parameters

Table 5—Environmental Parameters Influencing TLP Response

Environmental Condition	Environmental Parameter
Wind	Mean wind speed Mean wind direction Wind power spectral density function
Wave	Significant wave height Mean wave period Wave elevation spectral density function Mean wave direction Wave directional spreading function
Current	Surface current (speed and direction) Current profile (speed and direction)
Tide	Astronomical tide Storm surge

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

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 the100 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_e, 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).

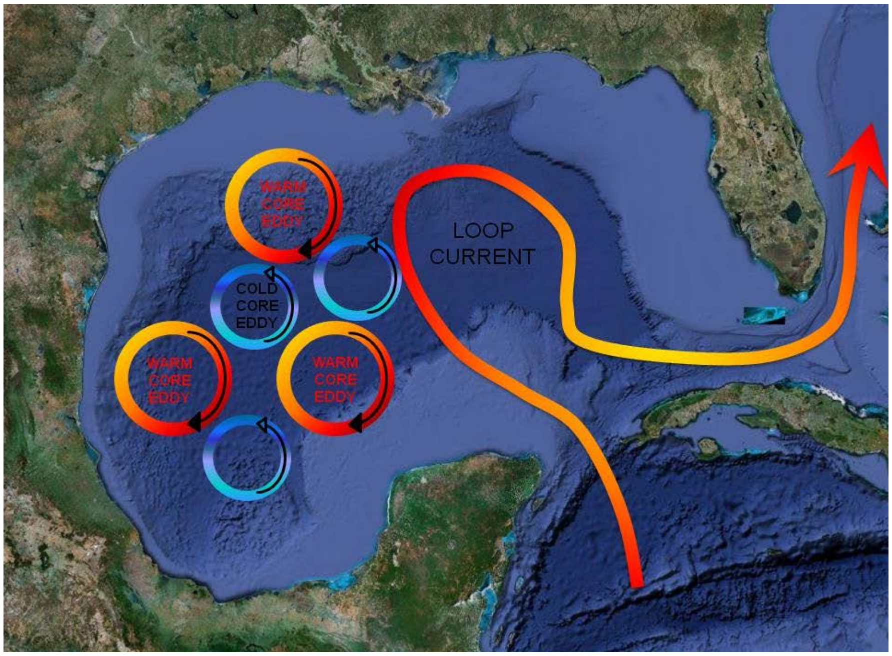


Figure 2-3 Loop Current (NOAA) /23/

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 API RP 2A 21st Edition.

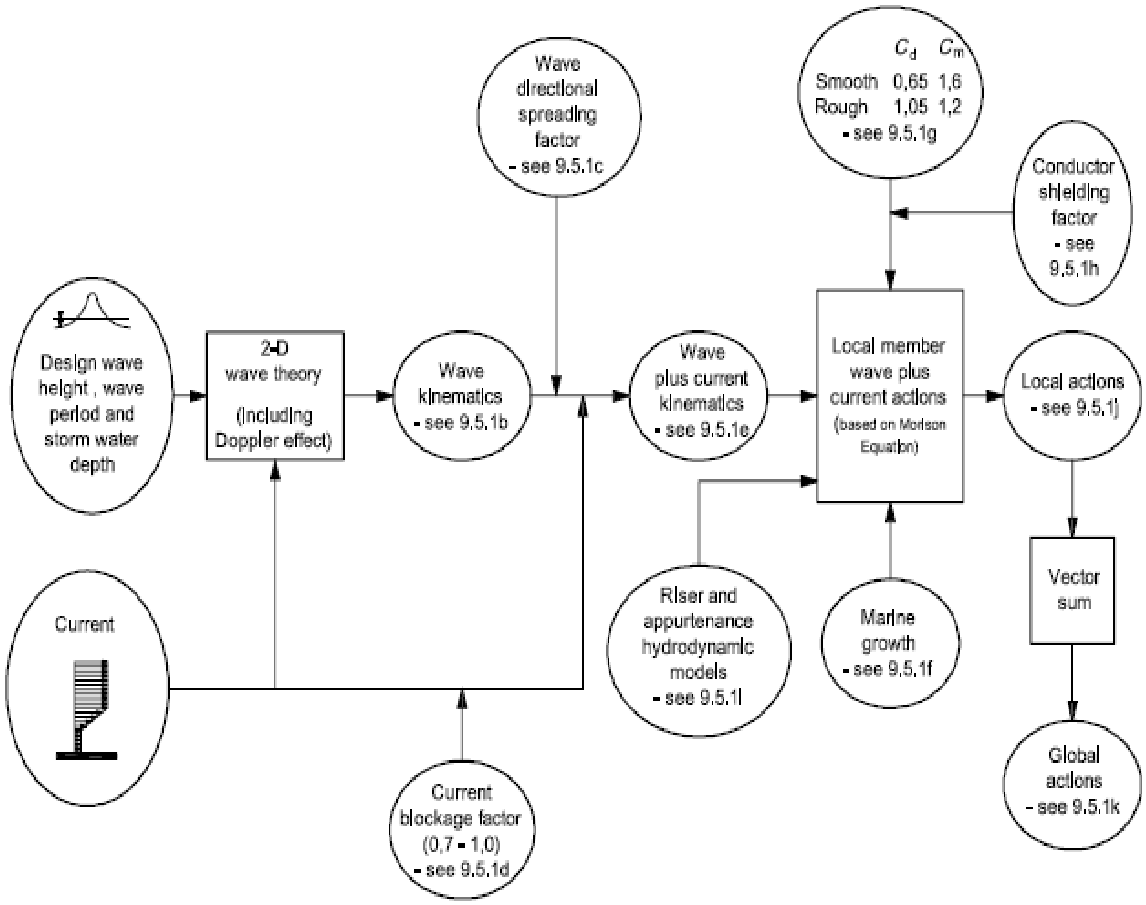


Figure 9.5-1 — Procedure for calculating the quasi-static action caused by wave plus current

Figure 2-4 Wave and Current load combination procedure

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:

$$F_d = \gamma_{f,G1}\, G_1 + \gamma_{f,G2}\, G_2 + \gamma_{f,Q1}\, Q_1 + \gamma_{f,Q2}\, Q_2 + \gamma_{f,Eo}\,(E_o + \gamma_{f,D}\, D_o) + \gamma_{f,Ee}\,(E_e + \gamma_{f,D} D_e) \qquad (2.1)$$

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;

Do 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.

Table 2-5 Partial Factors

Table 9.10-1 — Partial action factors for in-place situations and exposure level L1

Design situation	Partial action factors [a]					
	$\gamma_{f,G1}$	$\gamma_{f,G2}$	$\gamma_{f,Q1}$	$\gamma_{f,Q2}$	$\gamma_{f,Eo}$	$\gamma_{f,Ee}$
Permanent and variable actions only	1,3	1,3	1,5	1,5	0,0	0,0
Operating situation with corresponding wind, wave, and/or current conditions[b]	1,3	1,3	1,5	1,5	0,9 $\gamma_{f,E}$	0,0
Extreme conditions when the action effects due to permanent and variable actions are additive[c]	1,1	1,1	1,1	0,0	0,0	$\gamma_{f,E}$
Extreme conditions when the action effects due to permanent and variable actions oppose[d]	0,9	0,9	0,8	0,0	0,0	$\gamma_{f,E}$

[a] A value of 0 for a partial action factor means that the action is not applicable to the design situation.

[b] For this, check that G_2, Q_1 and Q_2 are the maximum values for each mode of operation.

[c] For this, check that G_1, G_2 and Q_1 include those parts of each mode of operation that can reasonably be present during extreme conditions.

[d] For this, check that G_2 and Q_1 exclude any parts associated with the mode of operation considered that cannot be ensured of being present during extreme conditions.

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 $3x10^{-5}$. 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 $5x10^{-4}$. 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 10.5-1 — Comparison of extreme and abnormal environmental action requirements

Requirement	Situation	
	Extreme environmental actions	Abnormal environmental actions
Governing clause for actions	Clause 9	Clause 10
Limit state	ULS	ALS
Return period	100 years	See 10.1.5, default 10 000 years
Partial action factor	See Clause 9, default 1,35	1,0
Partial resistance factors	See Clauses 13, 14, 15, 17; generally 1,05 to 1,25 but up to 2,0	1,0
Wave crest height	Associated with 100 year return event	Associated with abnormal environmental event

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:

Table 2-7 Action Combinations - LSD

Table 4 — Partial action factors (γ_f) and combinations

Limit state	Partial action factor γ_f — Action category: Permanent (G)	Variable (Q)	Environmental (E)	Repetitive (R)	Accidental (A)
ULS-a	1,3	1,3	0,7	—	—
ULS-b	1,0	1,0	1,3	—	—
SLS	1,0	1,0	1,0	1,0	—
Pre-ALS	1,0	1,0	—	—	1,0
Post-ALS	1,0	1,0	1,0	—	—

In the ULS-a condition, an action factor of 1,0 shall be used for the permanent action, the variable action, or both, where this gives a more unfavourable combined action effect than 1,3.

The action factor for permanent actions in ULS-a may be reduced from 1,3 to 1,2 if the action and action effects are determined with great accuracy (for example, external hydrostatic fluid pressures acting on a rigid body).

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 γ_r, nor the partial material factor, γ_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 2-8 Action Combinations - WSD

Table 5 — Action combination factors

Limit state	Action combination factor				
	Action category				
	Permanent (G)	Variable (Q)	Environmental (E)	Repetitive (R)	Accidental (A)
ULS-a	1,0	1,0	—	—	—
ULS-b	1,0	1,0	1,0	—	—
SLS	1,0	1,0	1,0	1,0	—
Pre-ALS	1,0	1,0	—	—	1,0
Post-ALS	1,0	1,0	1,0	—	—

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 (η). Thus, the design check may be expressed as

$$F_d \leq \frac{R_d}{C_{SF}} \qquad (2.2a)$$

or, alternatively:

$$F_d \leq \eta R_d \qquad (2.2b)$$

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 4 – Combination of environmental actions with expected mean values and annual probability of exceedance 10^{-2} and 10^{-4}

Limit state	Wind	Waves	Current	Ice	Snow	Earthquake	Sea level [a]
Ultimate Limit State	10^{-2}	10^{-2}	10^{-1}	-	-	-	10^{-2}
	10^{-1}	10^{-1}	10^{-2}	-	-	-	10^{-2}
	10^{-1}	10^{-1}	10^{-1}	10^{-2}	-	-	m
	-	-	-	-	10^{-2}	-	m
	-	-	-	-	-	10^{-2}	m
Accidental Limit State	10^{-4}	10^{-2}	10^{-1}	-	-	-	m*
	10^{-2}	10^{-4}	10^{-1}	-	-	-	m*
	10^{-1}	10^{-1}	10^{-4}	-	-	-	m*
	-	-	-	10^{-4}	-	-	m
	-	-	-	-	-	10^{-4}	m

[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.

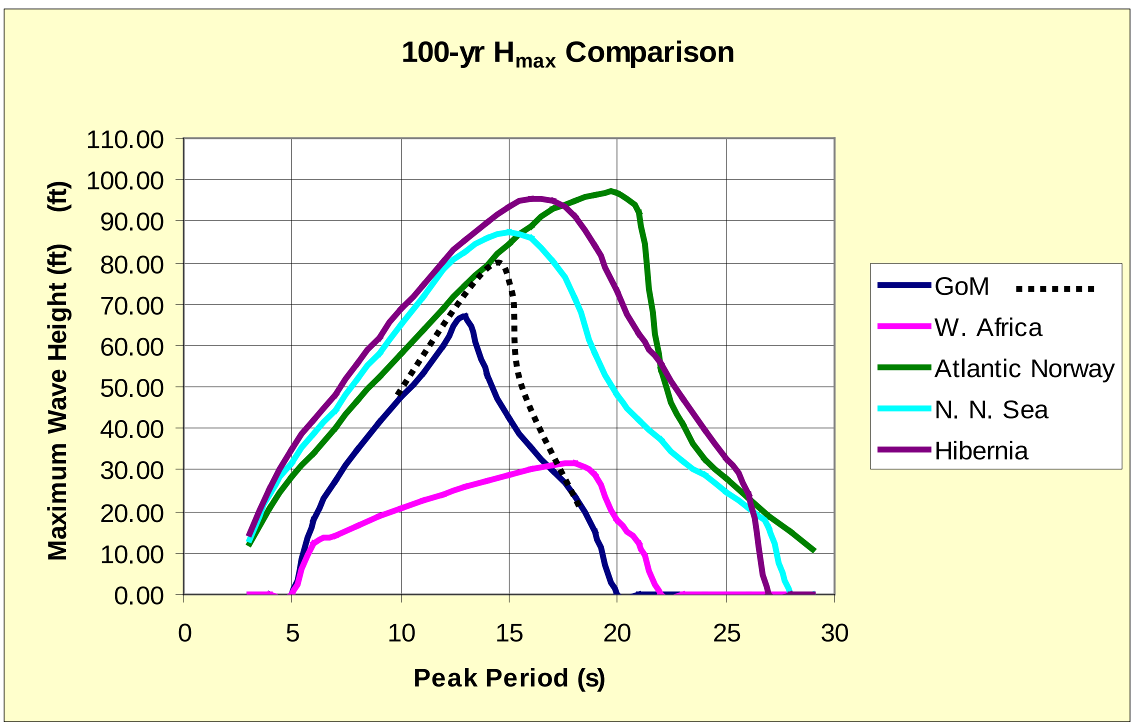


Figure 2-5 Regional Wave Design Criteria

2.4.2 N-001 - Integrity of offshore Structures

In Section 6.2 of NORSOK N-001defines 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 1 – Partial action factor for the limit states

Limit state	Action combinations	Permanent actions (G)	Variable actions (Q)	Environmental actions (E) [d]	Deformation actions (D)[e]
ULS	a[a]	1,3	1,3	0,7	1,0
ULS	b	1,0	1,0	1,3	1,0
SLS		1,0	1,0	1,0	1,0
ALS	Abnormal effect [b]	1,0	1,0	1,0	1,0
ALS	Damaged condition [c]	1,0	1,0	1,0	1,0
FLS		1,0	1,0	1,0	1,0

[a] For permanent actions and/or variable actions, an action factor of 1,0 shall be used where this gives the most unfavourable action effect
[b] Actions with annual probability of exceedance = 10^{-4}

Limit state	Action combinations	Permanent actions (G)	Variable actions (Q)	Environmental actions (E) [d]	Deformation actions (D)[e]

[c] Environmental actions with annual probability of exceedance = 10^{-2}
[d] Earthquake shall be handled as environmental action within the limit state design for ULS and ALS (abnormal effect)
[e] Applicable for concrete structures

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.

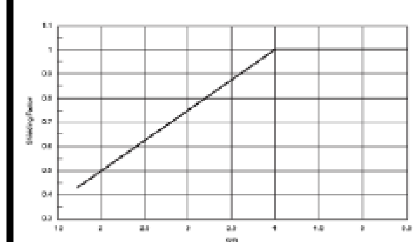
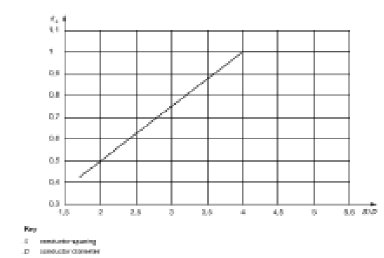
With regards to deck clearance requirements, it is noted that all three codes require 1.5m (5 ft) air gap above the 100-year wave crest elevation. As stated in Section 2.2.1, Bulletin 2INT-DG gaves guidance on design using the new metocean criteria of 2INT-MET and significantly increased the requirement 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 1.5 m 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

	API RP 2A/API 2INT-MET	ISO 19901-1/ISO 19902	NORSOK N-003
Waves	**Section 2.3.1**	**Annex A8**	**Section 6**
Wave Kinematics factor	**API RP 2A, 2.3.1** 0.85 to 0.95 for tropical storms 0.95 to 1.00 for extra-tropical storm	**ISO 19902, A.24.7.3** 0.88 for tropical cyclones 1.0 for winter storms	**N-003, 6.2.4** 0.95 for North Sea Conditions
Marine Growth	**API RP 2A, 2.3.4** 1.5" (38mm) from MHHW to -150 ft (-48 m) in GoM;	**ISO 19901-1, C.2.8 & C.4** Table C.1 for UK Sector Table C.2 same as NORSOK for Areas offshore Norway; GoM: LAT +3 m to -50 m: 38mm; Offshore southern and central California: 200mm are common	**N-003, 6.6.1** Area Offshore Norway only Water Depth m / 56° to 59° mm / 59° to 72° mm Above + 2 / 0 / 0 -2 to -40 / 100 / 60 Under -40 / 50 / 30
Drag and Inertia Coefficient	**API RP 2A, 2.3.1** smooth C_d=0.65; C_m=1.6 rough C_d=1.05; C_m=1.2 Applicable to $U_{mo}T_{app}/D > 30$;	**ISO 19902, 9.5.2** smooth C_d=0.65; C_m=1.6 rough C_d=1.05; C_m=1.2 Applicable to $U_{mo}T_i/D > 30$;	**N-003, 6.2.4** smooth C_d=0.65; C_m=1.6 rough C_d=1.05; C_m=1.2 Applicable to $U_{max}T_i/D > 30$;
Conductor Shielding Factor	**API RP 2A, 2.3.1** Figure 2.3.1-4, applicable to $U_{mo}/T_{app}/S > 5\pi$ (extreme waves); For less severe waves, with Umo/Tapp/S < 5p as in fatigue analyses, there may be less shielding  Figure 2.3.1-4—Shielding Factor for Wave Loads on Conductor Arrays as a Function of Conductor Spacing	**ISO 19902, 9.5.2** Figure 9.5-2, applicable to $U_{mo}/T_{app}/S > 5\pi$ (extreme waves); For less severe waves, with Umo/Tapp/S < 5p as in fatigue analyses, the shielding shall not be invoked. 	**N-003, 6.2.4** (referred to ISO 19902)
Wind	**Section 2.3.2 Wind**		
Wind profiles and Gusts	**API RP 2A, 2.3.2** $u(z,t) = U(z) \times [1-0.41 \times I_u(z) \times \ln(t/t_o)$; $U(z) = U_o \times [1 + C \times \ln(z/32.8)$ $C = 5.73 \times 10^{-2} \times (1+0.0457 \times U_o)^{1/2}$ $I_u(z) = 0.06 \times [1+0.0131 \times U_o] \times (z/32.8)^{-0.22}$ where $U_{o\,(ft/s)}$ is the 1 hour mean wind speed at 32.8 ft	**ISO 19901-1, C.7.3** $U_{w,T}(z,t) = U_{w,1h}(z) \times [1-0.41 \times I_u(z) \times \ln(T/T_o)$; $U_{w,1h}(z) = U_{w0} \times [1 + C \times \ln(z/z_r)$ $C = 5.73 \times 10^{-2} \times (1+0.15 \times U_{w0})^{1/2}$ $I_u(z) = 0.06 \times [1+0.043 \times U_{w0}] \times (z/z_r)^{-0.22}$ where $U_{w,1h\,(m/s)}$ is the 1 hour mean wind speed at Z_r= 10 m; (SI units)	**N-003, 6.3.2** $u(z,t) = U(z) \times [1-0.41 \times I_u(z) \times \ln(t/t_o)$; $U(z) = U_o \times [1 + C \times \ln(z/32.8)$ $C = 5.73 \times 10^{-2} \times (1+0.15 \times U_o)^{1/2}$ $I_u(z) = 0.06 \times [1+0.043 \times U_o] \times (z/10)^{-0.22}$ where $U_{o\,(m/s)}$ is the 1 hour mean wind speed at 10m; All in SI units
Wind Spectra	**API RP 2A, 2.3.2** $S(f) = [3444 \times (U_o/32.8)^2 \times (z/32.8)^{0.45}]/(1+\tilde{f}^n)^{(5/3n)}$ $\tilde{f} = 172 \times f\,(z/32.8)^{2/3} \times (U_o/32.8)^{-0.75}$ where n = 0.468 S(f) ($ft^2/s^2/Hz$) = spectral energy density at frequency f (Hz) z(ft) = height above sea level Uo (ft/s) = the 1 hour mean wind speed at 32.8 ft above see level	**ISO 19901-1, A.7.4** $S(f,z) = [320\ m/s^2 \times (U_{w0}/U_{ref})^2 \times (z/z_r)^{0.45}]/(1+\tilde{f}^n)^{(5/3n)}$ $\tilde{f} = 172 \times f\ (z/z_r)^{2/3} \times (U_{w0}/U_{ref})^{-0.75}$ where n = 0.468, U_{ref} = 10m/s S(f) ($m^2/s^2/Hz$) = spectral energy density at frequency f (Hz) z(m) = height above mean sea level Uw0 (m/s) = the 1 hour mean wind speed at z_r	**N-003, 6.3.2** $S(f,z) = [320 \times (U_o/10)^2 \times (z/10)^{0.45}]/(1+\tilde{f}^n)^{(5/3n)}$ $\tilde{f} = 172 \times f\ (z/10)^{2/3} \times (U_o/10)^{-0.75}$ where n = 0.468, U_{ref} = 10m/s S(f) ($m^2/s^2/Hz$) = spectral density at frequency f (Hz) z(m) = height above sea level Uw0 (m/s) = the 1 hour mean wind speed at 10m above see level

Spatial Coherence	**API RP 2A, 2.3.2** 3 second gust is approriate for determining the maximum static wind load on individual member; 5 seconds gusts are appropriate for maximum total loads on structures whose maximum horizontal dimension is less than 164 ft (50 m); 15 seconds gusts are appropriate for the maximum total static wind load on larger structures; 1 minute sustained wind is appropriate for total static super structure wind loads associated with maximum wave forces for structure 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.	3 second gust is approriate for determining the maximum quasi-static wind load on individual member; 5 seconds gusts are appropriate for maximum quasi-static local or global actions on structures whose maximum horizontal dimension is less than 50 m; 15 seconds gusts are appropriate for the maximum quasi-static global actions on larger structures; For structures that are moderately dynamically sensitive, but do not require a full dynamic analysis, 1 minute sustained wind is appropriate for total static super structure wind loads associated with maximum wave forces for structure that respond dynamically to wind excitation but which do not require a full dynamic wind analysis; For structures with negligible dynamic response, 1h sustained wind can be used to determine quasi-static global actions caused by wind in conjunction with extreme or abnormal quasi-static actions due to waves and currents; For structures with significant dynamic response to excitation with periods longer than 20s, a full dynamic response analysis to fluctuating winds should be considered.	**N-003, 6.3.3** In case of structures or structural parts where the maximum dimension is less than approximately 50m, 3s wind gusts may be used when calculating static wind actions; In the case of structures or structural parts where the maximum length is greater than 50m, the mean period for wind may be increased to 15s; When design actions due to wind need to be combined with extreme actions due to waves and current, the mean wind speed over a 1 min period can be used.
Wind speed and force relationship	**API RP 2A, 2.3.2** $F = (\rho/2)u^2C_sA$ F = wind force; ρ = mass density of air, (slug/ft^3, 0.0023668 slugs/ft^3 for standard temperature and pressure) u = wind speed (ft/s) C_s = shape coefficient A = Area of object (ft^2)	**ISO 19902, 9.7** $F = (\rho_a/2)U_w^2C_sA$ F = wind force; in wind velocity direction ρ_a = mass density of air (at standard temperature and pressure), 1.226 kg/m^3 U_w = wind speed C_s = shape coefficient A = Area of object; normal to wind velocity direction	**N-003, 6.3.3** $F = (\rho/2)U_m^2C_sA\sin(\alpha)$ F = wind force; acting normal to the member axes or surface ρ = mass density of air U_m = wind speed C_s = shape coefficient A = Area of the member or surface area normal to the direction of the force α = the angle between the direction of the wind and the axis of the exposed member or surface
Shape Coefficient	**API RP 2A, 2.3.2** Beams - 1.5 Sides of buildings - 1.5 Cylindrical Sections - 0.5 Overall projected area of platform - 1.0	**ISO 19902, 9.7** Flat walls of building - 1.50 Overall projected area of structure - 1.00 Beams - 1.50 Cylinders - Smooth, Re > 5 x 10^5, 0.65 Smooth, Re <= 5 x 10^5, 1.20 Rough, all Re, 1.05 Covered with ice, all Re, 1.20	**Section 6.3.3** C_s = 0.65 for Reynold's number > 5 x 10^5 C_s = 1.20 for Reynolds's number < 5 x 10^5 Tubular structures covered with ice, C_s = 1.2 for all Reynolds numbers Further Details, refer to ENV 1991-2-4 and DNV Classification Note 30.5

Current Blockage Factor	**API RP 2A, 2.3.2**			**ISO 19902, A.95**			**N-003, 6.2.3**
	# of Legs	Heading	Factor	# of Legs	Heading	Factor	Section 6.2.3.2 - 0.9 for 3 legs, 0.85 for more than 3 legs; Refer to ISO 19902 for further details.
	3	all	0.9	3	all	0.9	
	4	end-on	0.8	4	end-on	0.8	
		diagonal	0.85		diagonal	0.85	
		broadside	0.8		broadside	0.8	
	6	end-on	0.75	6	end-on	0.75	
		diagonal	0.85		diagonal	0.85	
		broadside	0.8		broadside	0.8	
	8	end-on	0.7	8	end-on	0.7	
		diagonal	0.85		diagonal	0.85	
		broadside	0.8		broadside	0.8	

Snow and Ice	Section 2.3.5 Ice, Refer to API Bulletin 2N - Planning, Designing, and Constructing Fixed Offshore Platforms in Ice Environments	Details refer to ISO19906 - Arctic Offshore Structures; 19901-1 Annex C2.8.3 - UK sector 19901-1 Annex C6 - East Coast of Canada	Section 6.4 with details data Snow 0.5 Kpa for the entire Norwegian Continental Shelf Sea ice and iceberge referred to API Bulletin 2N

Deck Clearance	**API RP 2A, 2.3.4** Omindirectional 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. 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.	**ISO-19902 A.6.3.3.2** If storm surge is not expected to occur at the same time as the abnormal wave crest, deck elevation $h = (a^2 + s^2 + t^2)^{1/2} + f$ If storm surge is expected to occur at the same time as the abnormal wave crest, $h = [(a + s)^2 + t^2]^{1/2} + f$ where: a is the abnormal wave crest height; s is the extreme storm surge; t 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 For deep and intermediate water depth a can be approximated to $a > 1.3\ a_{100}$ $a > a_{100} + 1.5$ m a_{100} is the extreme wave crest height with a return period of 100 years	**N-004 K4.4.4** - The air gap should be sufficient to allow for the free passage of 10^{-4} wave without hitting deck members when described as Stokes 5th order kinematic theory. **N-003 10.2.5.5** - due to the complexity and uncertainty associated with determining actions associated with waves hitting the platforms decks, an air gap margin of 1.5 m on the 10-2 wave event, is recommended for fulfilling ULS criteria; The ALS criteria may be fulfilled by a positive air gap or by demonstrating survival of the platform subject to a 10^{-4} event.
Earthquake	**API RP 2A, 2.3.6** Separate comparison table provided	**ISO 19902 Section 11 & ISO 19901-2** Separate comparison table provided	**N-003, 6.5** Separate comparison table provided
Regional Design Metocean Criteria	Section 2.3.4 Hydrodynamic Force Guidelines for US Waters, and 2.3.4c hurricanes to be replaced by API 2INT-MET	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	NORSOK N-003 mainly focuses on North-West Coast of Europe and refers to ISO;

Note: 1. API RP 2A focuses on GoM and hurrican criteria(2.3.4c & 17.6.2a are replaced by API 2INT-MET);
2. ISO 19901-1 provides environmetal 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) but different way of calculation the maximum wave crest: API and ISO use the 100-yr return period waves;
NORSOK provides two options: ULS: 10^{-2} wave crest + 1.5 m or free passage of 10^{-4} 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 allocated 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 β is defined as

$$\beta = -\Phi^{-1}(P_f), \qquad (2.3)$$

where Φ^{-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 P_f in API Sec. 17 and NORSOK for unmanned and manned platforms

CoV(X_R) Capacity	API		Norsok		
	Unmanned	Manned	Unmanned	Manned (ULS)	Manned (ALS)
0.2	$5 \cdot 10^{-2}$	$1 \cdot 10^{-3}$ *)	$1 \cdot 10^{-4}$	$5 \cdot 10^{-5}$	$2 \cdot 10^{-5}$
0.1	$3 \cdot 10^{-2}$	$3 \cdot 10^{-4}$	$2 \cdot 10^{-4}$	$6 \cdot 10^{-5}$	$2 \cdot 10^{-5}$

*) Calibrated value

Table 2-13 ISO 19906 Reliability Targets for ULS and ALS

Exposure Level	Maximum Acceptable Annual Failure Probability
L1*	1.0×10^{-5}
L2*	1.0×10^{-4}
L3*	1.0×10^{-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	$P_F = 10^{-3}$ $\beta = 3.09$	$P_F = 10^{-4}$ $\beta = 3.71$
II. Significant warning prior to occurrence of failure in a non-redundant structure	$P_F = 10^{-4}$ $\beta = 3.71$	$P_F = 10^{-5}$ $\beta = 4.26$
III. No warning before the occurrence of failure in a non-redundant structure	$P_F = 10^{-5}$ $\beta = 4.26$	$P_F = 10^{-6}$ $\beta = 4.75$

Table 2-15 Guidelines for Design of Wind Turbines, DNV/Riso, 2002*

Table 2-3. Target annual failure probabilities P_{FT} and corresponding reliability indices β_T.

	Failure consequence		
Failure type	**Less serious** LOW SAFETY CLASS (small possibility for personal injuries and pollution, small economic consequences, negligible risk to life)	**Serious** NORMAL SAFETY CLASS (possibilities for personal injuries, fatalities, pollution, and significant economic consequences)	**Very serious** HIGH SAFETY CLASS (large possibilities for personal injuries, fatalities, significant pollution, and very large economic consequences)
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)	$P_F = 10^{-5}$ $\beta_T = 4.26$	$P_F = 10^{-6}$ $\beta_T = 4.75$	$P_F = 10^{-7}$ $\beta_T = 5.20$

*Reference NKB, 1978.

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

Design Loading Combinations

API RP 2A - WSD

API RP 2A -WSD Section 2.2.2

The loading conditions should include environmental conditions combined with appropriate dead and live loads in the following manner:

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

API RP 2A - LRFD Section C

Design Conditions	D_1	D_2	L_1	L_2	W_o	W_e	D_n
Factored Gravity Loads	1.3	1.3	1.5	1.5		-	-
Operating Wind Wave and Current Load	1.3	1.3	1.5	1.5	1.2	-	1.5
Extreme conditions when the actions effects due to permanent and variable actions are additive	1.1	1.1	1.1	-	-	1.35	1.6875
Extreme conditions when the actions effects due to permanent and variable actions are oppose	0.9	0.9	0.8	-	-	1.35	1.6875

D_1 = self weight of the structure

D_2 = the load imposed on the platform by weight of equipment and other objects

L_1 = Live load including the weight of consumable supplies and fluids in pipes and tanks

L_2 = the short duration force exerted on the structure from operations

W_o = the owner defined operating wind wave and current load

W_e = the force applied to the structure due to the combined action of the extreme wave (typically 100-yr return period) and associated current and wind

D_n = inertia load

ISO 19901-1/ISO 19902

ISO 19902 Table 9.10-1 - Partial action factors for in-place situations and exposure level L1

Design Condition	Partial action factors					
	$\gamma_{f,G1}$	$\gamma_{f,G2}$	$\gamma_{f,Q1}$	$\gamma_{f,Q2}$	$\gamma_{f,Eo}$	$\gamma_{f,Ee}$
Permanent and variable actions only	1.3	1.3	1.5	1.5	0.0	0.0
Operating situation will corresponding wind, wave, and/or current conditions	1.3	1.3	1.5	1.5	$0.9\gamma_{f,E}$	0.0
Extreme conditions when the actions effects due to permanent and variable actions are additive	1.1	1.1	1.1	0.0	0.0	$\gamma_{f,E}$
Extreme conditions when the actions effects due to permanent and variable actions are oppose	0.9	0.9	0.8	0.0	0.0	$\gamma_{f,E}$

$$F_d = \gamma_{f,G1} G_1 + \gamma_{f,G2} G_2 + \gamma_{f,Q1} Q_1 + \gamma_{f,Q2} Q_2 + \gamma_{f,Eo} (E_o + \gamma_{f,D} D_o) + \gamma_{f,Ee} (E_e + \gamma_{f,D} D_e)$$

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 environment 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 Eo

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 for the various permanent and variable actions discussed in 9.9 and for which values for different design situations are given in Table above;

$\gamma_{f,Eo}$ $\gamma_{f,Ee}$ are the partial action for the various permanent and variable actions discussed in 9.9 and for which values for different design situations are given in Table above;

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

Where no information on partial action factors that are specific to the case under consideration is available, these factors may be taken to be $\gamma_{f,E}$ = 1.35 and $\gamma_{f,D}$ = 1.25

NORSOK N-001, N003

N-001, Table 1 - Partial action factor for the limit state

Limit State	Action conmbinations	Permanent actions (G)	Variable actions (Q)	Environmental actions (E)[d]	Deformation actions (D)[e]
ULS	a[a]	1.3	1.3	0.7	1.0
ULS	b	1.0	1.0	1.3	1.0
SLS		1.0	1.0	1.0	1.0
ALS	Abnormal effect[b]	1.0	1.0	1.0	1.0
ALS	Damaged condition[c]	1.0	1.0	1.0	1.0
FLS		1.0	1.0	1.0	1.0

a For permanent actions and/or variable actions, an action factor of 1.0 shall be used where this gives the most unfavourable action effect

b Actions with annual probability of exceedance = 10^{-4}

c Environmental actions with annual probability of exceedance = 10^{-2}

d Earthquake shall be handled as environmental action within the limit state design for ULS and ALS (abnormal effect)

e Applicable for concrete structures

G = Permanent actions - the actions that will not vary in magnitude, position or direction during the time period considered.

Q = Variable actions - the actions originate from normal operation of the structure and vary in position magnitude and direction during the period considered.

E = Environmental actions

D = Deformation actions - the actions caused by deformations, imposed on the structure. They may be caused by the structure's function or the surrounding environmental conditions, or by construction processes.

Evironmental Condtions

API RP 2A

1. Typically, a 1-year to 5-year winter storm is used as an operating condition in the Gulf of Mexico.
2. Earthquake load, where applicable, should be imposed on the platform as a separate environmental loading conditions.

ISO 19901-1/ISO 19902

One of three methods is normally used for defining an environment that generates the extreme direct action Ee and generally also the extreme action effect, caused by the combined extreme wind, waves and current conditions

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.

Method a) has been used for Gulf of Mexico designs

b) has been used in the North Sea and many other areas

c) is a more recent development, suitable when a database of joint occurrences of wind, waves and current is available.

ISO-19902 A.9.10.3.2.1: Typically a 1 year to 5 year winter storm is used as an operating wind, wave and current condition in the Gulf of Mexico.

NORSOK

N-003, Table 4 - Conbination of environmental actions with expected mean values and annual probability of exceedance 10^{-2} and 10^{-4}

Limit State	Wind	Waves	Current	Ice	Snow	Earthquake	Sea Level
Ultimate Limit State	10^{-2}	10^{-2}	10^{-1}	-	-	-	10^{-2}
	10^{-1}	10^{-2}	10^{-2}	-	-	-	10^{-2}
	10^{-1}	10^{-1}	10^{-1}	10^{-2}	-	-	m
	-	-	-	-	10^{-2}		m
	-	-	-	-	-	10^{-2}	m
Accidental Limit State	10^{-4}	10^{-2}	10^{-1}	-	-	-	m*
	10^{-2}	10^{-4}	10^{-1}	-	-	-	m*
	10^{-1}	10^{-1}	10^{-4}	-	-	-	m*
	-	-	-	10^{-4}	-	-	m
	-	-	-	-	-	10^{-4}	m

m - Mean sea level

m* - mean water level, including the effect of possible storm surge

Seismic response analysis should be carried out for the most crtical water level

N-003, Table 5 - Charateristic actions and action combinations

	Temporary Conditions					Normal Operations				
	Servicecability limit state	Fatigue limit state	Ultimate limit state	Accidental limit state: Abnormal effect	Accidental limit state: Damaged conditions	Serviceability limit state	Fatigue limit state	Ultimate limit state	Accidental limit state: Abnormal effect	Accidental limit state: Damaged conditions
Permanent actions	EXPECTED VALUE									
Variable functional	SPECIFIED VALUE									
Environmeantal actions	Dependent on operational requirements	Expected action history	Value dependent on measures taken			Dependent on operational requirements	Expected action history	Annual probability of exceedance = 10^{-2}	Annual probability of exceedance = 10^{-4}	Annual probability of exceedance = 10^{-2}
Deformation actions	EXPECTED VALUE									
Accidental actions	Not applicable			Dependent on measures taken	Not applicable				Annual probability of exceedance:10^{-4}	Not applicable

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 414MPa. 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.

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

Tubular Members Code Provisions - TABLE 1

API WSD			API LRFD			ISO			NORSOK		
Stress/ Parameter	Formulation	Limits	Stress/ Parameter	Formulation	Limits	Stress/ Parameter	Formulation	Limits	Stress/ Parameter	Formulation	Limits
	3.2.1 AXIAL TENSION			**D.2.1 AXIAL TENSION**			**13.2.2 AXIAL TENSION**			**6.3.2 AXIAL TENSION**	
	$F_t = 0.6F_y$		f_t	$f_t \le \phi_t F_y$	$\phi_t = 0.95$	σ_t	$\sigma_t \le f_t/\gamma_{Rt}$	$\gamma_{Rt} = 1.05$, $f_t = f_y$		$N_{Sd} <= N_{t,Rd} = Af_y/\gamma_M$	$\gamma_M = 1.15$
	3.2.2 AXIAL COMPRESION			**D.2.2 AXIAL COMPRESION**			**13.2.3 AXIAL COMPRESION**			**6.3.3 AXIAL COMPRESION**	
			f_c	$f_c \le \phi_c F_{cn}$	$\phi_c = 0.85$	σ_c	$\sigma_c \le f_c/\gamma_{Rc}$	$\gamma_{Rc} = 1.18$		$N_{Sd} \le N_{c,Rd} = Af_c/\gamma_M$	see below
										$\gamma_M = 1.15$	$\lambda < 0.5$
	Column Buckling	$D/t \le 60$		Column Buckling			*Column Buckling*			$\gamma_M = 0.85+0.60\lambda_s$	$0.5 \le \lambda \le 1.0$
										$\gamma_M = 1.45$	$\lambda < 1.0$
	$F_a = \dfrac{(1-(Kl/r)^2/(2C_c^2))F_y}{5/3+3(Kl/r)/(8C_c)-(Kl/r)^3/(8C_c^3)}$	$Kl/r<C_c$								$\lambda_s = \sigma_{cSd}(f_y/f_{cle})^{1/2}/f_{cl} + (\sigma_{pSd}/f_h)^2(f_y/f_{he})^{1/2}$	
				$F_{cn} = (1-0.25\lambda^2)F_y$	$\lambda < 2^{0.5}$		$f_c = (1-0.278\lambda^2)f_{yc}$	$\lambda \le 1.34$		$f_c = (1 - 0.28\lambda^2)f_y$	$\lambda < 1.34$
	$F_a = 12\pi^2E/[23(Kl/r)^2]$	$Kl/r>C_c$		F_y/λ^2	$\lambda \ge 2^{0.5}$		$0.9f_{yc}/\lambda^2$	$\lambda > 1.34$		$0.9f_y/\lambda^2$	$\lambda > 1.34$
	$C_c= (2\pi^2E/F_y)^{1/2}$			$\lambda = [Kl/\pi r](F_y/E)^{0.5}$			$\lambda = (Kl/\pi r)(f_{yc}/E)^{1/2}$			$\lambda = kl/\pi i(f_{cl}/E)^{1/2}$	
	F_y in above eqn is lesser of F_{xe}, F_{xc}, or F_y			F_y in above eqn is lesser of F_{xe}, F_{xc}, or F_y			F_{yc} in above eqn is given by lesser of expressions below			f_{cl} in above eqn is given by lesser of expressions below	
	Local Buckling			Local Buckling			*Local Buckling*			*Local Buckling*	
Elastic Local Buckling Stress			Elastic Local Buckling Stress				$f_{yc} = f_y$	$f_y/f_{xe} \le 0.17$		$f_{cl} = f_y$	$f_y/f_{cle} \le 0.17$
	$F_{xe} = 2CEt/D$, $C=0.3$	$60< D/t <300$; $t>= 6$ mm		$F_{xe} = 2C_xE(t/D)$, $C_x=0.3$	$D/t < 300$;		$f_{yc} = (1.047 - 0.274f_y/f_{xe})f_y$	$0.17 < f_y/f_{xe}$		$f_{cl} = (1.047 - 0.274f_y/f_{cle})f_y$	$0.17 < f_y/f_{cle} < 1.911$
										$f_{cl} = f_{cle}$	$f_y/f_{cle} > 1.911$
Inelastic Local Buckling Stress			Inelastic Local Buckling Stress				$f_{xe} = 2C_xEt/D$	$C_x=0.3$		$f_{cle} = 2C_eEt/D$	$C_e=0.3$
	$F_{xc} = F_y[1.64-0.23(D/t)^{1/4}] \le F_{xe}$	$60< D/t <300$; $t>= 6$ mm		$F_{xc} = F_y[1.64-0.23(D/t)^{1/4}]$	$D/t > 60$						
	$F_{xc} = F_y$	for $(D/t) \le 60$		$F_{xc} = F_y$	$D/t \le 60$						
	3.2.3 BENDING			**D.2.3 BENDING**			**13.2.4 BENDING**			**6.3.4 BENDING**	
				$f_b \le \phi_b F_{bn}$	ϕ_b - 0.95		$\sigma_b = M/Z_e <= f_b/\gamma_{Rb}$	γ_{Rb} - 1.05		$M_{Sd} \le M_{Rd} = f_mW/\gamma_M$	see above for γ_M
	$F_b = 0.75F_y$	$D/t \le 10340/F_y$ (SI Units)		$F_{bn} = (Z/S)F_y$	$D/t \le 10340/F_y$ (F_y in MPa)		$f_b = (Z_p/Z_e)F_y$	$f_yD/Et \le 0.0517$		$f_m = (Z/W)F_y$	$f_yD/Et \le 0.0517$
	$F_b = [0.84-1.74F_yD/Et]F_y$	$10340/F_y < D/t \le 20680/F_y$		$F_{bn} = [1.13-2.58F_yD/Et](Z/S)F_y$	$10340/F_y < D/t \le 20680/F_y$		$f_b = [1.13-2.58F_yD/Et](Z_p/Z_e)F_y$	$0.0517 < f_yD/Et \le 0.1034$		$f_m = [1.13-2.58F_yD/Et](Z/W)F_y$	$0.0517 < f_yD/Et \le 0.1034$
	$F_b = [0.72-0.58F_yD/Et]F_y$	$20680/F_y< D/t \le 300$		$F_{bn} = [0.94-0.76F_yD/Et](Z/S)F_y$	$20680/F_y< D/t \le 300$		$f_b = [0.94-0.76F_yD/Et](Z_p/Z_e)F_y$	$0.1034< f_yD/Et \le 120f_y/E$		$f_m = [0.94-0.76F_yD/Et](Z/W)F_y$	$0.1034< f_yD/Et \le 120f_y/E$
				Z: plastic section modulus			$Z_e = \pi/64[D^4-(D-2t)^4]/(D/2)$			$W = (\pi/32)[D^4-(D-2t)^4]/D$	
				S: elastic section modulus			$Z_p = [D^3 - (D-2t)^3]/6$			$Z = [D^3 - (D-2t)^3]/6$	
	3.2.4 SHEAR			**D.2.4 SHEAR**			**13.2.5 SHEAR**			**6.3.5 SHEAR**	
Beam Shear			Beam Shear			Beam Shear			Beam Shear		
	$F_v = 0.4F_y$		f_v	$f_v \le \phi_v F_{vn}$	$f_v = 2V/A$ $\phi_v = 0.95$		$\tau_b = 2V/A \le f_v/\gamma_{Rv}$	$f_v = f_y/3^{0.5}$ $\gamma_{Rv} = 1.05$		$V_{Sd} \le V_{Rd} = 0.5Af_y/(3^{0.5}\gamma_M)$	$\gamma_M = 1.15$
					$F_{vn} = F_{vtn} = f_y/3^{0.5}$						
Torsional Shear			Torsional Shear			Torsional Shear			Torsional Shear		
	$F_{vt} = 0.4F_y$		f_{vt}	$f_{vt} \le \phi_{vt}F_{vtn}$	$f_{vt} = M_{vt}D/2I_p$		$\tau_t = M_{vt}D/2I_p \le f_v/\gamma_{Rv}$	$f_v = f_y/3^{0.5}$ $\gamma_{Rv} = 1.05$		$M_{TSd} \le 2I_pf_y/(D3^{0.5}\gamma_M)$	$\gamma_M = 1.15$

Table 3-2 Effective Length Factor

API WSD / API LRFD

3.3.1.d Member Slenderness / D.3.2.3 Slenderness Ratio and Reduction Factor

Situation	Effective Length Factor K	Reduction Factor C_m
Superstructure Legs		
Braced	1.0	a
Portal	K	a
Jacket legs and piling		
Grouted Composite Section	1.0	c
Ungrouted Jacket Legs	1.0	c
Ungrouted Piling Between Shim Points	1.0	b
Deck Truss Web Members		
In-Plane Action	0.8	b
Out-of-plane Action	1.0	a or b
Jacket Braces		
Face-to-face length of Main Diagonals	0.8	b or c
Face of leg to Centerline of Joint Length of K Braces	0.8	c
Longer Segment Length of:		
X Braces	0.9	c
Secondary Horizontals	0.7	c
Deck Truss Chord Members	1.0	a, b or c

a 0.85
b 0.6 - 0.4 (M_1/M_2), but not less than 0.4, nor more than 0.85
c 1 - 0.4(f_a/F_e'), or 0.85, whichever is less

K Use Effective Length Alignment Chart in Commentary of AISC

ISO 19902

13.5 Effective lenghts and moment reduction factors

Structural component	K	C_m
Topside Legs		
Braced	1.0	a
Portal	K	a
Structure legs and piling		
Grouted Composite Section	1.0	c
Ungrouted Jacket Legs	1.0	c
Ungrouted Piling Between Shim Points	1.0	b
Structure brace members		
Primary diagonals and horizontals	0.7	b or c
K-Braces	0.7	b or c
X-braces		
Longer segment length	0.8	b or c
Full length	0.7	b or c
Secondary horizontals	0.7	b or c

a 0.85
b 0.6 - 0.4 (M_1/M_2), but shall not be larger than 0.85
c 1 - 0.4(σ_c/f_e), or 0.85, whichever is less

K See Effective Length Alingment Chart

NORSOK

6.3.8.2 Axial compression and bending

Structural element	k	C_m
Superstucture Legs		
Braced	1	a
Portal	k	a
Jacket legs and piling		
Grouted Composite Section	1	c
Ungrouted Jacket Legs	1	c
Ungrouted Piling Between Shim Points	1	b
Jacket braces		
Primary diagonals and horizontals	0.7	b or c
K-Braces	0.7	c
Longer segments of X-braces	0.8	c
Secondary horizontals	0.7	c

a 0.85
b 0.6 - 0.4 (M_{1Sd}/M_{2Sd})
c 1 - 0.4(N_{Sd}/N_e), or 0.85, whichever is less

k Use Effective Length Alingment in Clause 12

Table 3-3 Tubular Member Design Check (Contd.)

COMPARISON - TABLE 2

API WSD		API LRFD		ISO		NORSOK	
Stress/ Parameter Formulation	Limits	Stress/ Parameter Formulation	Limits	Stress/ Parameter Formulation	Limits	Stress/ Parameter Formulation	Limits
3.2.5.b Hoop Buckling		**D.2.5.2 Hoop Buckling**		**13.2.6.2 Hoop Buckling**		**6.3.6.1 Hoop Buckling**	
$f_h = pD/2t \le F_{hc}/SF_h$		$f_h = pD/2t \le \phi_h F_{hc}$	$\phi_h = 0.80$	$\sigma_h = pD/2t \le f_h/\gamma_{Rh}$	$\gamma_{Rh} = 1.25$	$\sigma_{p,Sd} = p_{Sd}D/2t \le f_{hRd} = f_h/\gamma_M$	γ_M, See Table 1
$F_{hc} = F_{he}$	$F_{he} \le 0.55F_y$	$F_{hc} = 0.7F_y(F_{he}/F_y)^{0.4} \le F_y$	$F_{he} > 0.55F_y$	$F_h = F_y$	$2.44F_y < F_{he}$	$f_h = f_y$	$2.44F_y < F_{he}$
$F_{hc} = 0.45F_y + 0.18F_{he}$	$0.55F_y < F_{he} \le 1.6F_y$	$F_{hc} = F_{he}$	$F_{he} \le 0.55F_y$	$F_h = 0.7F_y(F_{he}/F_y)^{0.4}$	$0.55F_y < F_{he} \le 2.44F_y$	$f_h = 0.7f_y(f_{he}/f_y)^{0.4}$	$0.55F_y < F_{he} \le 2.44F_y$
$F_{hc} = 1.31F_y/(1.15 + F_y/F_{he})$	$0.55F_y < F_{he} < 1.6F_y$			$F_h = F_{he}$	$F_{he} \le 0.55F_y$	$f_h = f_{he}$	$F_{he} < 0.55F_y$
$F_{hc} = F_y$	$6.2F_y < F_{he}$						
$F_{he} = 2C_hEt/D$		$F_{he} = 2C_hEt/D$		$F_{he} = 2C_hEt/D$		$f_{he} = 2C_hEt/D$	
$C_h = 0.44t/D$	$1.6D/t < M$	$C_h = 0.44t/D$	$1.6D/t \le M$	$C_h = 0.44t/D$	$1.6D/t \le \mu$	$C_h = 0.44t/D$	$1.6D/t \le \mu$
$C_h = 0.44t/D + 0.21(D/t)^3/M^4$	$0.825D/t < M < 1.6D/t$	$C_h = 0.44t/D + 0.21(D/t)^3/M^4$	$0.825D/t \le M < 1.6D/t$	$C_h = 0.44t/D + 0.21(D/t)^3/\mu^4$	$0.825D/t \le \mu < 1.6D/t$	$C_h = 0.44t/D + 0.21(D/t)^3/\mu^4$	$0.825D/t \le \mu < 1.6D/t$
$C_h = 0.736/(M-0.636)$	$3.5 \le M < 0.825D/t$	$C_h = 0.737/(M-0.579)$	$1.5 \le M < 0.825D/t$	$C_h = 0.737/(\mu-0.579)$	$1.5 \le \mu < 0.825D/t$	$C_h = 0.737/(\mu - 0.579)$	$1.5 \le \mu < 0.825D/t$
$C_h = 0.755/(M-0.559)$	$1.5 \le M < 3.5$	$C_h = 0.8$	$M < 1.5$	$C_h = 0.8$	$\mu < 1.5$	$C_h = 0.8$	$\mu < 1.5$
$C_h = 0.8$	$M < 1.5$						
$M = L/D\ (2D/t)^{1/2}$		$M = L/D\ (2D/t)^{1/2}$		$\mu = L/D\ (2D/t)^{1/2}$		$\mu = L_r/D\ (2D/t)^{1/2}$	
3.3.2 TENSION and BENDING		**D.3.1 TENSION and BENDING**		**13.3.2 TENSION and BENDING**		**6.3.8.1 TENSION and BENDING**	
$f_a/(0.6F_y) + (f_{bz}^2 + f_{by}^2)^{1/2}/F_b < 1.0$		$1-\cos[(\pi/2)f_t/(\phi_tF_y)] + [f_{by}^2+f_{bz}^2]^{0.5}/\phi_bF_{bn} \le 1$	$\phi_t = \phi_b = 0.95$	$(\gamma_{Rt}\sigma_t/f_t + \gamma_{Rb}(\sigma_{by}^2+\sigma_{bz}^2)^{1/2}/f_b < 1.0$	$\gamma_{Rt} = 1.05$, $\gamma_{Rb} = 1.05$ $f_t = f_y$	$(N_{Sd}/N_{tRd})^{1.75} + (M_{ySd}^2 + M_{zSd}^2)^{1/2}/M_{Rd} \le 1.0$	
3.3.1 COMPRESSSION and BENDING		**D.3.2 COMPRESSSION and BENDING**		**13.3.3 COMPRESSSION and BENDING**		**6.3.8.2 COMPRESSSION and BENDING**	
$f_a/F_a + C_m(f_{bx}^2 + f_{by}^2)^{1/2}/[(1-f_a/F_e')F_b] < 1.0$ or $f_a/F_a + \{[C_{mx}f_{bx}^2/[(1-f_a/F_{ex}')]^2 + [C_{my}f_{by}2/[(1-f_a/F_{ey}')]2\}^{1/2}/F_b < 1.0$ and $f_a/(0.6F_y) + (f_{bz}^2 + f_{by}^2)^{1/2}/F_b < 1.0$		$f_c/(\phi_cF_{cn}) + \{[C_{my}f_{by}/(1-f_c/(\phi_cF_{ey})]^2 + [C_{mz}f_{bz}/(1-f_c/(\phi_cF_{ey})]^2\}^{0.5}/(\phi_bF_{bn}) \le 1.0$ and $1-\cos[(\pi/2)f_c/(\phi_cF_{xc})] + [f_{by}^2+f_{bz}^2]^{0.5}/(\phi_bF_{bn}) \le 1$		$\gamma_{Rc}\sigma_c/f_c + \gamma_{Rb}/f_b\{[C_{my}\sigma_{by}/(1-\sigma_c/f_{ey})]^2 + [C_{mz}\sigma_{bz}/(1-\sigma_c/f_{ez})]^2\}^{0.5} \le 1.0$ and $\gamma_{Rc}\sigma_c/f_{yc} + \gamma_{Rb}(\sigma_{by}^2 + \sigma_{bz}^2)^{0.5}/f_b \le 1.0$		$N_{Sd}/N_{cRd} + 1/M_{Rd}\{[C_{my}M_{ySd}/(1-N_{Sd}/N_{ey})]^2 + [C_{mz}M_{zSd}/(1-N_{Sd}/N_{ez})]^2\}^{0.5} \le 1.0$ and $N_{Sd}/N_{clRd} + (M_{ySd}^2 + M_{zSd}^2)^{1/2}/M_{Rd} \le 1.0$	
		$F_c < \phi_cF_{xc}$	$\phi_c = 0.85$ $\phi_b = 0.95$	$f_{ey} = \pi^2E/(K_yL_y/r)^2$ K and C_m from Section 3.3.1.d $\gamma_{Rc} = 1.18$ $\gamma_{Rb} = 1.05$	$f_{ez} = \pi^2E/(K_zL_z/r)^2$	$N_{clRd} = f_{cl}A/\gamma_M$ $N_{Ey} = \pi^2EA/[kl/i]_y^2$ k and C_m from Table 6-2	$N_{Ez} = \pi^2EA/[kl/i]_z^2$
$f_a/F_a + (f_{bz}^2 + f_{by}^2)^{1/2}/F_b < 1.0$	for $f_a/F_a \le 0.15$ only						
						6.3.8.3 SHEAR and BENDING	
						$M_{Sd}/M_{Rd} \le (1.4 - V_{Sd}/V_{Rd})^{0.5}$	$V_{Sd}/V_{Rd} \ge 0.4$
						$M_{Sd}/M_{Rd} \le 1.0$	$V_{Sd}/V_{Rd} < 0.4$
						6.3.8.4 SHEAR, BENDING and TORSION	
						$M_{Sd}/M_{RedRd} \le (1.4 - V_{Sd}/V_{Rd})^{0.5}$	$V_{Sd}/V_{Rd} \ge 0.4$
						$M_{Sd}/M_{RedRd} \le 1.0$	$V_{Sd}/V_{Rd} < 0.4$
						$M_{RedRd} = Wf_{mRed}/\gamma_M$ $f_{mRed} = f_m[1-3(\tau_{TSd}/f_d)^2]^{0.5}$	$\tau_{TSd} = M_{TSd}/(2\pi R^2t)$ $f_d = f_y/\gamma_M$

Table 3-4 Tubular Member Design Check (Contd.)

COMPARISON - TABLE 3

API WSD	API LRFD	ISO	NORSOK
Stress/ Parameter — Formulation	Stress/ Parameter — Formulation	Stress/ Parameter — Formulation	Stress/ Parameter — Formulation
3.3.3 TENSION, BENDING AND HYDROSTATIC PRESSURE $A^2 + B^2 + 2\nu\lvert A\rvert B \le 1.0$ $A = [(f_t + f_b - 0.5f_h)/F_y]SF_x$ $B = [f_h/F_{hc}]SF_h$ $\nu = 0.3$ SF_x = Axial Tension Safety Factor (see below) SF_h = Hoop Compression Safety Factor (see below) Design Condition / Loading: Axial Tension, Bending, Axial*** Compr., Hoop Compr. 1. Where the basic allowable stresses would be used, e.g., pressures which will definitely be encountered during the installation or life of the structure: 1.67; F_y/F_b**; 1.67 to 2.0; 2.0 2. Where the one-third increase in allowable stresses is appropriate, e.g., when considering interaction with storm loads: 1.25; $F_y/1.33F_b$; 1.25 to 1.5; 1.5	**D.3.3 TENSION, BENDING AND HYDROSTATIC PRESSURE** $A^2 + B^2 + 2\nu\lvert A\rvert B \le 1.0$ $A = [f_t + f_b - 0.5f_h]/(\phi_t F_y)$ $B = f_h/(\phi_h F_{hc})$ $\nu = 0.3$ $\eta = 5 - 4F_{hc}/F_y$ $\phi_t = 0.95$ $\phi_h = 0.80$	**13.4.2 TENSION, BENDING AND HYDROSTATIC PRESSURE** $\gamma_{Rt}\sigma_{tc}/f_{th} + \gamma_{Rb}(\sigma_{by}^2 + \sigma_{bz}^2)^{0.5}/f_{bh} \le 1.0$ $f_{th} = f_y[(1+0.09B^2-B^{2\eta})^{0.5} - 0.3B]$ $f_{bh} = f_b[(1+0.09B^2-B^{2\eta})^{0.5} - 0.3B]$ $B = \gamma_{Rh}\sigma_h/F_h$ $\quad B \le 1.0$ $\eta = 5 - 4f_h/f_y$ $\gamma_{Rt} = \gamma_{Rb} = 1.05$	**6.3.9.1 TENSION, BENDING AND HYDROSTATIC PRESSURE** Method A (σ_{aSd} in tension) If $\sigma_{aSd} \ge \sigma_{qSd}$ (net axial tension condition) $(\sigma_{aSd} - \sigma_{qSd})/f_{thRd} + (\sigma_{mySd}^2+\sigma_{mzSd}^2)^{0.5}/f_{mhRd} \le 1.0$ $f_{thRd} = f_y/\gamma_M[(1+0.09B^2-B^{2\eta})^{0.5} - 0.3B]$ $f_{mhRd} = f_m/\gamma_M[(1+0.09B^2-B^{2\eta})^{0.5} - 0.3B]$ γ_M = see Table 1 $\eta = 5 - 4f_h/f_y$ $B = \sigma_{pSd}/f_{hRd}$ $\quad B \le 1.0$ If $\sigma_{aSd} < \sigma_{qSd}$ (net axial compression condition) $\lvert\sigma_{aSd} - \sigma_{qSd}\rvert/f_{clRd} + (\sigma_{mySd}^2+\sigma_{mzSd}^2)^{0.5}/f_{mhRd} \le 1.0$ $f_{clRd} = f_{cl}/\gamma_M$ when $\sigma_{cSd} > 0.5 f_{he}/\gamma_M$ and $f_{cle} > 0.5f_{he,}$ the following eqn should be satisfied, $(\sigma_{cSd} - 0.5f_{he}/\gamma_M)/(f_{cle}/\gamma_M - 0.5f_{he}/\gamma_M) + [\sigma_{pSd}/(f_{he}/\gamma_M)]^2 \le 1.0$ $\sigma_{cSd} = \sigma_{mSd} + \sigma_{qSd} - \sigma_{aSd}$ $\sigma_{mSd} = (M_{zSd}^2 + M_{ySd}^2)^{0.5}/W$ Method B (σ_{acSd} is in tension) $\sigma_{acSd}/f_{thRd} + (\sigma_{mySd}^2+\sigma_{mzSd}^2)^{0.5}/f_{mhRd} \le 1.0$
3.3.4 COMPRESSION, BENDING AND HYDROSTATIC PRESSURE $[(f_a + 0.5f_h)/F_{xc}]SF_x + f_b/F_y(SF_b) \le 1.0$ $[f_h/F_{hc}]SF_h \le 1.0$ If $f_x > 0.5F_{he}$, then $(f_x - 0.5F_{ha})/(F_{aa}-0.5F_{ha}) + (f_h/F_{ha})^2 \le 1.0$ $F_{ha} = F_{he}/SF_h$ $F_{aa} = F_{xe}/SF_x$ $f_x = f_a + f_b + 0.5f_h$; f_x should reflect the maximum compressive stress combination	**D.3.4 COMPRESSION, BENDING AND HYDROSTATIC PRESSURE** $f_h = pD/2t \le \phi_h F_{hc}$ $F_{hc} = 0.7F_y(F_{he}/F_y)^{0.4} \le F_y$ $\quad F_{he} > 0.55F_y$ $F_{hc} = F_{he}$ $\quad F_{he} \le 0.55F_y$ and $f_c/(\phi_c F_{cn}) + \{[C_{my}f_{by}/(1-f_c/(\phi_c F_{ey}))]^2 + [C_{mz}f_{bz}/(1-f_c/(\phi_c F_{ey}))]^2\}^{0.5}/(\phi_b F_{bn}) \le 1.0$ and $1-\cos[(\pi/2)f_c/(\phi_c F_{xc}] + [f_{by}^2+f_{bz}^2]^{0.5}/(\phi_b F_{bn}) \le 1$ and $f_c < \phi_c F_{xc}$ if $f_x > 0.5\ \phi_h F_{he}$, then $(f_x-0.5\phi_h F_{he})/(\phi_c F_{xe}-0.5\phi_h F_{he}) + [f_h/(\phi_h F_{he})]^2 \le 1.0$ $f_x = f_c + f_b + 0.5f_h$ $\phi_b = 0.95$ $\quad \phi_c = 0.85$ $\quad \phi_h = 0.80$	**13.4.3 COMPRESSION, BENDING AND HYDROSTATIC PRESSURE** $\gamma_{Rc}\sigma_{cc}/f_{yc} + \gamma_{Rb}[\sigma_{by}^2 + \sigma_{bz}^2]/f_{bh} \le 1.0$ and $\gamma_{Rc}\sigma_c/f_{ch} + \gamma_{Rb}/f_{bh}\{[C_{my}\sigma_{by}/(1-\sigma_c/f_{ey})]^{2+}[C_{mz}\sigma_{bz}/(1-\sigma_c/f_{ez})]^2\}^{0.5} \le 1.0$ $f_{ch} = 0.5f_{yc}\{(1.0-0.278\lambda^2) - 2\sigma_q/f_{yc} + [(1.0-0.278\lambda^2)^2 + 1.12\lambda^2\sigma_q/f_{yc}]^{0.5}\}$ for $\lambda \le 1.34[(1-2\sigma_q/f_{yc})^{-1}]^{0.5}$ $f_{ch} = 0.9f_{yc}/\lambda^2$ $\quad$ for $\lambda > 1.34[(1-2\sigma_q/f_{yc})^{-1}]^{0.5}$ If $\sigma_x > 0.5f_{he}/\gamma_{Rh}$ and $f_{xe}/\gamma_{Rc} > 0.5f_{he}/\gamma_{Rh,}$ the following eqn shall also be satisfied: $(\sigma_x-0.5f_{he}/\gamma_{Rh})/(f_{xe}/\gamma_{Rc}-0.5f_{he}/\gamma_{Rh}) + (\gamma_{Rh}\sigma_h/f_{he})^2 \le 1.0$ $\gamma_{Rh} = 1.25$ $\quad \gamma_{Rc} = 1.18$	**6.3.9.2 COMPRESSION, BENDING AND HYDROSTATIC PRESSURE** Method A (σ_{aSd} is in compression) $\sigma_{aSd}/f_{chRd} + 1/f_{mhRd}\{[C_{my}\sigma_{mySd}/(1-\sigma_{aSd}/f_{Ey})]^{2+}[C_{mz}\sigma_{mzSd}/(1-\sigma_{aSd}/f_{Ez})]^2\}^{0.5} \le 1.0$ and $(\sigma_{aSd} + \sigma_{qSd})/f_{clRd} + (\sigma_{mySd}^2+\sigma_{mzSd}^2)^{0.5}/f_{mhRd} \le 1.0$ $f_{Ey} = \pi^2E/[kl/i]_y^2$ $\quad f_{Ez} = \pi^2E/[kl/i]_z^2$ $f_{chRd} = 0.5(f_{cl}/\gamma_M)[\xi - 2\sigma_{qSd}/f_{cl} + (\xi^2 + 1.12\lambda^2\sigma_{qSd}/f_{cl})^{0.5}]$ for $\lambda < 1.34[(1-2\sigma_{qSd}/f_{cl})^{-1}]^{0.5}$ $f_{chRd} = 0.9f_{cl}/(\lambda^2\gamma_M)$ $\quad$ for $\lambda \ge 1.34[(1-2\sigma_{qSd}/f_{cl})^{-1}]^{0.5}$ $\lambda = kl/(\pi i)(f_{cl}/E)^{1/2}$ when $\sigma_{cSd} > 0.5 f_{he}/\gamma_M$ and $f_{cle} > 0.5f_{he,}$ the following eqn should be satisfied, $(\sigma_{cSd} - 0.5f_{he}/\gamma_M)/(f_{cle}/\gamma_M - 0.5f_{he}/\gamma_M) + [\sigma_{pSd}/(f_{he}/\gamma_M)]^2 \le 1.0$ $\sigma_{cSd} = \sigma_{mSd} + \sigma_{qSd} + \sigma_{aSd}$ Method B (σ_{acSd} in Compression) (a) $(\sigma_{acSd} > \sigma_{qSd})$ $(\sigma_{acSd}-\sigma_{qSd})/f_{chRd} + 1/f_{mhRd}\{[C_{my}\sigma_{mySd}/(1-(\sigma_{acSd}-\sigma_{qSd})/f_{Ey})]^{2+}[C_{mz}\sigma_{mzSd}/(1-(\sigma_{acSd}-\sigma_{qSd})/f_{Ez})]^2\}^{0.5} \le 1.0$ and $\sigma_{acSd}/f_{clRd} + (\sigma_{mySd}^2+\sigma_{mzSd}^2)^{0.5}/f_{mhRd} \le 1.0$ when $\sigma_{cSd} > 0.5 f_{he}/\gamma_M$ and $f_{cle} > 0.5f_{he,}$ the following eqn should be satisfied, $(\sigma_{cSd} - 0.5f_{he}/\gamma_M)/(f_{cle}/\gamma_M - 0.5f_{he}/\gamma_M) + [\sigma_{pSd}/(f_{he}/\gamma_M)]^2 \le 1.0$ $\sigma_{cSd} = \sigma_{mSd} + \sigma_{acSd}$ $\sigma_{mSd} = (M_{zSd}^2 + M_{ySd}^2)^{0.5}/W$ (b) $(\sigma_{acSd} \le \sigma_{qSd})$ $\sigma_{acSd}/f_{clRd} + (\sigma_{mySd}^2+\sigma_{mzSd}^2)^{0.5}/f_{mhRd} \le 1.0$ when $\sigma_{cSd} > 0.5 f_{he}/\gamma_M$ and $f_{cle} > 0.5f_{he,}$ the following eqn should be satisfied, $(\sigma_{cSd} - 0.5f_{he}/\gamma_M)/(f_{cle}/\gamma_M - 0.5f_{he}/\gamma_M) + [\sigma_{pSd}/(f_{he}/\gamma_M)]^2 \le 1.0$ $\sigma_{cSd} = \sigma_{mSd} + \sigma_{acSd}$ $\sigma_{mSd} = (M_{zSd}^2 + M_{ySd}^2)^{0.5}/W$

Table 3-5 Tubular Joint Check

API WSD

4.3.1 Validity Range

$0.2 \le \beta \le 1.0$

$10 \le \gamma \le 50$

$30^\circ \le \theta \le 90^\circ$

$F_y \le 72$ ksi (500 MPa)

$g/D > -0.6$ (for K joints)

4.3.2 Basic Capacity

$P_a = Q_u Q_f F_{yc} T^2/(FS\sin\theta)$

$M_a = Q_u Q_f F_{yc} T^2 d/(FS\sin\theta)$

(plus 1/3 increase in both cases where applicable)

$FS = 1.60$

4.3.6 Strength Check

$IR = |P/P_a| + (M/M_a)_{ipb}^2 + |M/M_a|_{opb} \le 1.0$

4.3.3 Strength Factor Q_u

Joint	Brace Load: Axial Tension	Axial Compr.	IPB	OPB
K	$(16+1.2\gamma)\beta^{1.2}Q_g$ but $\le 40\beta^{1.2}Q_g$		$(5+0.7\gamma)\beta^{1.2}$	$2.5+(4.5+0.2\gamma)\beta^{2.6}$
T/Y	30β	$2.8+(20+0.8\gamma)\beta^{1.6}$ but $\le 2.8+36\beta^{1.6}$		
X	23β for $\beta \le 0.9$; $20.7+(\beta-0.9)(17\gamma-220)$ for $\beta>0.9$	$[2.8+(1.2+0.1\gamma)\beta]Q_\beta$		

$Q_\beta = 0.3/[\beta(1-0.833\beta)]$ for $\beta > 0.6$

$Q_\beta = 1.0$ for $\beta \le 0.6$

$Q_g = 1+0.2[1-2.8g/D]^3$ for $g/D \ge 0.05$, but $Qg \ge 1.0$

$Q_g = 0.13+0.65\phi\gamma^{0.5}$ for $g/D \le -0.05$ where $\phi = tF_{yb}/(Tf_y)$

4.3.4 Chord Load Factor Q_f

$Q_f = [1+C_1(FSP_c/P_y)-C_2(FSM_{ipb}/M_p)-C_3A^2]$

$A = [(FSP_c/P_y)^2 + (FSM_c/M_p)^2]^{0.5}$

where 1/3 increase applicable, FS=1.20

Value for C_1, C_2, C_3

Joint Type	C_1	C_2	C_3
K joints under brace axial loading	0.2	0.2	0.3
T/Y joints under brace axial loading	0.3	0	0.8
X join under brace axial loading $\beta \le 0.9$	0.2	0	0.5
$\beta = 1.0$	-0.2	0	0.2
All joints under brace moment loading	0.2	0	0.4

API LRFD

E.1 CONNECTIONS OF TENSION AND COMPRESSION MEMBERS

$[\gamma\tau\sin(\theta)]/[11+1.5/\beta](F_{yb}/F_y) \le 1.0$

E.3 TUBULAR JOINTS

Ultimate Capacity

$P_{uj} = F_y T^2 Q_u Q_f/\sin(\theta)$

$M_{uj} = F_y T^2(0.8d)Q_u Q_f/\sin(\theta)$

Strength Check

$P_D < \phi_j P_{uj}$

$M_D < \phi_j M_{uj}$

$1-\cos[\pi/2(P_D/\phi_j P_{uj})] + [(M_D/\phi_j M_{uj})_{ipb}^2 + (M_D/\phi_j M_{uj})_{opb}^2]^{0.5} \le 1.0$

$\phi_j = 0.95$ except for tension loaded Y, T and X joints when $\phi_j = 0.90$

Values for Q_u

Joint	Axial Tension	Axial Compr.	IPB	OPB
K	$(3.4+19\beta)Q_g$		$(3.4+19\beta)$	$(3.4+19\beta)Q_g$
T & Y	$(3.4+19\beta)$		$(3.4+19\beta)$	$(3.4+19\beta)Q_g$
Cross Joint W/O diaphragms	$(3.4+19\beta)$	$(3.4+19\beta)Q_\beta$		
Cross Joint W diaphragms	$(3.4+19\beta)$			

$Q_\beta = 0.3/[\beta(1-0.833\beta)]$ for $\beta > 0.6$

$Q_\beta = 1.0$ for $\beta \le 0.6$

$Q_g = 1.8-0.1g/T$ for $\gamma \le 20$

$Q_g = 1.8-4g/D$ for $\gamma > 20$

but $Qg \ge 1.0$

$Q_f = 1.0-\lambda A^2$

λ = 0.030 for brace axial stress; 0.045 for brace IPB stress; 0.021 for brace OPB stress

$A = (f_{ax}^2 + f_{ipb}^2 + f_{opb}^2)^{1/2}/(\phi_q F_y)$

$\phi_q = 0.95$

Set $Q_f = 1.0$ when all extreme fibre stresses in the chord are tensile

ISO 19902

14.3.1 Validity Range

$0.2 \le \beta \le 1.0$

$10 \le \gamma \le 50$

$30^\circ \le \theta \le 90^\circ$

$f_y \le 500$ N/mm^2

$gT > -1.2\gamma$ (for K joints)

14.3.2 Basic Joint Strength

$P_{uj} = Q_u Q_f f_y T^2/(\sin\theta)$

$M_{uj} = Q_u Q_f f_y T^2 d/(\sin\theta)$

14.3.6 Strength Check

$U_j = |P_B/P_d| + (M_B/M_d)_{ipb}^2 + |M_B/M_d|_{opb} \le 1.0$ for all joints

$U_j = |P_B/P_d| + (M_B/M_d)_{ipb}^2 + |M_B/M_d|_{opb} \le U_b/\gamma_{zj}$ for critical joints

U_b = the utilization of brace at the end adjoining the joint, which may concervatively be taken as the maximum utilization along the brace or even more conservatively as unity

γ_{zj} = 1.17 normally; may be relaxed to a value within the range 1.0-1.17 if this can be justified by designer, giving a total resistance factor between 1.05 and 1.23

$P_d = P_{uj}/\gamma_{Rj}$

$M_d = M_{uj}/\gamma_{Rj}$ $\gamma_{Rj} = 1.05$

Values for Q_u

Joint	Axial Tension	Axial Compression	IPB	OPB
K	$(1.9+19\beta)Q_\beta^{0.5}Q_g$	$(1.9+19\beta)Q_\beta^{0.5}Q_g$	$4.5\beta\gamma^{0.5}$	$3.2\gamma^{(0.5\beta 2)}$
Y	30β	$(1.9+19\beta)Q_\beta^{0.5}$	$4.5\beta\gamma^{0.5}$	$3.2\gamma^{(0.5\beta 2)}$
X	23β for $\beta \le 0.9$; $20.7+(\beta-0.9)(17\gamma-220)$ for $\beta>0.9$	$[2.8+(12+0.1\gamma)\beta]Q_\beta$	$4.5\beta\gamma^{0.5}$	$3.2\gamma^{(0.5\beta 2)}$

$Q_\beta = 0.3/[\beta(1-0.833\beta)]$ for $\beta > 0.6$

$Q_\beta = 1.0$ for $\beta \le 0.6$

$Q_g = 1.9-0.7\gamma^{-0.5}(g/T)^{0.5}$ for $g/T \ge 2.0$, but $Q_g \ge 1.0$

for $-2.0 < g/T < +2.0$, the gap factor Q_g may be found by linear interpolation.

$Q_g = 0.13+0.65\phi\gamma^{0.5}$ for $g/T \le -2.0$ where $\phi = tf_{yb}/(Tf_y)$

$Q_f = 1.0-\lambda q_A^2$

λ = 0.030 for brace axial stress; 0.045 for brace IPB stress; 0.021 for brace OPB stress

$q_A = [C_1(P_c/P_y)^2+C_2(M_c/M_p)_{ipb}^2+C_2(M_c/M_p)_{opb}^2]^{0.5}\gamma_{Rq}$

$\gamma_{Rq} = 1.05$

Values for the coefficient C_1 and C_2

Joint Type	C_1	C_2
forces	25	11
X joints for calculating strength against brace axial	20	22
K joints for calculating strength against brace axial	14	43
All Joints for calculating strength against brace moments	25	43

NORSOK N-004

6.4.3.1 Validity Range

$0.2 \le \beta \le 1.0$

$10 \le \gamma \le 50$

$30^\circ \le \theta \le 90^\circ$

$g/D > -0.6$ (for K joints)

$F_y \le 72$ ksi (500 MPa)

6.4.3.2 Basic Resistance

$N_{Rd} = Q_u Q_f f_y T^2/(\gamma_M\sin\theta)$

$M_{Rd} = Q_u Q_f f_y T^2 d/(\gamma_M\sin\theta)$

$\gamma_M = 1.15$

6.4.3.6 Strength Check

$N_{Sd}/N_{Rd} + (M_{ySd}/M_{yRd})^2 + M_{zSd}/M_{zRd} \le 1$

Values for Q_u

Joint	Axial Tension	Axial Compression	IPB	OPB
K	$(1.9+19\beta)Q_\beta^{0.5}Q_g$	$(1.9+19\beta)Q_\beta^{0.5}Q_g$	$4.5\beta\gamma^{0.5}$	$3.2\gamma^{(0.5\beta 2)}$
Y	30β	$(1.9+19\beta)Q_\beta^{0.5}$	$4.5\beta\gamma^{0.5}$	$3.2\gamma^{(0.5\beta 2)}$
X	23β for $\beta<=0.9$; $21+(\beta-0.9)(17\gamma-220)$ for $\beta>0.9$	$2.8+14\beta Q_\beta$	$4.5\beta\gamma^{0.5}$	$3.2\gamma^{(0.5\beta 2)}$

$Q_\beta = 0.3/[\beta(1-0.833\beta)]$ for $\beta > 0.6$

$Q_\beta = 1.0$ for $\beta \le 0.6$

$Q_g = 1.9-(g/D)^{0.5}$ for $g/T \ge 2.0$, but $Q_g \ge 1.0$

but ≥ 1.0

$Q_g = 0.13+0.65\phi\gamma^{0.5}$ for $g/T \le -2.0$ where $\phi = tf_{yb}/(Tf_y)$

$Q_f = 1.0-\lambda A^2$

λ = 0.030 for brace axial stress; 0.045 for brace IPB stress; 0.021 for brace OPB stress

$A^2 = C_1(\sigma_{aSd}/f_y)^2 + C_2[(\sigma_{mySd}^2 + \sigma_{mzSd}^2)/1.62f_y^2]$

Values for C_1 and C_2

Joint Type	C_1	C_2
T/Y joints under brace axial forces	25	11
X joints under brace axial forces	20	22
K joints under balanced brace axial forces	20	22
All Joints under brace moments	25	30

API WSD	API LRFD	ISO 19902	NORSOK N-004
3.4.1.c Unstiffened Cone-cylinder Junctions	**D.4.1.3 Unstiffened Cone-Cylinder Junctions**	**13.6 Conical transitions**	**6.5 Strength of conical transitions**
1. Longitudinal Stress	1. Longitudinal Stress	$\alpha \leq 30$ deg	
$f_b' = [0.6t\,(Dt + Dt_c)^{0.5}(f_a + f_b)\tan\alpha]/t_e^2$	$f_b' = [0.6t\,(Dt + Dt_c)^{0.5}(f_a + f_b)\tan\alpha]/t_e^2$	**13.6.2.1 Equivalent axial stress**	**6.5.2.1 Equivalent design axial stress in the cone design**
2. Hoop Stress	2. Hoop Stress	$\sigma_{a,eq} = (\sigma_{ac} + \sigma_{bc})/\cos\alpha$	$\sigma_{equSd} = (\sigma_{asSd} + \sigma_{mcSd})/\cos\alpha$
$f_h' = 0.45\,(D/t)^{0.5}\,(f_a + f_b)\tan\alpha$	$f_h' = 0.45\,(D/t)^{0.5}\,(f_a + f_b)\tan\alpha$	$\sigma_{ac} = P_s/[\pi(D_s - t_c\cos\alpha)t_c]$ $\sigma_{bc} = 4M_s/[\pi(D_s - t_c\cos\alpha)^2 t_c]$	$\sigma_{acSd} = N_{Sd} / [\pi(D_s - t_c\cos\alpha)t_c]$ $\sigma_{mcSd} = M_{Sd}/[0.25\pi(D_s - t_c\cos\alpha)^2 t_c]$
$f_h' \leq 0.6\,F_y$	$f_h' \leq \phi_t\,F_y$	**13.6.2.2.2 Bending stress**	**6.5.2.2 Local bending stress at unstiffened junctions**
$f_h' \leq 0.5\,F_{hc}$	$f_h' \leq \phi_h\,F_{hc}$		
$F_{hc} = F_{he}$ $F_{he} \leq 0.55F_y$ $F_{hc} = 0.45F_y + 0.18F_{he}$ $0.55F_y < F_{he} \leq 1.6F_y$ $F_{hc} = 1.31F_y/(1.15 + F_y/F_{he})$ $0.55F_y < F_{he} < 1.6F_y$ $F_{hc} = F_y$ $6.2F_y < F_{he}$	$F_{hc} = F_{he}$ $F_{he} \leq 0.55F_y$ $F_{hc} = 0.7F_y(F_{he}/F_y)^{0.4} \leq F_y$ $F_{he} > 0.55F_y$ $F_{he} = 0.4\,Et/D$	$\sigma_{b,jt} = \{0.6t\,[D_j(t + t_c)]^{0.5}\,(\sigma_{at} + \sigma_{bt})\tan\alpha\}/t^2$ $\sigma_{b,jc} = \{0.6t\,[D_j(t + t_c)]^{0.5}\,(\sigma_{at} + \sigma_{bt})\tan\alpha\}/t_c^2$ **13.6.2.2.3 Hoop stresses**	$\sigma_{mltSd} = 0.6t\,[D_j\,(t + t_c)]^{0.5}\,[(\sigma_{atSd} + \sigma_{mtSd})/t^2]\tan\alpha$ $\sigma_{mlcSd} = 0.6t\,[D_j\,(t + t_c)]^{0.5}\,[(\sigma_{atSd} + \sigma_{mtSd})/t_c^2]\tan\alpha$ **6.5.2.3 Hoop stress at unstiffened junctions**
$F_{he} = 0.4\,Et/D$		$\sigma_{h,t} = 0.45\,(D_j/t)^{0.5}\,(\sigma_{at} + \sigma_{bt})\tan\alpha$ $\sigma_{h,c} = 0.45\,(D_j/t)^{0.5}\,(t/t_c)\,(\sigma_{at} + \sigma_{bt})\tan\alpha$	$\sigma_{hcSd} = 0.45\,(D_j/t)^{0.5}\,(\sigma_{atSd} + \sigma_{mtSd})\tan\alpha$
		13.6.3.2 Strength requirements without hydrostatic pressure	**6.5.3 Strength requirements without external hydrostatic pressure**
		13.6.3.2 Local buckling	**6.5.3.1 Local buckling under axial compression**
		$\sigma_{a,eq} \leq (f_{yc}/\gamma_{Rc})$	$\sigma_{equSd} \leq (f_{clc}/\gamma_M)$ f_{clc}- local buckling strength of conical transition
		13.6.3.3 Junction yielding <u>if σ_{max} is tensile:</u> $[\sigma_{max}^2 + (\sigma_j)^2 - \sigma_j\sigma_{max}]^{0.5} \leq f_y/\gamma_{Rt}$ <u>if σ_{max} is compressive:</u> $[\sigma_{max}^2 + (\sigma_j)^2 - \sigma_j\lvert\sigma_{max}\rvert]^{0.5} \leq f_y/\gamma_{Rt}$	$f_{clc} = f_y$ $f_y/f_{cle} \leq 0.17$ $f_{clc} = (1.047 - 0.274f_y/f_{cle})f_y$ $0.17 < f_y/f_{cle} < 1.911$ $f_{clc} = f_{cle}$ $f_y/f_{cle} > 1.911$ $f_{cle} = 2C_eEt/D_e$ C_e=0.3 $D_e = D_s/\cos\alpha$
		$\sigma_{max} = \sigma_{at} + \sigma_{bt} + \sigma_{bjt}$ for tubular side of the $\sigma_{max} = (\sigma_{ac} + \sigma_{bc})/\cos\alpha + \sigma_{bjc}$ for cone side of the junction	**6.5.3.2 Junction yielding** <u>if σ_{totSd} is tensile:</u> $(\sigma_{totSd}^2 + \sigma_{hcSd}^2 - \sigma_{hcSd}\sigma_{totSd})^{0.5} \leq f_y/\gamma_M$ <u>if σ_{totSd} is compressive:</u> $(\sigma_{totSd}^2 + \sigma_{hcSd}^2 + \sigma_{hcSd}\lvert\sigma_{totSd}\rvert)^{0.5} \leq f_y/\gamma_M$
		13.6.3.4 Junction buckling <u>if σ_{max} is tensile:</u> $A^2 + B^{2\eta} + 2\nu AB \leq 1.0$ $A = \gamma_{Rt}\,\sigma_{max}/f_y$ $B = \gamma_{Rh}\sigma_j/f_h$ <u>if σ_{max} is compressive:</u> $\sigma_{max} \leq f_h/\gamma_{Rh}$ $\sigma_j \leq f_h/\gamma_{Rh}$	$\sigma_{totSd} = \sigma_{atSd} + \sigma_{mtSd} + \sigma_{mltSd}$ $\sigma_{totSd} = (\sigma_{acSd} + \sigma_{mcSd})/\cos\alpha + \sigma_{mlcSd}$ **6.5.3.3 Junction buckling** <u>if σ_{totSd} is tensile:</u> $a^2 + b^{2\eta} + 2\nu ab \leq 1.0$ $a = \gamma_M\sigma_{totSd}/f_y$ $b = \gamma_M\sigma_{hcSd}/f_h$ <u>if σ_{totSd} is compressive:</u> $\sigma_{totSd} \leq f_{clj}/\gamma_M$ $\sigma_{hcSd} \leq f_h/\gamma_M$
		13.6.4 Strength requirements with external hydrostatic pressure	
		13.6.4.1 Hoop buckling	
		Similar to member design (Sec. 13.4), substituting:	**6.5.4 Strength requirements with external hydrostatic pressure**
		$D \Rightarrow D_{eq} = D'/\cos\alpha$ D' - diameter at the larger end of the cone $\sigma_{ac} \Rightarrow \sigma_{tc}$ or σ_{cc}, σ_t or σ_c (as appropriate)	**6.5.4.1 Hoop buckling** $(\sigma_{equSd} + \sigma_{qSd})/f_{clRd} + (\sigma_{mySd}^2 + \sigma_{mzSd}^2)/f_{mhRd} \leq 1.0$

		13.6.4.2 Junction yielding and buckling
		
		$\sigma_{hJ} = \sigma_j + \sigma_h$
		
		if σ_{hJ} is tensile:
		
		$[\sigma_{max}^2 + (\sigma_{hJ})^2 - \sigma_{hJ}\sigma_{max}]^{0.5} \le f_y/\gamma_{Rt}$, for tensile σ_{max}
		
		$[\sigma_{max}^2 + (\sigma_{hJ})^2 - \sigma_{hJ}\lvert\sigma_{max}\rvert]^{0.5} \le f_y/\gamma_{Rt}$, for compresive σ_{max}
		
		if σ_{hJ} is compressive:
		
		$A^2 + B^{2\eta} + 2\nu AB \le 1.0$, for tensile σ_{max}
		$A = \gamma_{Rt}\,\sigma_{max}/f_y$
		$B = \gamma_{Rh}\sigma_j/f_h$
		
		$\sigma_{max} \le f_h/\gamma_{Rh}$, for compresive σ_{max}
		$\sigma_{hJ} \le f_h/\gamma_{Rh}$
			$(\sigma_{equSd}-\sigma_{qSd})/f_{chRd} + 1/f_{mhRd}\{[C_{my}\sigma_{mySd}/(1-(\sigma_{acSd}-\sigma_{qSd})/f_{Ey})]^2 +$
			$+ [C_{mz}\sigma_{mzSd}/(1-(\sigma_{acSd}-\sigma_{qSd})/f_{Ez})]^2\}^{0.5} \le 1.0$
			
			6.5.4.2 Junction yielding and buckling
			
			$\sigma_{hJSd} = \sigma_{hcSd} + \sigma_{hSd}$
			
			if σ_{hJSd} is tensile:
			
			$(\sigma_{totSd}^2 + \sigma_{hJSd}^2 - \sigma_{hcSd}\sigma_{totSd})^{0.5} \le f_y/\gamma_M$, for tensile σ_{totSd}
			
			$(\sigma_{totSd}^2 + \sigma_{hJSd}^2 + \sigma_{hcSd}\lvert\sigma_{totSd}\rvert)^{0.5} \le f_y/\gamma_M$, for compressive σ_{totSd}
			
			if σ_{hJSd} is compressive:
			
			$a^2 + b^{2\eta} + 2\nu ab \le 1.0$, for tensile σ_{totSd}
			$a = \gamma_M\sigma_{totSd}/f_y$
			$b = \gamma_M\sigma_{hJSd}/f_h$
			
			$\sigma_{totSd} \le f_{cl}/\gamma_M$ and $\sigma_{hJSd} \le f_h/\gamma_M$, for compressive σ_{totSd}

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.

Table 4-1 NORSOK N-004 Design Fatigue Factors

Table 8-1 Design fatigue factors

Classification of structural components based on damage consequence	Access for inspection and repair		
	No access or in the splash zone	Accessible	
		Below splash zone	Above splash zone
Substantial consequences	10	3	2
Without substantial consequences	3	2	1

"Substantial consequences" in this context means that failure of the joint will entail
danger of loss of human life;
significant pollution;
major financial consequences.

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.

API RP 2A refers to ISO 19902. ISO 19902 and DNV-RP-C203 give the similar guidance. They all refer to BS 7910 "Guide on Methods for Assessing the Acceptability of Flaws in Fusion Welded Structures". API RP 2A refers to 1999 edition, ISO 19902 refers to 1991edition and DNV-RP-C203 refers to 2005 edition.

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).

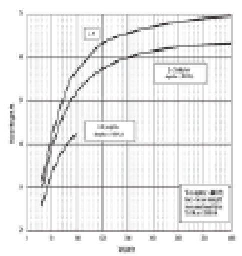
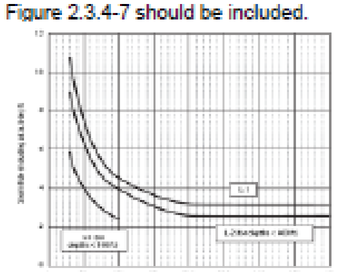
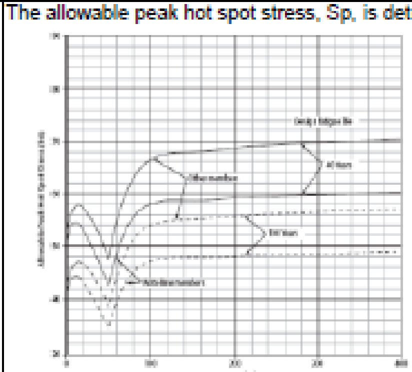
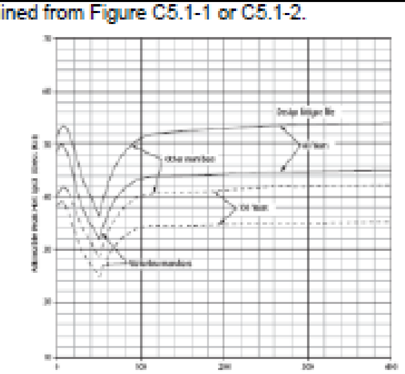
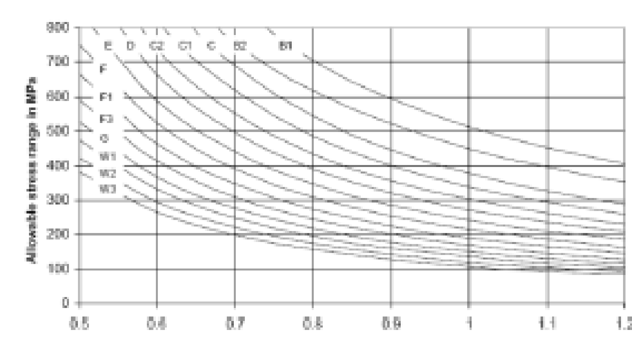
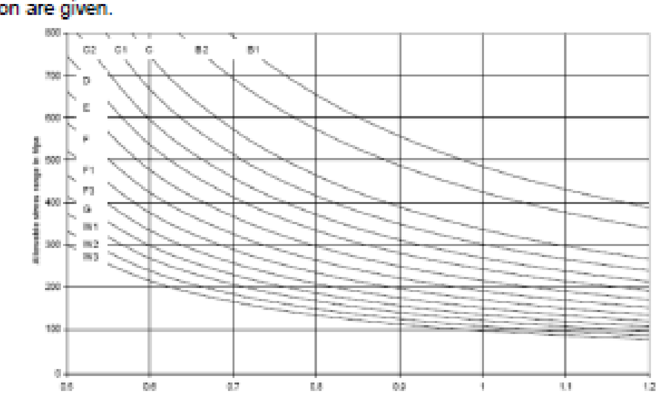
Detail fatigue assumptions, loading definitions, hot spot stress range calculation, stress concentration factor formulas, S-N curves for tubular joints, and required DFF values are specified in all three codes. In addition, details of the spectral analysis, utilization of fracture mechanics, and fatigue 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 - Simplified Fatigue

	API RP 2A - WSD	NORSOK N-001, N003/DNV-RP-C203
	Simplified fatigue analysis (section 5.1 and C 5.1)	**DNV-RP-C203 5. Simplified fatigue analysis**
Limitations	- Have been calibrated for the design wave climate - May be applied to tubular joints in Category L-3 template type platforms as defined in Section 1.7 1. Are constructed of notch-tough ductile steels 2. Have redundant, inspectable strutcural framing 3. Have natural periods less than 3 seconds - Particularly useful for preliminary design of all structure categories and types, in water depths up to 400 feet (122 m)	These design charts have been derived based on an assumption of an allowable fatigue damage e=1.0 during 10^8 cycles (20 years service life which corresponds to an average period 6.3 sec).
Evironmental Data	Fatigue design wave: - Fatigue design wave is the reference level -wave for the platform water depth as defined in Figure 2.3.4-3.  - Wave Foces - 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 0.88. - Wave 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 hot stress at each member end for the fatigue design wave.	
	Tide as defined in Figure 2.3.4-7 should be included.  Figure 2.3.4-7—Guideline Storm Tide vs. MLLW and Platform Category, Gulf of Mexico, North of 27° N and West of 86° W	
Allowable Peak Hot Spot Stresses	The allowable peak hot spot stress, Sp, is determined from Figure C5.1-1 or C5.1-2.  Figure C5.1-1—Allowable Peak Hot Spot Stress, S_p (AWS Level I)  Figure C5.1-2—Allowable Peak Hot Spot Stress, S_p (AWS Level II)	-Design charts for steel components in air and in seawater with cathodic protection are given.  Figure 4.3 Allowable extreme stress range during 10^8 cycles for components in air 

Peak Hot Spot Stress for the Fatigue Design Wave

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:

$$|SCF_{ax} f_{ax}| + \sqrt{[(SCF_{ipb} f_{ipb})^2 + (SCF_{opb} f_{opb})^2]} \quad (C5.5\text{-}1)$$

where f_{ax}, f_{ipb} and f_{opb} are the nominal member end axial, in-plane bending and out-of-plane bending stresses; SCF_{ax}, SCF_{ipb} and SCF_{opb} are the corresponding stress concentrations factor for axial, in-plane bending and out-of-plane bending stresses for the chord or the brace side.
Table C5.1-1 includes SCF's developed from the referenced examples, to be used with equation (C5.1-1) for simple joints.

DFF

Table 5.2.5-1 Fatigue Life Safety Factors

Failure critical	Inspectable	Not Inspectable
No	2	5
Yes	5	10

- Table above is for assessment of Category L-1 structures;
- A reduced safety factor is recommended for Category L-2 and L-3 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 cases being half those in the table;

Design fatigue factor in DNV-RP-C203 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 satisifes the requirement to damaged condition according to the ALS with failure in the actual element as the defined damage.

Table A1 Design fatigue factors (DFF)

DFF	*Structural element*
1	Internal structure, accessible and not welded directly to the submerged part.
1	External structure, accessible for regular inspection and repair in dry and clean conditions.
2	Internal structure, accessible and welded directly to the submerged part.
2	External structure not accessible for inspection and repair in dry and clean conditions.
3	Non-accessible areas, areas not planned to be accessible for inspection and repair during operation.

NORSOK N-004, Tables 8-1 and K.4-1 also give the DFFs and are included in Detailed Fatigue comparison table.

SCF

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

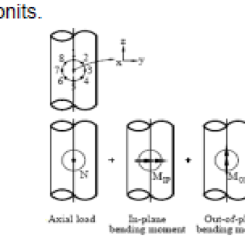
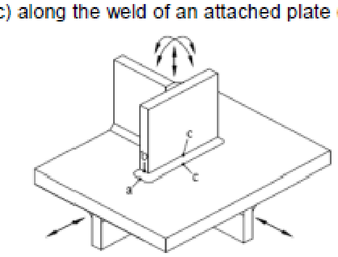
	Joint Type	α	Axial Load	In-Plane Bending	Out-of-Plane Bending
Chord SCF	K	1.0	αA	2/3A	3/2A
	T & Y	1.7			
	X $\beta < 0.98$	2.4			
	X $\beta \geq 0.98$	1.7			
Brace SCF's			$1.0+0.375\,[1+(\tau/\beta)^{0.5}\,SCF_{chord}] \geq 1.8$		

Where $A = 1.8\,\gamma^{0.5}\,\tau \sin(\theta)$, $\beta = d/D$, $\gamma = D/(2T)$, $\tau = t/T$
d = Brace outside diameter, in. (mm)
D = Chord outside diameter, in. (mm)
t = Brace wall thickness at intersection, in. (mm)
T = Chord wall thickness at intersection, in. (mm)

SCF Formulas for tubular joints are given in DNV-RP-C203 Appendix B "SCF's for Tubular Joints"; SCF's for Penetrations with Reinforcements are given in Appendix C;

Table 4-3 Detailed Fatigue

Table - Detailed Fatigue

	API RP 2A - WSD	ISO 19902/19904	Norsok N-004/DNV RP-C203
Validity	The fatigue assessment of welded joints is based on the assumption that the connection has full-penetration single or double sided welding.	- 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 exisitng structures, the proviso 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, unless otherwise stated.	- In this standard, the requirements in relation to fatigue analyses are based on fatigue tests and fracture mechanics. -Reference is made to DNV-RP-C203 for more details with respect to fatigue design. - DNV RP-C203 1) It 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. 2) This RP is valid for material temperatures of up to 100°C. For higher temperatures the fatigue resistance data may be modified with a reduction factor.
Loading	The 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 blockage are not required; - For the Gulf of Mexico a steepness between 1:20 and 1:25 is generally used. A minimum height equal one foot and a maximum height equal to the design wave height should be used. - Wave kinematics factor = 1.0 - Conductor shieding factor = 1.0 - For small waves (1.0 < K < 6.0 for platform legs at mean water level), values of C_m = 2.0, C_d = 0.8 for rough members and C_d = 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.	- 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. - Fatigue analysis can normally be counducted with no current. - Wave kinematics factor = 1.0 - Conductor shielding factor = 1.0 - For small waves with KC referred to the mean water level in the range 1.0<KC<6, the hydrodynamic coefficients can be taken to be: C_d = 0.65 and C_m = 2.0 (smooth member); C_d = 0.80 and C_m = 2.0 (rough members) - In lack of site specific data, the wave periods shall be determined based on a wave steepness of 1/20. - For a stochastic fatigue analysis, it is important to select periods such that response amplifications and cancellations are included. Also selection of wav periods in relation to the platform fundamental period of vibration is important. The number of periods inlcuded in the analysis should not be less than 30, and be in the range from T = 2 s to at least T = 20 s.
Stress Range	**hot spot stress and hot spot stress range (HSSR)** - A minimal 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: $HSS_{sa} = SCF_{axsa} f_{ax}$ +/- $SCF_{opb} f_{opb}$ $HSS_{cr} = SCF_{axcr} f_{ax}$ +/- $SCF_{ipb} f_{ipb}$ +CE where f = nominal stress sa = saddle cr = crown ax = axial ipb = in-plane bending opb = out-of-plane bending CE = the effect of nominal cyclic stress in the chord **- Other than tubular joints** Where variations of stress are applied to conventional weld details, idetified 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.	**geometric stress (GS) and geometric stress range (GSC)** **-Tubular Joints** - A minimum of eight stress range locations need to be considered around each chord/brace intersection weld in order to adequately cover all relevant locations. These are: the chord sides at two crown positions, the brace sides at two crown positions, the chord sides at two saddle posotions and the brace sides at two saddle positions. - The GSRs for the chord and the brace side of the weld are determined: $\sigma_{GS,s}(t) = C_{ax,s}\sigma_{ax}(t)$ +/- $C_{opb,s}\,\sigma_{opb}(t)$ $\sigma_{GS,c}(t) = C_{ax,c}\sigma_{ax}(t)$ +/- $C_{ipb,c}\,\sigma_{ipb}(t) + \sigma_{C,c}(t)$ where σ_{Gs} = the geometric stress on the chord or the brace side of the weld between chord and brace σ_{ax} = the nominal axial stress in the brace (or stub) σ_{ipb} = the nominal in-plane bending stress in the brace (or stub) σ_{opb} = the nominal out-of-plane bending stress in the brace (or stub) $\sigma_{C,c}$ = the nominal stress in the chord (or chord can) at the crown position C_{ax} = the stress concentration factor for axial brace stress C_{ipb} = the stress concentration factor for in-plane bending stresses in the brace C_{opb} = the stress concentration factor for out-of-plane bending stresses in the brace t = time s = the subscript denoting the saddle position c = the subscript denoting the crown position - The effect of nominal variable stresses in the chord member can be covered by including the stress due to axial force in the chord can member, combined with an axial SCF of 1.25, i.e. $\sigma_{C,c}(t) = 1.25\,\sigma_{ax,C}(t)$ **- Other than tubular joints** - The stree range indicated in Tables A.16.10-7 to A.16.10-11 and used as the GSR is the maximum principle stress range adjacent to the detail under consideration, except for the throat of load carrying fillet or partial penetration welds, for which it is the shear stress range calculated on the minimum throat area. - For details that are not expressly classified, the following minimum classification class should be used, unless a higher class can be justified from published experimental work, or by specific tests: - W_1 for load carrying fillet or partial penetration weld metal; - F_2 for other cases	**hot spot stress and hot spot stress range** **- Tubular Joints and Members** The stresses are calculated at the crown and the saddle ponits. $\sigma_1 = SCF_{AC}\sigma_x + SCF_{MIP}\sigma_{my}$ $\sigma_2 = \frac{1}{2}(SCF_{AC}+SCF_{AS})\sigma_x + \frac{1}{2}\sqrt{2}\,SCF_{MIP}\sigma_{my} - \frac{1}{2}\sqrt{2}\,SCF_{MOP}\sigma_{mz}$ $\sigma_3 = SCF_{AS}\sigma_x - SCF_{MOP}\sigma_{mz}$ $\sigma_4 = \frac{1}{2}(SCF_{AC}+SCF_{AS})\sigma_x - \frac{1}{2}\sqrt{2}\,SCF_{MIP}\sigma_{my} - \frac{1}{2}\sqrt{2}\,SCF_{MOP}\sigma_{mz}$ (3.3.1) $\sigma_5 = SCF_{AC}\sigma_x - SCF_{MIP}\sigma_{my}$ $\sigma_6 = \frac{1}{2}(SCF_{AC}+SCF_{AS})\sigma_x - \frac{1}{2}\sqrt{2}\,SCF_{MIP}\sigma_{my} + \frac{1}{2}\sqrt{2}\,SCF_{MOP}\sigma_{mz}$ $\sigma_7 = SCF_{AS}\sigma_x + SCF_{MOP}\sigma_{mz}$ $\sigma_8 = \frac{1}{2}(SCF_{AC}+SCF_{AS})\sigma_x + \frac{1}{2}\sqrt{2}\,SCF_{MIP}\sigma_{my} + \frac{1}{2}\sqrt{2}\,SCF_{MOP}\sigma_{mz}$  Figure 3-7 Superposition of stresses **- Welded connections other than tubular joints** In plates structures, three types of hot spots at weld toes can be identified: a) at the weld toe on the plates surface at ending attachment b) at the weld toe around the plate edege of an ending attachment c) along the weld of an attached plate (weld toes on both the plate and attachment surface)  Figure 4-3 Different hot spot positions

Stress Concentration Factor (SCF)

SCF = HSSR at location (excluding notch effect) / Norminal Brace Stress Range;

1. The SCF should include all stress raising effects associated with the joint geometry and type of 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 guage adjacent to the toe 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 strain gauges are centred within 6 mm to $0.1(rt)^{0.5}$ from the weld toes with a guage length of 3 mm. r and t refer to the outside radius and thickness of the member instrumented, whether chord or brace.
3. The Efthymiou equations (in Tables C5.3.2-1 to C5.3.2-4) are recommended because this set of equations is considered to offer the best option for all joint types and load types and is the only widely vetted set that covers overlapped K and KT joints.

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

$\beta = d/D$ from 0.2 to 1.0
$\tau = t/T$ from 0.2 to 1.0
$\gamma = g/2T$ from 8 to 32
$\alpha = L/2D$ (length) from 4 to 40
θ from 20 to 90 degresss
$\xi = g/D$ (gap) from $-0.6\beta/\sin(\theta)$ to 1.0

4. For all welded tubular joints under all three types of loading, a minimum SCF of 1.5 should be used.
5. SCFs for internally ring-stiffened joints can be determined by applying the Lloyds reduction factors to the SCFs for the equivalent unstiffened joint. For ring-stiffened joints analyzed by such means, 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.

SCF = the range of the GS at a particular location of the intersection weld (excluding notch effect) / the range of the nominal brace stress

1. The recommeded S-N curves and SCF equations used is ISO are based on European definition and are consistent.
2. The **Efthymiou equations** are recommended because this set of equations is considered to offer either the best option or a very good option for all joint types and types of brace forces and is the only set which covers overlapped K- and KT-joints.
3. The validity ranges for the Efthymiou parametric SCF equations are as follows:

$\beta = d/D$ from 0.2 to 1.0
$\tau = t/T$ from 0.2 to 1.0
$\gamma = g/2T$ from 8 to 32
$\alpha = L/2D$ (length) from 4 to 40
θ from 20 to 90 degresss
$\xi = g/D$ (gap) from $-0.6b/\sin(\theta)$ to 1.0

4. Increasing the chord wall thickness is an effective way of reducing stress concentrations. For T/Y- and X-joints, a doubling of the chord wall thickness reduces the saddle SCFs by a factor of 4; crown SCFs are also reduced considerably.
5. SCF Equations for tubulars are given in ISO 19902 Table A.16.10-2 "Equations for SCFs in T/Y joints" and Table A.16.10-4 "Equations for SCFs in gap/overlap K-joints".

SCF = hot spot stress range/nominal stress range

- The local weld notch effect is excluded by using stress values just outside the weld notch region and extrapolating these (linearly) to the weld toe. The European definition is based on maximum principal stress, i.e. the stress components are extrapolated to the weld toes and then used in Mohr's Circle to establish the maximum principal stress at the toe.The stress normal to the weld toe, used in the US definition, is somewhat lower than this, but for the all-important saddle location the two are virtually identical.
- SCF Formulas for tubular joints are given in DNV-RP-C203 Appendix B "SCF's for Tubular Joints", Table B1 - B5; SCF's for Penetrations with Reinforcements are given in Appendix C;
- The validity range for the equations in Table B-1 to Table B-5 is as follows:

$\beta = d/D$ from 0.2 to 1.0
$\tau = t/T$ from 0.2 to 1.0
$\gamma = g/2T$ from 8 to 32
$\alpha = L/2D$ (length) from 4 to 40
θ from 20 to 90 degresss
$\xi = g/D$ (gap) from $-0.6b/\sin(q)$ to 1.0

...r Tubular Connections

The basic **tubular joint** S-N curve has been derived from an analysis of data on tubular joints manufaured using welds conforming to a standard flat profile given in AWS.

The basic design S-N curve is of the form:

$\mathrm{Log}_{10}(N) = \mathrm{Log}_{10}(k1) - m\mathrm{Log}_{10}(S)$ (5.4.1-1)

where N = the predicted number of cycles to failure under stress range S,
k1 = a constant,
m = the inverse slope of the S-N curve

Table 5.5.1-1 - Basic Design S-N Curves

Curve	$\log_{10}(k1)$ S in ksi	$\log_{10}(k1)$ S in MPa	m
Welded Joints (WJ)	9.95 11.92	12.48 16.13	3 for $N < 10^7$ 5 for $N > 10^7$
Cast Joints (CJ)	11.80 13.00	15.17 17.21	4 for $N < 10^7$ 5 for $N > 10^7$

- The basic design S-N curves given in Table 5.5.1-1 are applicable for joints in air and submerged coated joints.
- These S-N curves are based on steels with yield strength less than 72 ksi (500 MPa)
- The WJ curve is based on 5/8-in (16mm) reference thickness. For material thickness above the reference thickness, the following thickness effect should be applied for as-welded joints:

$S = S_o (t_{ref}/t)^{0.25}$

where: tref = the reference thickness, 5/8-inch (16 mm),
S = allowable stress range
So = the allowable stress range from the S-N curve
t = member thickness for which the fatigue life is predicted

- The material thickness effect for castings is given by:

$S = So (t_{ref}/t)^{0.15}$

where the reference thickness t_{ref} is 1.5 in (38 mm)

- For Welded Joints in seawater with adequate cathodic protection, the m = 3 branch of the S-N curve should be reduced by a factor of 2.0 on life, with the m = 5 branch remaining unchanged and the position of the slope change adjusted accordingly.

The basic design S-N curve is of the form:

$\log_{10}(N) = \log_{10}(k1) - m\log_{10}(S)$ (16.11-1)

where N = the predicted number of cycles to failure under constant amplitude stress range S,
k1 = a constant, (k1 = N for S=1)
m = the inverse slope of the S-N curve
S = the constant amplitude stress range, which is the geometrical stress range

Table 16.11-1 - Basic representative S-N curves for air and sea water

Curve	Air: $\log_{10}(k1)$ S in MPa	Air: m	Sea water with adequate corrosion protection: $\log_{10}(k1)$ S in MPa	Sea water with adequate corrosion protection: m
Welded Joints (WJ)	12.48 16.13	3.0 for $N\le10^7$ 5.0 for $N>10^7$	12.18 16.13	3.0 for $N\le 1.8 \times 10^6$ 5.0 for $N>1.8 \times 0^6$
Cast Joints (CJ)	15.17	4		
Other joints (OJ)				
B	15.01 17.01	4.0 for $N\le10^7$ 5.0 for $N>10^7$	14.61 17.01	4.0 for $N\le10^5$ 5.0 for $N>10^5$
C	13.63 16.47	3.5 for $N\le10^7$ 5.0 for $N>10^7$	13.23 16.47	3.5 for $N\le4.68 \times 10^5$ 5.0 for $N>4.68 \times 10^5$
D	12.18 15.63	3.0 for $N\le10^7$ 5.0 for $N>10^7$	11.78 15.63	3.0 for $N\le10^6$ 5.0 for $N>10^6$
E	12.02 15.37	3.0 for $N\le10^7$ 5.0 for $N>10^7$	11.62 15.37	3.0 for $N\le10^6$ 5.0 for $N>10^6$
F	11.80 15.00	3.0 for $N\le10^7$ 5.0 for $N>10^7$	11.40 15.00	3.0 for $N\le10^6$ 5.0 for $N>10^6$
F_2	11.63 14.71	3.0 for $N\le10^7$ 5.0 for $N>10^7$	11.23 14.71	3.0 for $N\le10^6$ 5.0 for $N>10^6$

The basic design S-N curve is of the form:

$\log_{10}(N) = \log(\bar{a}) - m\log_{10}(\Delta\sigma)$ (2.4.1 DNV RP-C203)

where N = the predicted number of cycles to failure under constant amplitude stress range $\Delta\sigma$,
m = negative inverse slope of the S-N curve
$\Delta\sigma$ = stress range
$\log \bar{a} = \log a - 2s$ intercept of log N-axis by S-N curve
a = constant relating to mean S-N curve
s = standard deviation of log N

Table 2-1 S-N Curve in Air and Table 2-2 S-N curve in seawater with cathodic protection

S-N Curve	Air: $\log \bar{a}_1$ S in MPa	Air: m	Sea water with adequate cathodic protection: $\log_{10}(k1)$ S in MPa	Sea water with adequate cathodic protection: m
T	12.164 15.606	3.0 for $N\le10^7$ 5.0 for $N>10^7$	11.764 15.606	3.0 for $N\le 10^6$ 5.0 for $N>10^6$
C	12.592 16.320	3.0 for $N\le10^7$ 5.0 for $N>10^7$	12.192 16.320	3.0 for $N\le 10^6$ 5.0 for $N>10^6$
Other joints (OJ)				
B1	15.117 17.146	4.0 for $N\le10^7$ 5.0 for $N>10^7$	14.971 17.146	4.0 for $N\le 10^6$ 5.0 for $N>10^6$
B2	14.885 16.858	4.0 for $N\le10^7$ 5.0 for $N>10^7$	14.685 16.856	4.0 for $N\le 10^6$ 5.0 for $N>10^6$
C1	12.449 16.081	3.0 for $N\le10^7$ 5.0 for $N>10^7$	12.049 16.081	3.0 for $N\le10^6$ 5.0 for $N>10^6$
C2	12.301 15.835	3.0 for $N\le10^7$ 5.0 for $N>10^7$	11.901 15.835	3.0 for $N\le10^6$ 5.0 for $N>10^6$
D	12.164 15.606	3.0 for $N\le10^7$ 5.0 for $N>10^7$	11.764 15.606	3.0 for $N\le10^6$ 5.0 for $N>10^6$
E	12.010 15.350	3.0 for $N\le10^7$ 5.0 for $N>10^7$	11.610 15.350	3.0 for $N\le10^6$ 5.0 for $N>10^6$

S-N Curves fo

- The curve for cast joints is only applicable to castings having an adequate fabrication inspection plan.

G	11.40 14.33	3.0 for $N \leq 10^7$ 5.0 for $N > 10^7$	11.00 14.33	3.0 for $N \leq 10^6$ 5.0 for $N > 10^6$
W_1	10.97 13.62	3.0 for $N \leq 10^7$ 5.0 for $N > 10^7$	10.57 13.62	3.0 for $N \leq 10^6$ 5.0 for $N > 10^6$

- The basic design S-N curves given in Table 16.11-1 are applicable for joints in air and seawater with adequate corrosion protection.
- These S-N curves are based on steels with yield strength less than 500 MPa.
- The WJ curve is based on 16mm) material thickness. For material thickness above 16 mm, the following thickness effect should be applied for as-welded joints:

$S = S_0\,(16/t)^{0.25}$

where: S = the stress range of S-N curve, when adjusted for thickness effects
So = the stress range from the S-N curve in Table 16.11-1
t = member thickness in mm for which the fatigue life is predicted

- The material thickness effect for castings is given by:

$S = So\,(38/t)^{0.15}$

- The curve for cast joints is only applicable to castings having an adequate fabrication inspection plan.

F	11.855 15.091	3.0 for $N \leq 10^7$ 5.0 for $N > 10^7$	11.455 15.091	3.0 for $N \leq 10^6$ 5.0 for $N > 10^6$
F1	11.699 14.832	3.0 for $N \leq 10^7$ 5.0 for $N > 10^7$	11.299 14.832	3.0 for $N \leq 10^6$ 5.0 for $N > 10^6$
F3	11.546 14.576	3.0 for $N \leq 10^7$ 5.0 for $N > 10^7$	11.146 14.576	3.0 for $N \leq 10^6$ 5.0 for $N > 10^6$
G	11.398 14.330	3.0 for $N \leq 10^7$ 5.0 for $N > 10^7$	10.998 14.330	3.0 for $N \leq 10^6$ 5.0 for $N > 10^6$
W1	11.261 14.101	3.0 for $N \leq 10^7$ 5.0 for $N > 10^7$	10.861 14.101	3.0 for $N \leq 10^6$ 5.0 for $N > 10^6$
W2	11.107 13.845	3.0 for $N \leq 10^7$ 5.0 for $N > 10^7$	10.707 13.845	3.0 for $N \leq 10^6$ 5.0 for $N > 10^6$
W3	10.970 13.617	3.0 for $N \leq 10^7$ 5.0 for $N > 10^7$	10.570 13.617	3.0 for $N \leq 10^6$ 5.0 for $N > 10^6$

- S-N curves are obtained from fatigue tests. The design S-N curves which follows are based on the mean-minus-two-standard-deviation curves for relevant experimental data. The S-N curves are associated with 97.6% probability of survival.
- The thickness effect is accounted for by a modification on stress such that the design S-N curve for thickness larger than the reference thickness reads:

$\log_{10}(N) = \log(\bar{a}) - m\log_{10}(\Delta\sigma(t/t_{ref})^k)$ (2.4.3 DNV RP-C203)

where t_{ref} = reference thickness equal 25 mm for welded connections other than tubular joints. For tubular joints the reference thickness is 32 mm. For bolts t_{ref} = 25 mm
k = thickness exponent on fatigue strength as given in Tables 2-1, 2-2 and 2-3
k = 0.10 for tubular butt welds made from one side
k = 0.25 for threaded bolts subjected to stress variation in the axial direction

- It is recommended to use the C curve for cast nodes. t_{ref} = 38 mm may be used.
- For forged nodes the B1 curve may be used for nodes designed with a Design Fatigue Factor equal to 10. For designs with DFF less than 10 it is recommended to use the C-curve to allow for wled repair if fatigue cracks should occur during service life.

Fatigue damage design factors

Table 5.2.5-1 Fatigue Life Safety Factors

Failure critical	Inspectable	Not Inspectable
No	2	5
Yes	5	10

- Table above is for assessment of Category L-1 structures;
- A reduced safety factor is recommended for Category L-2 and L-3 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 cases being half those in the table;

ISO 19902 - Table A.16.12.-1 - Fatigue damage design factors, γ_{FD}

Failure critical component	Inspectable	Not Inspectable
No	2	5
Yes	5	10

- The factors given in Table A.16.12-1 should be considered to relate to exposure Level L1, but should also be used for exposure levels L2 and L3. There is currently insufficient background to establish different factors for lower exposure levels.

ISO 19904 Table 6- Fatigue damage design safety factors

Consequence of fauilure	Degree of accessibility for inspection and repair		
	Not accessible	Underwater access	Dry access
Substantial	10	5	2
Non-substantial	5	2	1

Table 8-1 Design Fatigue Factors (DFF's)

Classification of structural components based on damage consequence	Access for inspection and repair	
	No access or in the splash zone	Accessibility Below splash zone
Substantial consequences	10	3
Without substantial consequences	3	2

"Subtantial consequences" in this context means that failure of the joint will entail danger of loss of human life; significant pollution; major financial consequences
"Without substantial consequences" is understood failure where it can be demonstrated that the structure satisfy the requirement to damaged condition according to the ALSs with failure in the actual joint as the defined damage

Table K.4-1 Fatigue design factors in jackets

Classification of structural components based on damage consequence	Access for inspection and repair	
	No access or in the splash zone	Accessibility Below splash zone
- Brace/stub to chord welds in main loadtransferring joints in vertical plans - chord/cone to leg welds, between leg connections - Brace to stub and Brace to Brace welds in main loadtransferring members in vertical plans - Shear plates and yoke plates incl. stiffening - Piles and bucket foundation plates incl. stiffening	10	3
- Brace/stub to chord welds in joints in horizontal plans - chord/cone to leg welds, between leg connections - Chord/cone to brace welds and welds between sections in horizontal plans - Appurtenance supports - Anodes, doubler plates - Outfitting steel	3	2

The cumulative fatigue damage ratio, D,

$$D = \sum (n/N) \qquad (5.2.4\text{-}1)$$

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)

Fatigue damage accumulation	where n = number of cycles applied at a given stress range N = number of cycles for which the given stress range would be allowed by appropriate S-N curve When faigue damage can occur due to other cyclic loadings, such as tansportation, the following equations should be satisfied: $$\sum_j SF_j D_j < 1.0 \qquad (5.2.5\text{-}1)$$ where Dj = the fatigue damage ratio for each type of loading SFj = the associated safety factor	$$D = k_{LE} \cdot \gamma_{FD} \cdot \sum_i \frac{n_i}{N_i} \qquad (16.12\text{-}1)$$ where D is a non-dimensional number, the Palmgren-Miner sum or damage ratio for a time T; k_{LE} is a local experience factor, see 16.12.3; γ_{FD} is a fatigue damage design factor, see 16.12.2; n_i is the number of cycles of stress range, S_i, occurring during time period, T; N_i is the number of cycles to failure under constant amplitude stress range, S_i, taken from the relevant S–N curve.	$$D = \sum_{i=1}^{k} \frac{n_i}{N_i} = \frac{1}{\bar{a}} \sum_{i=1}^{k} n_i \cdot (\Delta\sigma_i)^m \le \eta \qquad (2.2.1)$$ where D = accumulated fatigue damage $\bar{a}$ = intercept of the design S-N curve with the log N axis m = negative inverse slope of the S-N curve k = number of stress blocks n_i = number of stress cycles in stress block i N_i = number of cycles to failure at constant stress range $\Delta\sigma_i$ η = usage factor = 1 / Design Fatigue Factor from OS-C101 Section 6 Fatigue Limit States.
Spectral Analysis Technique	**1. Transfer functions developed using regular waves in the time domain** - Charaterize 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 jacket response transfer functions - Select a wave height corresponding to each frequency; 1) For GoM, a steepness between 1:20 and 1:25 is generally used. 2) A minimum height of one foot and a maximum height equal to the design wave height should be used. 3) Compute a stress range transfer function at each point where fatigue damage is to be accumulated for a minimum of four platform directions (end-on, broadside and two diagonals). More directions may be required for jackets with unusual geometry or where wave directionality or spreading or current is considered 4) A minimum of four hot spot locations at both the brace and chord side of the connection should be considered. 5) Compute the stress response spectra. **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. **3. Transfer functions developed using random waves in the time domain**: Nonliearities arising from wave-structure interaction can be taken into account and difficulties in selecting wave heights and frequencies 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 stress spectra - Generate "exact" transfer functions from wave and response stress spectra - Calculate pseudo transfer functions for all the remaining sea states in the scatter diagram using the few " exact" transfer functions - Calculate pesudo response stress spectra as described in Section C5.2.2-1	- A pratical method that is best able to represent the random nature of the wave environment - only applicable to linear system as it is based on superimposition of many individual frequency components; this formal constraint can be overcome by suitable linearization of non-linear elements. - **Stress transfer functions** 1) to be determined by performing global stress analyses directly in the frequency domain; If this method is chosen, the global analyses shall be performed using linear wave theory and the drag term in Morison's equation shall be linearized. The calculated stresses are linearly dependent on the wave height and non-linear wave height influences are not included. 2) to be determined by performing global stress analyses in the time domain by stepping a full wave cycle past the structure. Various wave theories can be used and linear drag term can be allowed. - **Selection of wave frequencies** Select a sufficient number of frequencies to define all the peaks and valleys - **Selection of wave heights** Typical wave steepness values are in the range of 1:15 to 1:20. A wave height equal to the wave height with a one year return period should normally be used as a maximum. Typically, a broadside, an end-on and a diagonal wave direction are considered as a minimum. - **Short-term stress range statistics** - **Long-term stress range statistics**	Refer to DNV Classification Notes 30.7 - Fatigue Assessment of Ship Structures
chanics	The details refer to ISO 19902 Clause A.16.15	- Typical applications: 1) to assess the fitness-for-purpose of a component with or without known 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 - The principal modes of failure in offshore structures: 1) crack growth driven by fatigue followed by the onset of fracture due to exceedance of the fracture toughness at a critical crack size (not necessarily through-thickness) 2) the occurrence of plastic collapse - Fatigue crack growth law	- This method is recommeded for use in assessment of acceptable defects, evaluation of acceptance criteria for fabrication and for planning in-service inspection. - This can be achieved by performing the analysis according to the following procedures: 1) crack growth parameter C determined as mean plus 2 standard deviation 2) a careful evaluation of initial defects that might be present in the structure when taking into account the actual NDE inspection method used to detect cracks during fabrication 3) use of geometry functions that are on the safe side 4) use of utilization factors similar to those used when the fatigue analysis is based on S-N data The Paris' equation may be used to predict the crack propagation or the fatigue life:

Fracture Mec

whe $da/dN = C\,(\Delta K)^m$
N = the number of cycles to failure
ΔK = the stress intensity factor range
C and m = parameters of the crack growth rate

whe $\Delta K = Y(\Delta\sigma)\sqrt{\pi a}$ stress intensity factor
$\Delta\sigma$ = the stress range

$$\frac{da}{dN} = C(\Delta K)^m$$

where $\Delta K = K_{max} - K_{min}$

N = Number of cycles to failure

a = crack depth. It is assumed that the crack depth/length ratio is low (less than 1:5)

C, m = materall parameters, see BS 7910

$$K = \sigma g \sqrt{\pi a}$$

s = nominal stress in the member normal to the crack

g = factor depending on the geometry of the member and the crack

- See BS 7910 for more detailed guidelines related to fatigue assessment

Weld Improvement Techniques

- **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 grinding to a depth not less than 0.5mm 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.
NDE of the joint is required after grinding to verify that no significant defects remain, for fillet-welded connections, it is important that the required throat size is maintained.

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

For welded tubular nodes, full grinding of the surface profile to a radius of not less than 0.5t qualified for both the life improvement factor of 2 on curve WJ, and the 0.15 size effect exponent applicable to geometrically similar notch-free scale-ups

- **Hammer Peening -** The objective is to obtain a smooth groove at the weld toe.

The groove depth should be at least 0.3mm, but should 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.
Peened weld toes should be inspected directly after peening and any burr grinding with MPI

- **Post-Weld Heat Treatment**

Table 5.5.3-1—Factors on Fatigue Life for Weld Improvement Techniques

Weld Improvement Technique	Improvement Factor on S	Improvement Factor on N
Profile per 11.1.3d	$^a\tau^{-0.1}$	varies
Weld toe burr grind	1.25	2
Hammer peening	1.56	4
^{a} Chord side only.		

- **Post-weld heat treatment (PWHT)** - have a beneficial effect on the fatigue behaviour 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 weld 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 0.3mm, but should 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 16.16-1 — Achievable improvement factors on fatigue performance for weld improvement techniques

Weld improvement technique	Improvement factor
Weld toe burr grinding	2
Hammer peening	4

- **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 toes below any visible undercuts is performed the fatigue life may be increased by a factor given in Table 7-1.
The thickness effect may be reduced to an exponent k =0.20

- **TIG dressing**
- **Hammer peening with the following limitations:**

1) only be used on members where failure will be without substantial consequences
2) overload in compression must be avoided
3) It is recommended to grind a steering 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 weld toe.

Table 7-1 Improvement on fatigue life by different methods

Improvement method	*Minimum specified yield strength*	*Increase in fatigue life (factor on life)* [1)]
Grinding	Less than 350 MPa	$0.01 f_y$
	Higher than 350 MPa	3.5
TIG dressing	Less than 350 MPa	$0.01 f_y$
	Higher than 350 MPa	3.5
Hammer peening [3)]	Less than 350 MPa	$0.011 f_y$
	Higher than 350 MPa	4.0

1) The maximum S-N class that can be claimed by weld improvement is C1 or C depending on NDE and quality assurance for execution see Table A-5 in Appendix A.
2) f_y = characteristic yield strength for the actual material.
3) The improvement effect is dependent on tool used and workmanship. Therefore, if the fabricator is without experience with respect to hammer peening, it is recommended to perform fatigue testing of relevant detail (with and without hammer peening) before a factor on improvement is decided.

5 FOUNDATION DESIGN

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

- API-RP 2A WSD, Recommended Practice for Planning, Designing and Constructing Fixed Offshore Platforms—Working Stress Design, October 2007
- 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 5-1 Load combinations in API, ISO and NORSOK

Code – design condition	Load combinations	Pile material coefficient
ISO – extreme condition	$1.1D_1+1.1D_2+1.1L_1+\gamma_{f,Ee}(E_e+\gamma_{f,D}D_e)$ or $0.9D_1+0.9D_2+0.8L_1+\gamma_{f,Ee}(E_e+\gamma_{f,D}D_e)$	1.25
ISO – operating condition	$1.3D_1+1.3D_2+1.5L_1+1.5L_2+0.9\gamma_{f,Ee}(E_e+\gamma_{f,D}D_e)$	1.5
API LRFD – extreme condition	$1.1D_1+1.1D_2+1.1L_1+1.35(E_e+1.25D_e)$ or $0.9D_1+0.9D_2+0.8L_1+1.35(E_e+1.25D_e)$	1.25
API LRFD – operating condition	$1.3D_1+1.3D_2+1.5L_1+1.5L_2+1.2(E_e+1.25D_e)$	1.5
NORSOK – extreme ULS	$1.0D+1.0L_1+1.3E$ or $1.3D+1.3L_1+0.7E$	1.3 (applied to pile group)
NORSOK – 10 000 y ALS	$1.0D+1.0L_1+1.0E$	1.0

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:

$$F_{c,ax} = P + V + E \leq Q_{c,ax} / SF \qquad (5.1)$$

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:

$$F_{d,ax} = \gamma_{f,P} \cdot \mathrm{P} + \gamma_{\mathrm{f,V}} \cdot \mathrm{V} + \gamma_{\mathrm{f,Es}} \cdot (\mathrm{E_s} + \gamma_{\mathrm{f,Ed}} \cdot \mathrm{E}_d) \leq Q_{d,ax} = Q_{c,ax} / \gamma_m \qquad (5.2)$$

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:

$$F_{d,ax} = \gamma_{f,P} \cdot \mathrm{P} + \gamma_{\mathrm{f,V}} \cdot \mathrm{V} + \gamma_{\mathrm{f,E}} \cdot \mathrm{E} \leq Q_{d,ax} = Q_{c,ax} / \gamma_m \qquad (5.3)$$

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

Table 5-2 Load factors in ISO and NORSOK

	γ_m	$\gamma_{f,P}$	$\gamma_{f,V}$	$\gamma_{f,E}$ or $\gamma_{f,Es}$	$\gamma_{f,Es}$
NORSOK comb.a	1,30	1,30	1,30	0,70	n.a.
NORSOK comb.b	1,30	1,00	1,00	1,30	n.a.
ISO, extreme-c	1,25	1,10	1,10	1,35	1,25
ISO, extreme-t	1,25	0,90	0,80	1,35	1,25
ISO, operation	1,50	1,30	1,50	1,22	1,25

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 $\gamma_{f,Es}$ = 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: $$SF_{eqv} = \gamma_m \cdot \frac{\gamma_{f,E} \cdot E + \gamma_{f,P+V} \cdot (P+V)}{E+(P+V)} \quad (5.4)$$

ISO: $$SF_{eqv} = \gamma_m \cdot \frac{\gamma_{f,E} \cdot E / DAF \cdot (1+\gamma_{f,D} \cdot (DAF-1)) + \gamma_{f,P+Q} \cdot (P+V)}{E+(P+V)} \quad (5.5)$$

where $$DAF = \frac{E_s + E_d}{E_s} = \frac{E}{E_s}$$

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 -∞ or +∞. 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.

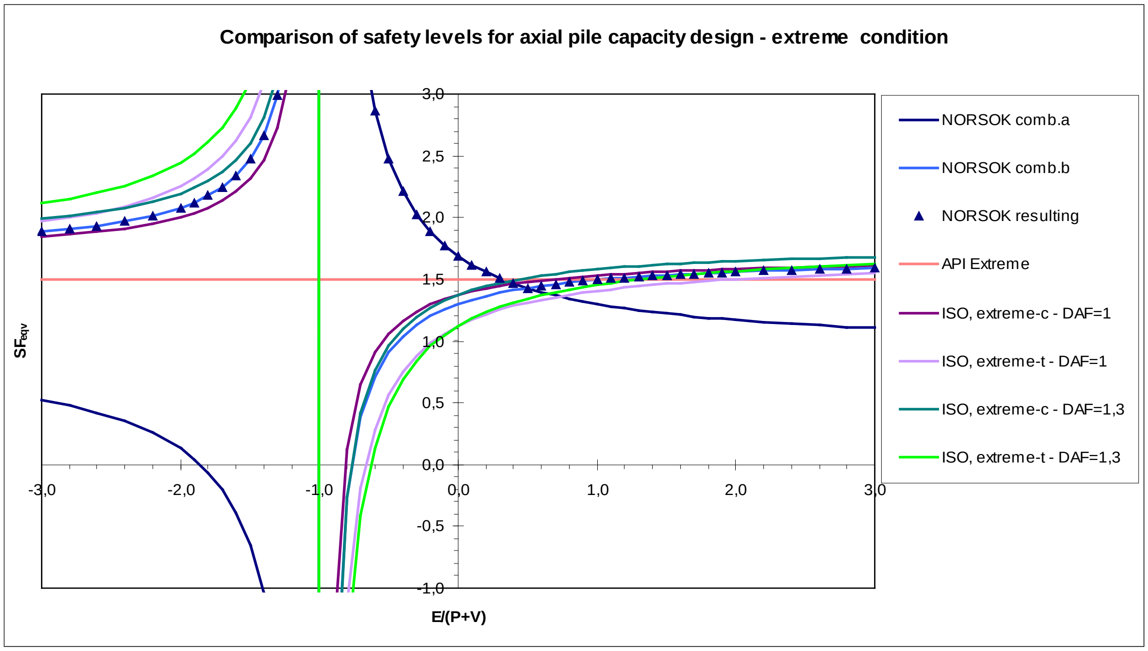


Figure 5-1 Comparison of safety levels for axial pile capacity design – extreme condition

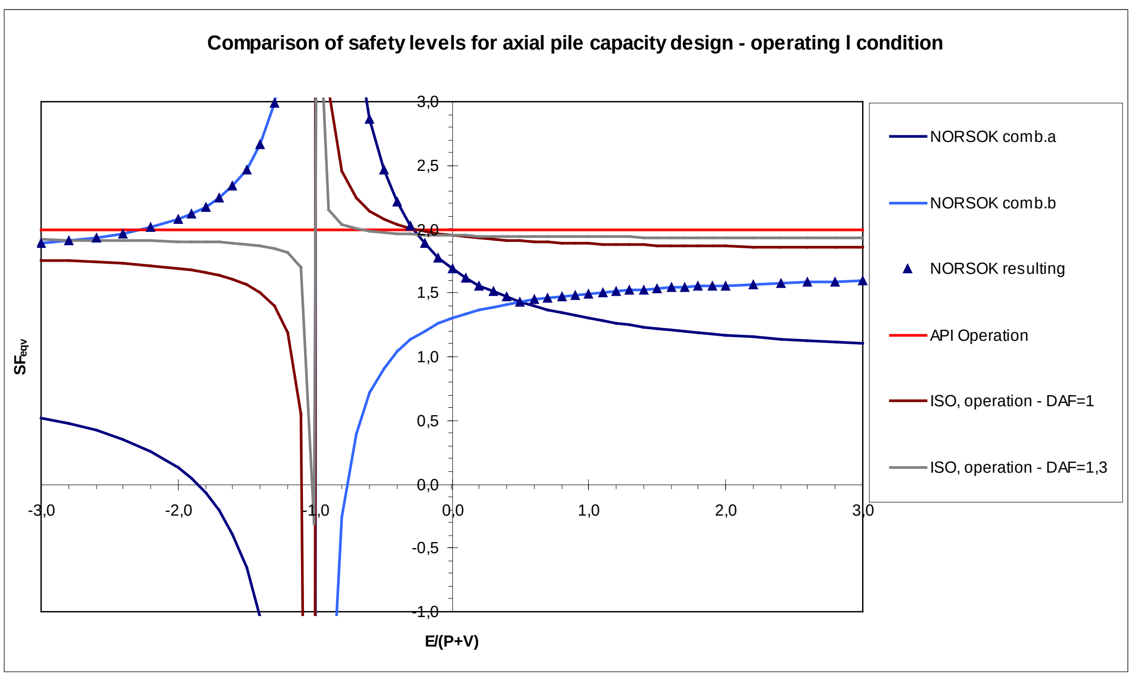


Figure 5-2 Comparison of safety levels for axial pile capacity design – operating condition

5.2 Axial Pile Capacity – Methods of Calculation

The following main comparison points are made:

- API and ISO prescribes 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

API WSD	API LRFD	ISO 19902	NORSOK N-004
3.3.1.b Cylindrical Piles $f_a/(0.6F_{xc}) + (f_{bz}^2 + f_{by}^2)^{1/2}/F_b < 1.0$ $F_{xc} = F_y[1.64-0.23(D/t)^{1/4}] \le F_{xe}$ $60 < D/t < 300$; $t >= 6$ mm $F_{xc} = F_y$ for $(D/t) \le 60$ **3.3.1.c Pile Overload Analysis** $P/A + 2/\pi\{\text{arc sin}[(M/Z)/F_{xc}]\} \le 1.0$ where the arc sin term is in radians and A = cross-sectional area, in^2. (m^2) Z = plastic section modulus, in^3 (m^3) P, M = axial loading and bending moment computed from a nonlinear analysis, including the (P-Δ) effect F_{xc} = Critical local buckling stress with a limiting value of $1.2F_y$ considering the effect of strain hardening	**D.3.2.2 Piles** $1-\cos[(\pi/2)f_c/(\phi_c F_{xc})] + [f_{by}^2+f_{bz}^2]^{0.5}/(\phi_b F_{bn}) \le 1$ where $\phi_c = 0.85$ $\phi_b = 0.95$ fby, fbz should include secondary moments or P-Δ effects	**17.3.4 (a) Pile strength** The pile strength shall be verified using the steel tubular strength checking equations given in 13.3 or 13.4 for conditions of combined axial force and bending.	
6.3.4 Pile Penetration The allowable pile capacities are determined by dividing the ultimate pile capacity by approriate factors of safety which should not be less than the following values: Load condition — Safety Factor 1 Design environmental conditions with appropriate drilling loads — 1.5 2 Operating environmental conditions during drilling operations — 2 3 Design environmental conditions with appropriate producing loads — 1.5 4 Operating environmental conditions during producing operations — 2 5 Design environmental conditions with minimum loads (for pullout) — 1.5	**G3.4.2 Foundation Capacity** The axial pile capacity should satisfy the following conditions: $P_{DE} \le \phi_{PE} Q_D$ $P_{DO} \le \phi_{PO} Q_D$ where Q_D = ultimate axial pile capacity determined from a coupled linear structure and nonlinear foundation model using factored loads ϕ_{PE} = pile resistance factor for extreme environmental conditions (=0.8) ϕ_{PO} = pile resistance factor for operating environmental conditions (= 0.7)	**17.3.4 (b) Pile Axial Resistance** The axial pile capacity should satisfy the following conditions: $P_{d,e} \le Q_d = Q_r/\gamma_{R,Pe}$ $P_{d,p} \le Q_d = Q_r/\gamma_{R,Pp}$ where Q_d = design axial pile capacity Q_r = the representative value of the axial pile capacity $P_{d,e}$ = design axial action on the pile,determined from a coupled linear structure and non-linear foundation model using the design actions for extreme conditions $P_{d,p}$ = design axial action on the pile,determined from a coupled linear structure and non-linear foundation model using the design actions for permanent or variable actions or the design axial action for operating situations $\gamma_{R,Pe}$ = the pile partial resistance factor for extreme conditions (= 1.25) $\gamma_{R,Pp}$ = the pile partial resistance factor for permanent and variable actions or operating situations (= 1.50)	**K.6.2.1 Axial Pile Resistance** For determination of design soil resistance against axial pile loads in ULS design, a material factor γ_M=1.3 is to be applied to all characteristic values of soil resistance. **K.6.2.2 Lateral Pile Resistance** When lateral soil resistance governs pile penetrations, the design resistance is to be checked with the limit state categories ULS and ALS, using following material factors appiled to characteristic resistance: γ_M = 1.3 for ULS condition γ_M = 1.0 for ALS condition
6.10 Pile Wall Thickness For piles that are to be installed by driving where sustained hard driving (250 blows per foot[820 blows per meter] with the largest size hammer to be used) is anticipated, the minimum piling wall thickness used should not be less than: t = 0.25 + D/100 or t = 6.35 + D/100 (Metric Formula) where t = wall thinkness, in. (mm) D = diameter, in. (mm)	**G.10.6 Minimum Wall Thickness** For piles that are to be installed by driving where sustained hard driving (250 blows per foot[820 blows per meter] with the largest size hammer to be used) is anticipated, the minimum piling wall thickness used should not be less than: t = 0.25 + D/100 or t = 6.35 + D/100 (Metric Formula) where t = wall thinkness, in. (mm) D = diameter, in. (mm)		

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.

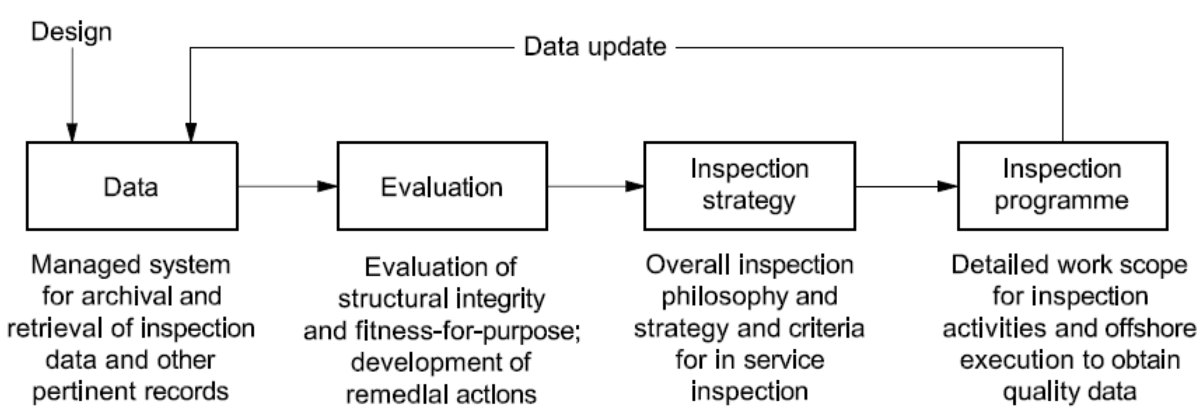


Figure 6-1 Phases of a structural integrity management cycle (ISO 19902)

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

Methods/techniques		Capability	Technical description	ROV applicability	Cost related aspects
Visual	Without cleaning	Scour, sea floor instability, gross damage, signs of gross damages, existence of anodes	Video based reporting.	ROV	Fast
	With cleaning	Follow-up investigation of general damage	Video based reporting	Work class ROV	Time consuming
Electro nic	MPE	Fatigue damage	Existence and length of surface crack.	Work class ROV	Cleaning required. Time consuming
	EC	Fatigue damage	Length of surface crack, may also be used for depth measurement. Independent evaluation possible	ROV	Relatively fast. Through coating up to ca. 10mm.
	ACPD	Fatigue damage	Depth of surface crack. Supporting MPE.	Work class ROV	Cleaning required. Time consuming
	ACFM	Fatigue damage	Both length and depth of surface crack. Independent evaluation possible.	ROV	Relatively fast. Through coating up to ca. 10mm.
Ultrasonic		Wall thickness, corrosion, fatigue and fabrication defect, post-repair inspection	Embedded defects.	Work class ROV, Usually performed by diver	Cost level depends on different applications.
FMD		Fatigue damage and post-event damage	Through thickness crack. Excellent tool for rapid screening.	ROV	Fast No cleaning necessary.
Cathodic Potential		Corrosion	Anode performance. Often combined with visual inspection.	ROV	Field calibration necessary, but the readings can be taken quickly
Other	Dimensional measurement	Scour, subsidence, marine growth, dent, out-of straightness, corrosion pit size, etc.	For special purposes.	May be performed by ROV, but with limited capability	
	Stress&def. monitoring	Structural behaviour monitoring	For special purposes.		
	Radiography	E.g. testing of hyperbaric welds	Internal defects		

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

API RP 2A WSD/LRFD	ISO 19902	NORSOK N-005
14. SURVEY		
14.3 SURVEY LEVELS	**23 IN-SERVICE INSPECTION AND SIM**	**5 PROGRAMME FOR CONDITION MONITORING**
LEVEL I 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, detect deteriorating coating systems, excessive corrosion, and bent, missing and damaged members. General examination of all structural members in splash zone and above water, concentrating on condition of the more critical areas such as deck legs, girders, trusses, 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 can't fully determine the extent of damage. Should Level I survey indicate that underwater damage could have occured, a Level II inspection **LEVEL II** General underwater visual inspection by divers or ROV to detect presence of: excessive corrosion, accidental or environmental overloading, scour and seafloor instability, fatigue damage, design or constructruction deficiencies, presence of debris, and excessive marine growth. The survey should include measurement of cathodic potentials of pre-selected critical areas. Detection 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.	**LEVEL I** 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, detect deteriorating coating systems, excessive corrosion, and bent, missing and damaged members. General examination of all structural members in splash zone and above water, concentrating on condition of the more critical areas such as deck legs, girders, trusses, 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 can't fully determine the extent of damage. Should Level I survey indicate that underwater damage could have occured, a Level II inspection **LEVEL II** General underwater visual inspection by divers or ROV to detect presence of: excessive corrosion, accidental or environmental overloading, scour and seafloor instability, fatigue damage, design or constructruction deficiencies, presence of debris, and excessive marine growth. The survey should include measurement of cathodic potentials of pre-selected critical areas. Detection 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.	The detailed condition monitoring programme of loadbearing structures depends on the design and mainenance philosophy, the current condition, the capability of the inspection methods availible, and the intended use of the structure. The focus should be put on the identified safety critical components, in addition to improving the accuracy and reliability of prediction of structural performance and in-service inspection methods.
LEVEL III An underwater visual inspection of preselected areas and/or, based on results of Level II survey, areas known or suspected damage. Such areas should be sufficiently cleaned of marine growth to permit thorough inspection. FMD can provide an acceptable alternative to close visual inspection (CVI). CVI for corrosion monitoring should be included as part of Level III survey. Detection of significant structural damage during Level III survey should become the basis for initiation of Level IV survey, where CVI alone can't not determine the extent of damage. Level IV survey, if required, should be conducted as soon as conditions permit.	**LEVEL III** An underwater visual inspection of preselected areas and/or, based on results of Level II survey, areas known or suspected damage. Such areas should be sufficiently cleaned of marine growth to permit thorough inspection. FMD can provide an acceptable alternative to close visual inspection (CVI). CVI for corrosion monitoring should be included as part of Level III survey. Detection of significant structural damage during Level III survey should become the basis for initiation of Level IV survey, where CVI alone can't not determine the extent of damage. Level IV survey, if required, should be conducted as soon as conditions permit.	
LEVEL IV 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 occured. Monitoring fatigue-sensitive and/or reported crack-like indications, can be an acceptable alternative to analyctical verification	**LEVEL IV** 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 occured. Monitoring fatigue-sensitive and/or reported crack-like indications, can be an acceptable alternative to analyctical verification	

14.4 SURVEY FREQUENCY

Frequency of surveys are dependent upon the exposure categories of the platform for both life safety and consequence of faliure considerations.

Survey Intervals

Exposure Category Level	Survey Level I	II	III	IV
L-1	1 yr	3-5 yrs	6-10 yrs	*
L-2	1 yr	5-10 yrs	11-15 yrs	*
L-3	1 yr	5-10 yrs	*	*

* survey should be performed if required, based on lower level inspection findings

Time interval between surveys for fixed platforms should not exceed the intervals shown in table above, unless experience and/or engineering analyses indicate that different intervals are justified. Justification for changing guideline survey intervals should be documented and retained by operator. Following factors, which either increase or decrease survey intervals, should be taken into account:

1. Original design/assessment criteria
2. Present structural condition
3. Service history of platform
4. Platform structural redundancy
5. Criticalness of platform to other operations
6. Platform location
7. Damage
8. Fatigue sensitivity

Special Surveys

Level I survey should be conducted after direct exposure to a design environmental event.
Level II survey should be conducted after severe accidental loading that could lead to structural degradation (i.e. boat collisonn, dropped objects), or after an event exceeding the platform;s original design/assessment criteria.
Areas critical to the structural integrity of the platform, which have undergone structural repair, should be subjected to a Level II survey approximately one year following completion of the repair. A Level III survey should be performed when exessive marine growth prevents visual inspection of the repaired areas.
Level II scour surveys in scour-prone areas should take account of local experience, and are usually more frequent than intervals indicated in table above.

14.5 PRESELECTED SURVEY AREAS

During initial platform design and any subsequent reanalysis, critical members and joints should be identified to assist in defining requirements for future platform surveys. Selection of critical areas should be based on such factors as joint and member loads, stresses, stress concentrations, structural redundancy, and fatigue lives determined during platform design/assessment.

14.6 RECORDS

Records of all surveys should be retained by the operator for the life of 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.
Description of detected damage should be thoroughly documented and included with survey results.
Any resulting repairs and engineering evaluations of the platform's condition should be documented and retained.

23.7 DEFAULT PERIODIC INSPECTION REQUIREMENTS

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

Survey Intervals

Exposure Level	Inspection Level I	II	III	IV
L-1	Annual	3 yrs	5 yrs	*
L-2	Annual	5 yrs	10 yrs	*
L-3	Annual	5 yrs	not required	not required

* determined from Level III inspection results

Inspection requirements less than default can be justified when an inspection strategy is developed and maintained.

Special inspections

Special inspections shall be undertaken: to assess performance of repairs undertaken to ensure the fitness-for-purpose of structure, conducted approximately 1 yr after completion of the repair, and to monitor known defects, damage, local corrosion, scour, or other conditions which could potentially affect the fitness-for-purpose of the structure

Unscheduled inspections

An inspection shall be conducted as soon as practical after the occurrence of an environmental event exceeding that for which structure was designed or assessed, or of a significant accidental action. The minimum scope shall include the following: a visual inspection without marine growth cleaning that provides full coverage from sea floor to top of structure, conductors, risers, and various appurtenaces, and which includes checking the seabed conditions at legs/piles and looking for debris and damage.

23.2 DATA COLLECTION AND UPDATE

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

4.3 CONDITION MONITORING PRINCIPLES

Operator shall monitor the condition of operated offshore installation in systematic manner, This may include development of an overall philosophy and strategy for condition monitorning, 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 in the nature of uncertainity 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 instalation

Periodic condition monitoring - "shall be carried out regularly"

6.4 INSPECTION RECORD

Operator shall maintain an up to date filing system for results and evaluations from condition monitoring programme throughout the lifetime of installation. The data may include video tape, inspection log, first hand inspection report, evaluation and recommendations.
Such data records should also include tools/techniques employed, planned and actual scope 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

	Design Criteria	Assessment Criteria
Environment	Forecast from existing data collection	As criteria for "new" platform, with inclusion of recent data collection and use of : - current state of art review - experience from adjacent fields - additional data from actual field sea-states
Loading	Possibly conservative evaluation from proposed use of structure	Conservative evaluation from as-built records and use of recent survey info on: - marine growth - appurtenances - removals/additions/modifications - topsides weight control - wind areas
Foundation	Forecast from site investigation and laboratory testing of soils	As criteria for "new" platform with inclusion of: - subsidence information - current state-of-the-art review - experience form adjacent fields - post-drive foundation analyses - scour survey and maintenance
Modeling	Topology and dimensions may be changed. No service inspection available. Conservative modeling using global percentages to cover not-finalized details and simple geometric assumptions	The structure dimensions are fixed and known. In-service inspection may be applied. Actual characteristic strength of steel based on actual material certificates may be used. Structural performance may have been measured and used to update structural analysis.
Stress Analysis	The time for analysis is critical. Strict compliance with code of practice and regulatory documents.	The quality of the analysis is critical. Sufficient time for model tests, removing of conservatism where possible, redundancy studies to determine ultimate strength of structure and foundation, sensitivity studies on various parameters to improve confidence levels
Results Evaluation	Structure has members and joints with acceptable utilization.	Structure has some stresses up to yield stress, but some assessment standards allow for some yielding if the structure has proven strength and redundancy.

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

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

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 Section4 (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.

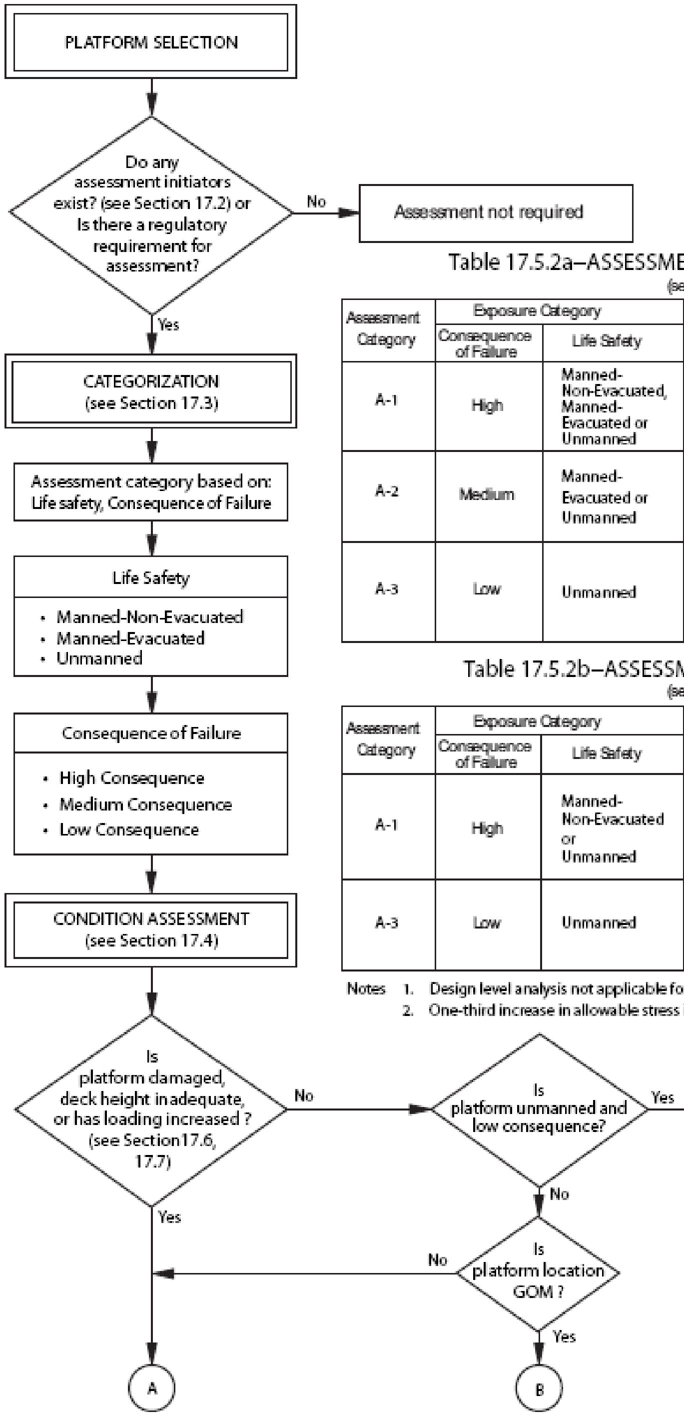


Table 17.5.2a—ASSESSMENT CRITERIA—U.S. GULF OF MEXICO
(see Table 17.6.2-1)

Assessment Category	Exposure Category: Consequence of Failure	Exposure Category: Life Safety	Design Level Analysis (see Notes 1 and 2)	Ultimate Strength Analysis
A-1	High	Manned-Non-Evacuated, Manned-Evacuated or Unmanned	High Consequence design level analysis loading (see Figure 17.6.2-2a)	High Consequence ultimate strength analysis loading (see Figure 17.6.2-2a)
A-2	Medium	Manned-Evacuated or Unmanned	Sudden hurricane design level analysis loading (see Figure 17.6.2-3a)	Sudden hurricane ultimate strength analysis loading (see Figure 17.6.2-3a)
A-3	Low	Unmanned	Minimum consequence design level analysis loading (see Figure 17.6.2-5a)	Minimum consequence ultimate strength analysis loading (see Figure 17.6.2-5a)

Table 17.5.2b—ASSESSMENT CRITERIA—OTHER U.S. AREAS
(see Table 17.6.2-2)

Assessment Category	Exposure Category: Consequence of Failure	Exposure Category: Life Safety	Design Level Analysis (see Notes 1 and 2)	Ultimate Strength Analysis
A-1	High	Manned-Non-Evacuated or Unmanned	85% of lateral loading caused by 100-year environmental conditions (see Section 17.6.2b)	Reserve strength ratio (RSR) ^{3} 1.6 (see Section 17.6.2b)
A-3	Low	Unmanned	50% of lateral loading caused by 100-year environmental conditions (see Section 17.6.2b)	(RSR) ^{3} 0.8 (see Section 17.6.2b)

Notes 1. Design level analysis not applicable for platforms with inadequate deck height.
2. One-third increase in allowable stress is permitted for design level analysis (all categories).

Figure 17.5.2—Platform Assessment Process—Metocean Loading

Figure 7-1 Platform Assessment Process

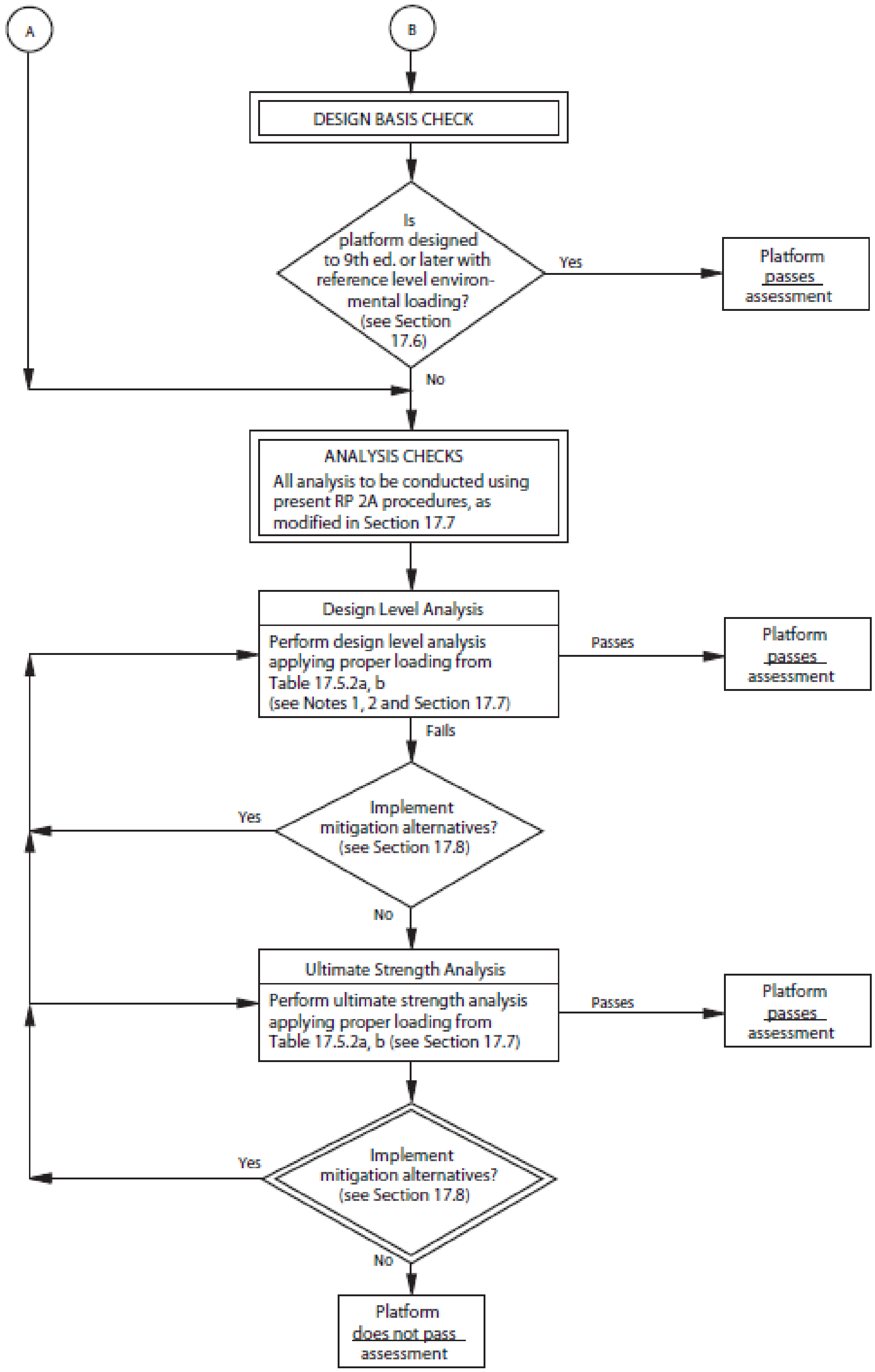


Figure 17.5.2—Platform Assessment Process—Metocean Loading (Continued)

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.

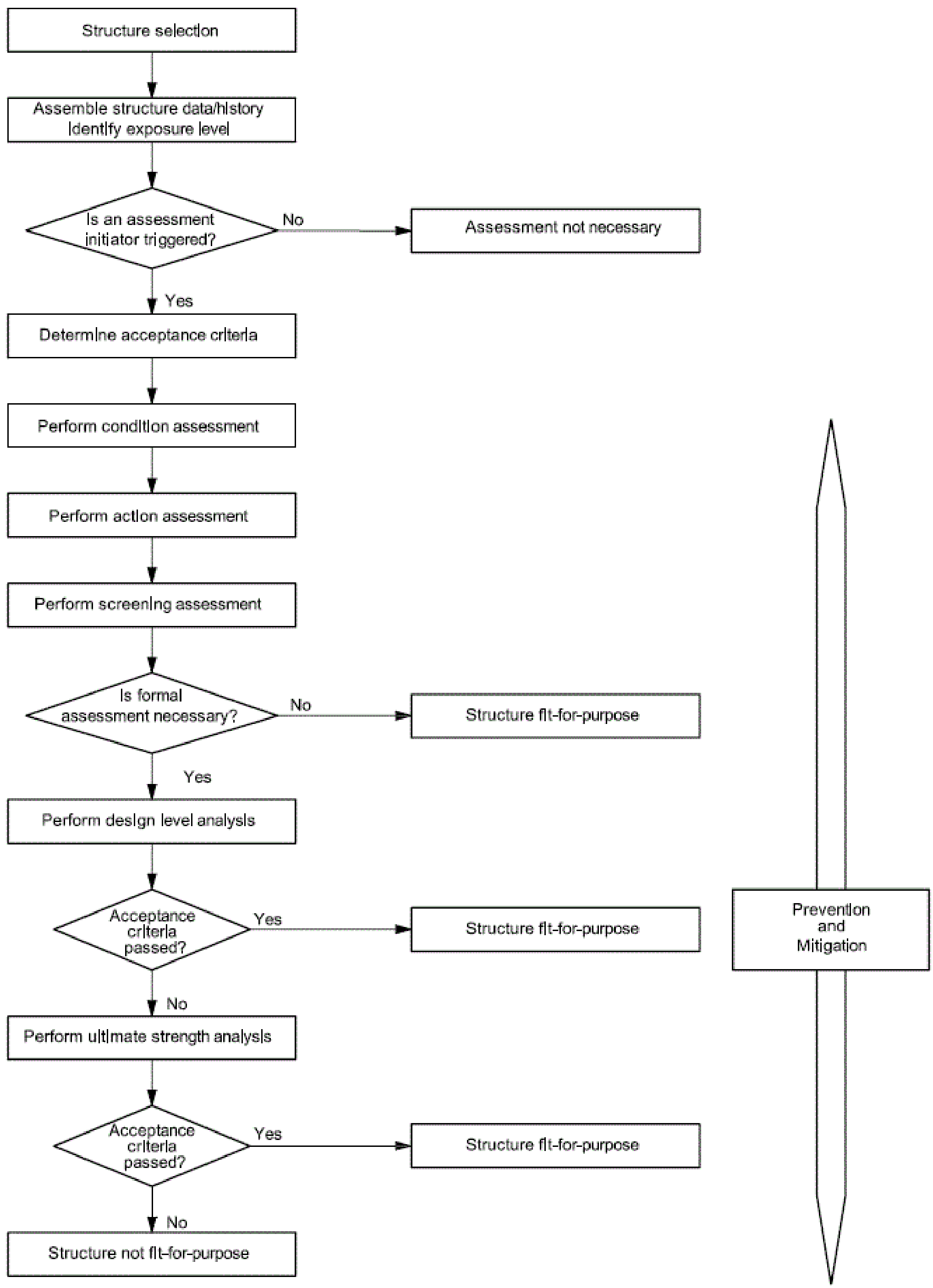


Figure 7-3 Flow chart of the assessment process

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

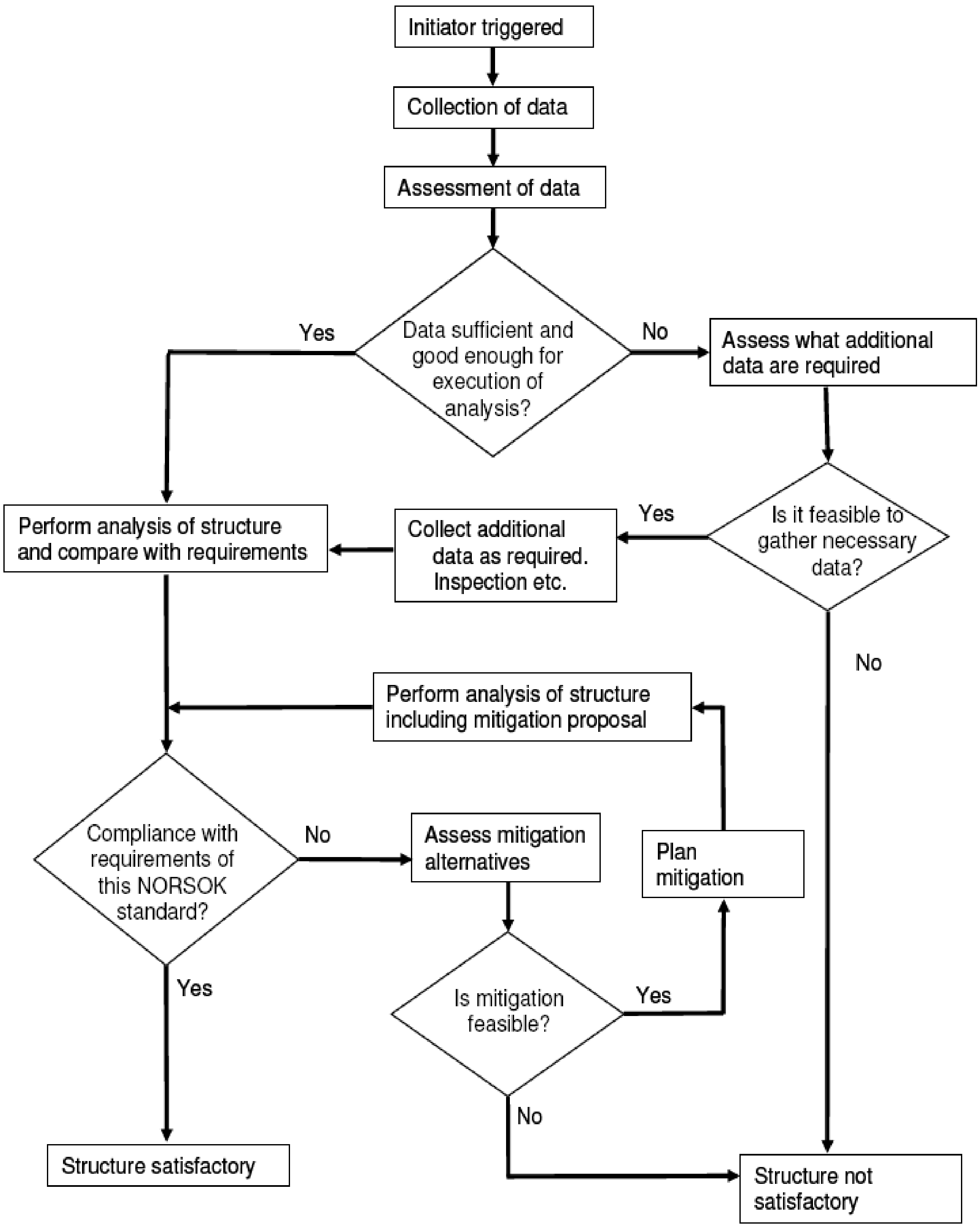


Figure 7-4 Flow Sheet of the Assessment Process

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 to 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>
<tr><th></th><th>API RP 2A- WSD Section 17 (To be replaced by API RP 2SIM)</th><th>ISO 19902</th><th>NORSOK N-006</th></tr>
<tr><td>Limitation</td><td>Only for the assessment of the following platforms:
- designed in accordance with the provisions in the 20th and earlier editions
- the platforms designed prior to the first editions</td><td>- the assessment of existing fixed steel offshore structures to demonstrate their fitness-for-purpose
- also applicable to topsides structures
- fit-for-purpose when the risk of structural failure leading to unaccepatble consequences is sufficiently low.</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;
- applicable to different types of materials used including steel, concrete, aluminium;
- 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></tr>
<tr><td>Assessment Process</td><td>1. Platform selection (Section 17.2)
2. Categorization (Section 17.3)
3. Condition assessment (Section 17.4)
4. Design basis check (Sections 17.5 and 17.6)
5. Analysis check (Sections 17.6 and 17.7)
6. Consideration of mitigations (Section 17.8)</td><td>24.2
a) assemble data on the structure, its history and exposure level, see 24.3
b) determine if any assessment initiators are triggered, see 24.4
c) determine acceptance criteria, see 24.5
d) assess the condition of the structure, see 24.6
e) assess the actions, see 24.7
f) screen the structure in comparison with similiar structures, see 24.8
g) perform a resistance assessment, see 24.9 using
1) design level analysis
2) ultimate strength level analysis
3) prevention and mitigation, see 24.10</td><td>4.1
- design, fabrication and installation resume and as-built drawings
- doucmentation of as-is condition
- planned changes and modifications of the facility
- updated design basis and specifications
- calibration of analysis models to measurements of behavior if such measurements exist
- the history of degradations and incidents
- prediction of future degradations and incidents
- the effect of degradation on future performance of the structure
- a documentation of technical and operational integrity
- planned mitigations
- a plan or strategy for the maintenance and inspection</td></tr>
<tr><td>Platform Assessment Initiators</td><td>Section 17.2
Definition of Significant: The total of the cumulative changes in greater than 10%
- Additional of personnel: life safety level changed to a more restrictive level
- Addition of facilities: addition of facilities or the consequence of failure level changed significantly
- Increased loading on structure: the new combined environmental/operational loading significantly increased
- Inadequate deck height: platforms with inadequate deck height for its exposure category and not designed for the impact of wave loading on the deck
- Damage found during inspections: significant damage to primary structural components found during any inspection</td><td>24.4
a) Changes from the original design or previous assessment basis, including
1) addtion of personnel or facilities
2) modification to the facilities
3) more onerous enviromental conditions and/or criteria
4) more onerous component or foundation resistance data and/or criteria
5) physical changes to the structure's design basis, e.g. excessive scour or subsidence
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.
b) Damage or deterioration of a primary structural component
c) Exceedance of design service life
- the fatigue life (including safety factors) is less than the required extended service life
- degradation of the structure due to corrosion is present, or is likely to occur, within the required extended service life
An extension of the design service life can be accepted without a full assessment if inspection of the structure shows that timeb-dependent degradation (i.e. fatigue and corrosion) has not become significant and that there have been no changes to the design</td><td>4.2
a) changes from the original design or previous assessment basis, including
- modification to the facilities,
- more onerous environmental conditions and/or criteria,
- more onerous component or foundation resistance data
- physical changes to the structure's design basis
- inadequate deck height
b) damage or deterioration of a primary structural component or a mechanical component
c) exceedance of design service life, if either
- the remaining fatigue life (including design fatigue factors) is less than the required extended service life
- degradation of the structure beyond design allowances, or is likely to occur within the required extended service life</td></tr>
<tr><td>Platform Assessment Categories</td><td>Section 17.3
Assessment categories based on: Life safety, Consequence of failure
Life Safety
- Manned-Non-Evacuated
- Manned-Evacuated
- Unmanned
Consequence of failure
- 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 platofrms in water depths greater than 400 ft are considered A-1
- A-2 - Medium Assessment Category: exisiting platforms where production would be shut-in during the design event; exisitng platforms that do not meet the A-1 or A-3 definitions
- A-3 - Low Assessment Category: exisiting platforms where production would be shut-in during the design event;</td><td>24.3.2
Acceptance criteria for assessment depend on the exposure level of the platform.
Table 6.6-1 — Determination of exposure level<table><tr><th rowspan="2">Life-safety category</th><th colspan="3">Consequence category</th></tr><tr><th>C1 High consequence</th><th>C2 Medium consequence</th><th>C3 Low consequence</th></tr><tr><td>S1 Manned non-evacuated</td><td>L1</td><td>L1</td><td>L1</td></tr><tr><td>S2 Manned evacuated</td><td>L1</td><td>L2</td><td>L2</td></tr><tr><td>S3 Unmanned</td><td>L1</td><td>L2</td><td>L3</td></tr></table></td><td>NORSOK defines lower safety factors for unmanned platforms</td></tr>
<tr><td>Condition Assessment</td><td>Section 17.4
- Topsides - only require the annual Level I survey: topside arrangement and configuration, platform exposure category, structural framing details etc.
- Underwater - Level II survey
- Soil Data - Available on- or near-site soil borings and geophysical data should be reviewed.</td><td>24.6
- Topsides surveys
- Underwater and splash zone surveys: Level II inspection as a minimum
- Foundation data: available on-site or near-site soil borings shall be reviewed.</td><td>See 4.1</td></tr>
</table>

Section 17.5, 17.6 and 17.7

Methods for determining acceptance criteria

1) design level analysis
2) ultimate strength analysis
3) assessment of similar platforms by comparison
4) assessment through the use of explicit probabilities of failure
5) 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

- Methods for determining acceptance (24.5)

a) through the use of explicitly calculated probabilities of failure
b) through risk based structural reserve strength ratio factors (RSR) developed for location specific and generic structure exposure level
c) by comparison with similiar platforms, a structure shall not be considered as fit-for-purpose by comparison to a similiar structure which itself has been determined to be fit-for-purpose by comparison to another structure
d) based on prior exposure, e.g. survival of 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 system 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
- revised resistance criteria, e.g. reduced RSRs

Exsiting structures shall meet the requirements of NORSOK N-001.
Exisiting 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 givein in 6.3.
- requirements to unmanned facilities according to NORSOK N-001 are satisfied.
- the evironmental 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 acceptable. see 6.5

Assessment for Metocean Loading

Table 17.5.2b–ASSESSMENT CRITERIA–OTHER U.S. AREAS
(see Table 17.6.2-2)

Assessment Category	Exposure Category: Consequence of Failure	Exposure Category: Life Safety	Design Level Analysis (see Notes 1 and 2)	Ultimate Strength Analysis
A-1	High	Manned-Non-Evacuated or Unmanned	85% of lateral loading caused by 100-year environmental conditions (see Section 17.6.2b)	Reserve strength ratio (RSR)^3 1.6 (see Section 17.6.2b)
A-3	Low	Unmanned	50% of lateral loading caused by 100-year environmental conditions (see Section 17.6.2b)	(RSR)^3 0.8 (see Section 17.6.2b)

Notes 1. Design level analysis not applicable for platforms with inadequate deck height.
2. One-third increase in allowable stress is permitted for design level analysis (all categories).

Table 17.5.2a–ASSESSMENT CRITERIA–U.S. GULF OF MEXICO
(see Table 17.6.2-1)

Assessment Category	Exposure Category: Consequence of Failure	Exposure Category: Life Safety	Design Level Analysis (see Notes 1 and 2)	Ultimate Strength Analysis
A-1	High	Manned-Non-Evacuated, Manned-Evacuated or Unmanned	High Consequence design level analysis loading (see Figure 17.6.2-2a)	High Consequence ultimate strength analysis loading (see Figure 17.6.2-2a)
A-2	Medium	Manned-Evacuated or Unmanned	Sudden hurricane design level analysis loading (see Figure 17.6.2-3a)	Sudden hurricane ultimate strength analysis loading (see Figure 17.6.2-3a)
A-3	Low	Unmanned	Minimum consequence design level analysis loading (see Figure 17.6.2-5a)	Minimum consequence ultimate strength analysis loading (see Figure 17.6.2-5a)

Notes: 1. RSR - defined as the ratio of a platform's ultimate lateral load carrying capacity to its 100-yr L-1 environmental condition lateral loading, computed using present API RP 2A criteria for new design as contained in Section 2.
2. The assessment process described herein is applicable for areas outside of the U.S., with the exception of the use of the reduced criteria which are applicable for indicated U.S. areas only.

Table 17.6.2-1—U.S. Gulf of Mexico Metocean Criteria

Criteria	A-1 Full Population Hurricanes: Design Level Analysis	A-1 Full Population Hurricanes: Ultimate Strength Analysis	A-2 Sudden Hurricanes: Design Level Analysis	A-2 Sudden Hurricanes: Ultimate Strength Analysis	A-3 Winter Storms: Design Level Analysis	A-3 Winter Storms: Ultimate Strength Analysis
Wave height and storm tide, ft	Fig. 17.6.2-2a	Fig. 17.6.2-2a	Fig. 17.6.2-3a	Fig. 17.6.2-3a	Fig. 17.6.2-5a	Fig. 17.6.2-5a
Deck height, ft	Fig. 17.6.2-2b	Fig. 17.6.2-2b	Fig. 17.6.2-3b	Fig. 17.6.2-3b	Fig. 17.6.2-5b	Fig. 17.6.2-5b
Wave and current direction	Omni-directional*	Fig. 2.3.4-4	Omni-directional**	Fig. 17.6.2-4	Omni-directional	Omni-directional
Current speed, knots	1.6	2.3	1.2	1.8	0.9	1.0
Wave period, seconds	12.1	13.5	11.3	12.5	10.5	11.5
Wind speed (1 hr @ 10 m), knots	65	85	55	70	45	50

Note: ft = feet; hr = hour; m = meters.
*If the wave height or current versus direction exceeds that required by Section 2, L-1 criteria for new designs, then the Section 2 criteria will govern.
**If the wave height or current versus direction exceeds that required for ultimate-strength analysis, then the ultimate-strength criteria will govern.

Notes: 1. Both hurricanes and winter storms are important to the assessment process. In calculating wave forces based on Section 2.3, a wave kinematics factor of 0.88 should be used for hurricanes, and 1.0 for winter storms.

Table C17.1-1—Comparison of Section 2 L-1 Wave Criteria and Section 17 Wave Criteria for 400 ft. Water Depth, Gulf of Mexico

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

* Wave heights and return periods for other water depths and in other regions will differ.

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

- Metocean parameters and Evrionmental Actions (24.7.2)

The Matocean data required for an assessment are the same as for a new structure design, as are environmental design situations and actions. In some cases, a reduced return period may be considered for assessment, see Clause 24.5.

- Deck Elevation and Additional Environmental Actions (24.7.3)

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

- Design Level Analysis

1. The assessment of strucrtural members shall comply with the requirements of Clause 13;
2. Assessment of structural connections shall comply with the requirements of Clauses 14 and 15, with the following exceptions:
  - there is no requirement for joint strength to be limited to its brace member strengths.
  - the strength of ungrouted and grouted joints may be based on experimental or analytical studies
3. Fitness-for-purpose
If all components within the structure and foundation are assessed to have utilizations less than or equal to unity, the structure may be considered to be fit-for-purpose, and no further analysis us required.

- Ultimate Strength Analysis

1. Local overstress and potential local damage are acceptable, but total collapse or excessive/damaging deformations shall be avoided.
2. Reserve Strength Ratio:

$R_{RS} = F_{collapse}/F_{100}$

where: $F_{collapse}$ is the unfactored global environmental action which, when co-existing unfactored permanent and variable are added, causes collapse of the structure

F_{100} is the unfactored 100 year global environmental action calculated in accordance with Clause 9.

3. If the minimum RSR value calculated from the the ultimate strength analysis meets or exceeds the acceptance criteria from 24.5.1, the structure may be considered to be fit-for-purpose, and no further analysis is required.
4. In the absence of specific acceptance criteria, fitness-for-purpose shall be assessed against the RSR value required for a new structure with the same exposure level and in the same location.

Table A.9.9-1 — Values of partial action factor $\gamma_{f,E}$ and RSR to achieve target failure rate $P_f < 3 \times 10^{-5}$/yr for new manned installations (exposure level L1)

Environment	Partial action factor, $\gamma_{f,E}$	Mean RSR
AUS	1,59	2,18
NS	1,40	1,92

ULS and ALS (section 8)

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

- Effects of degradation of the structure (e.g. corrosion, wear or damages from impacts) need to be properly monitored and accounted for in the assessments.

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

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

- 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:
1) no structural components will experience local or global buckling determined according to NORSOK N-004
2) tubular joints are not utilized above the capacity in NORSOK N-004 (first crack limit)
3) no plastic mechanism is formed
4) no part of the foundation has reached the ultimate soil capacity
5) joints are, by inspection, proven to be free from fatigue cracks or the calculated fatigue loading is negligible

- The cyclic check of the dimensioning storm should be made on low probability characteristic actions and 5% fractile resistance according to NORSOK N-001.

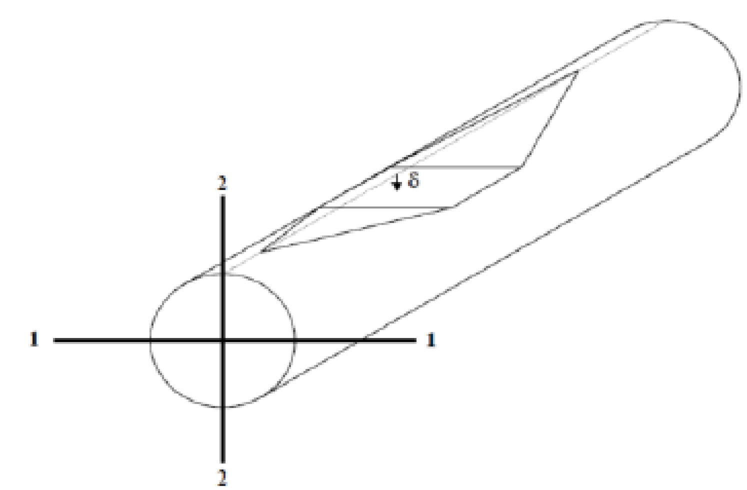
- No DFF should be applied when checking the cyclic storm actions.

FLS (section 7 and 9)

- Exsting facilities where structural details do not satisfy the criteria for FLS may continue to be used if requirements in Clause 7 and Clause 9 are fulfilled.
- Clause 7: Check of fatigue limit states (FLS)
The fatigue life is considered to be acceptable and within normal design criteria if the calculated fatigue life is longer than the total design service life times the DFF.
Otherwise a more detailed assessment including results from performed measurements of action effects and/or inspections throughout the prior service life is required.
- Clause 9: Requirements to in-service inspection after assessment
1) Inspection intervals shall be adjusted to take into account an increased likelihood of fatigue cracks as more fatigue damage is being accumulated.

	Table A.9.9-2 — Values of partial action factor, $\gamma_{f,E}$ and RSR to achieve target failure rate $P_f < 5 \times 10^{-4}$/yr for new unmanned installations (exposure level L2) Environment / Partial action factor, $\gamma_{f,E}$ / Mean RSR AUS / 1,17 / 1,60 GoM / 1,17 / 1,60 NS / 1,09 / 1,49 It is emphasized that the results in Table A.9.9-2 relate to new, unmanned (evacuated) structures. For exisitng 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 unexplained damage - a 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 unexplained 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 periodic	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 sustantial 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 criterion 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 8.4.
- 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 risk desicion and not applicable for seismic assessment purposes. 6. Ultimate strength analysis - is requiremed 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	- **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 strutcure may sustain considerable damage from ALE, but structural failures causing loss of life and /or major enviromental damage shall not be expected to occure.	
- 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 7-4 Damaged Members Formula Comparison

ISO 19902: 2007	NORSOK N-004
Stress/ Parameter — **Formulation** — **Limits**	**Stress/ Parameter** — **Formulation** — **Limits**
13.7 Dented tubular members	
13.7.2.2 AXIAL TENSION $\sigma_{t,d} = T/A$ $\sigma_{t,d} \le f_t/\gamma_{R,t,d}$ $\gamma_{R,t,d} = 1.05$, $f_t = f_y$ T = the member axial tensile force A = the cross-sectional area of the undamaged section $\gamma_{R,t,d}$ = the partial resistance factor for axial tensile strength for dented members	**10.6.2 AXIAL TENSION** $N_{Sd} <= N_{dent,t,Rd} = A_0 f_y/\gamma_M$ $\gamma_M = 1.15$ N_{Sd} = design axial force $N_{dent,t,Rd}$ = design axial tension capacity of the dented section A_0 = cross-sectional area of the undamaged section
13.7.2.3 AXIAL COMPRESION $\sigma_{c,d} = P/A$ $\sigma_{c,d} \le f_{c,d}/\gamma_{R,c,d}$ $\gamma_{R,c,d} = 1.18$ $\sigma_{c,d}$ = the axial compressive stress due to forces from factored actions on the undamaged cross-section P = the member axial compressive force $f_{c,d}$ = the representative axial strength of dented members, in stress unit $\gamma_{R,c,d}$ = the partial resistance factor for axial compressive strength for dented members *Column Buckling* $f_{c,d} = f_{c,d,o}$ for $\Delta y/L \le 0.001$ $f_{c,d}/f_{c,d,o} + [f_{c,d} A_d (\Delta y - 0.001L)]/[(1-f_{c,d}/(\xi_c f_{e,d})\xi_m f_b Z_e] = 1.0$ for $\Delta y/L > 0.001$ Δy = the maximum out-of-straightness of the dented member L = the unbraced member length, in place of buckling which coincides with the plane of Δy $f_{c,d,o}$ = the representative axial compressive strength of dented members when $\Delta y/L \le 0.001$ $f_{c,d,o} = (1-0.278\lambda_d^2)\ \xi_c\ f_{yc}$ $\lambda_d \le 1.34$ $0.9\ \xi_c\ f_{yc}/\lambda_d^2$ $\lambda_d > 1.34$ f_b = the representative bending strength as defined in 13.2.4, in stress units $f_{e,d}$ = the Euler buckling strength of the dented member, in stress units $fe,d = \pi^2 E/(K_d L/r_d)^2$ Ze = the elastic section modulus the undamaged members $\lambda_d = (f_{yc}/f_{e,d})^{1/2}$ the slenderness parameter of the dented member K_d = the effective length factor of the dented member, which may be assumed to be the same as that for the umdamaged member as defined in 13.2.3.2 $r_d = (I_d/A_d)$ the radius of gyration of the dented member $I_d = \xi_m\ I_o$ the effective moment of inertia of the dented cross-section Io = the moment of inertia of the undamaged member, as defined in 13.2.3.2 $A_d = \xi_c\ A$ the effective cross-sectional area of the dented section A = the cross-sectional area of the undamaged section, as defined in 13.2.3.2 $\xi_c = e^{-0.08h/t}$ for $h/t \le 10.0$ $\xi_m = e^{-0.06h/t}$ for $h/t \le 10.0$ h = the maximum depth of the dent t = the thickness of the member	**10.6.2.2 AXIAL COMPRESSION** $N_{Sd} \le N_{dent,c,Rd} = N_{dent,c}/\gamma_M$ $\gamma_M = 1.15$ $N_{dent,c} = (1.0-0.28\lambda_d^2)\xi_c f_y A_0$ for $\lambda_d \le 1.34$ $0.9\ \xi_c\ f_y\ A_0/\lambda_d^2$ for $\lambda_d > 1.34$ $N_{dent,c,Rd}$ = design axial compressive capacity of the dented section $N_{dent,c}$ = charateristic axial compressive capacity of dented member $\lambda_d = (\xi_c/\xi_M)^{0.5}\ \lambda_0$ reduced slenderness of the dented member λ_0 = reduced slenderness of undamaged member $\xi_c = e^{-0.08h/t}$ for $h/t \le 10.0$ $\xi_m = e^{-0.06h/t}$ for $h/t \le 10.0$ δ = dent depth t = wall thickness  **Figure 10-1 Definition of axes for dented section**

ISO 19902: 2007	NORSOK N-004
Stress/ Parameter — **Formulation** — **Limits**	**Stress/ Parameter** — **Formulation** — **Limits**
13.7.2.4.2 Positive Bending $\sigma^{+}_{b,d} = M^{+}/Z_e \le f_b/\gamma_{R,b,d}$ $\gamma_{R,b,d} = 1.05$ fb = the representative bending strength defined in 13.2.4, in stress units $\sigma^{+}_{b,d}$ = the positive bending stress due to forces from factored actions with respect to the undamaged cross-section $\gamma_{R,b,d}$ = the partial resistance factor for bending strength for dented members **13.7.2.4.3 Negative Bending** $\sigma^{-}_{b,d} = M^{-}/Z_e \le \xi_m f_b/\gamma_{R,b,d}$ $\sigma^{-}_{b,d}$ = the negative bending stress due to forces from factored actions with respect to the undamaged cross-section **13.7.2.4.4 Neutral Bending** $\sigma^{\cdot}_{b,d} = M^{\cdot}/Z_e \le f_b/\gamma_{R,b,d}$ $\sigma^{\cdot}_{b,d}$ = the neutral bending stress due to forces from factored actions with respect to the undamaged cross-section **13.7.2.5 Shear** $\tau_{b,d} = 2V/A$ for $h \le 0.25D$ $\tau_{b,d} = (2V/A)/(1.5-2h/D)$ for $0.25D < h \le 0.3D$	**10.6.2.3 Bending** $M_{Sd} \le M_{dent,Rd} = \xi_M M_{Rd}$ if the dented area acts in compression M_{Rd} otherwise
13.7.3 Dented tubular members subjected to combined forces **13.7.3.1.2 Axial Tension, Positive Bending and Neutral Bending** $\gamma_{R,t,d}\,\sigma_{t,d}/f_y + \gamma_{R,b,d}[(\sigma^{+}_{b,d})^2 + (\sigma^{\cdot}_{b,d})^2]^{0.5}/f_b \le 1.0$ **13.7.3.1.2 Axial Tension, Negative Bending and Neutral Bending** $\gamma_{R,t,d}\,\sigma_{t,d}/f_y + \gamma_{R,b,d}[(\sigma^{-}_{b,d})^{\alpha} + (\sigma^{\cdot}_{b,d})^2]^{0.5}/f_b \le 1.0$ $\alpha = 2 - 3h/D$ **13.7.3.2.2 Axial Compression, positive bending and neutral bending** $\gamma_{R,c,d}\,\sigma_{c,d}/f_{c,d} + \gamma_{R,b,d}\{[\sigma^{+}_{b,d}/(1-\sigma_{c,d})/f_e]^2 + [\sigma^{\cdot}_{b,d}(1-\sigma_{c,d}/f_e)^2]^{0.5}\}/f_b \le 1.0$ $\gamma_{R,c,d}\,\sigma_{c,d}/f_y + \gamma_{R,b,d}[(\sigma^{+}_{b,d})^2 + (\sigma^{\cdot}_{b,d})^2]^{0.5}/f_b \le 1.0$ $f_{yc,d}$ = the representative local buckling strength of the dented member, in stress units, $\xi_c f_{yc}$ f_{yc} = the representative local buckling strength of the undamaged member f_e = the smaller of the Euler buckling strengths of the undamaged member in the positive and neutral bending directions, in stress units **13.7.3.2.3 Axial Compression,negative bending and neutral bending** $$\frac{\gamma_{R,c,d}\,\sigma_{c,d}}{f_{c,d}} + \left\{\left[\frac{\gamma_{R,b,d}\,\sigma^{-}_{b,d}}{\left(1-\frac{\sigma_{c,d}}{\xi_c\,f_{e,d}}\right)\xi_m\,f_b}\right]^{\alpha} + \left[\frac{\gamma_{R,b,d}\,\sigma^{\cdot}_{b,d}}{\left(1-\frac{\sigma_{c,d}}{f_e}\right)f_b}\right]^2\right\}^{0,5} \le 1,0$$ $$\frac{\gamma_{R,c,d}\,\sigma_{c,d}}{f_{yc,d}} + \left[\left(\frac{\gamma_{R,b,d}\,\sigma^{-}_{b,d}}{\xi_m\,f_b}\right)^{\alpha} + \left(\frac{\gamma_{R,b,d}\,\sigma^{\cdot}_{b,d}}{f_b}\right)^2\right]^{0,5} \le 1,0$$	**10.6.2.4 Combined loading** $$\frac{N_{Sd}}{N_{dent,c,Rd}} + \sqrt{\left(\frac{N_{Sd}\,\Delta y_2 + C_{m1}M_{1,Sd}}{(1-\frac{N_{Sd}}{N_{E,dent}})\,M_{dent,Rd}}\right)^{\alpha} + \left(\frac{N_{Sd}\,\Delta y_1 + C_{m2}M_{2,Sd}}{(1-\frac{N_{Sd}}{N_E})\,M_{Rd}}\right)^2} \le 1$$, N in compression $$\frac{N_{Sd}}{N_{dent,t,Rd}} + \sqrt{\left(\frac{M_{1,Sd}}{M_{dent,Rd}}\right)^{\alpha} + \left(\frac{M_{2,Sd}}{M_{Rd}}\right)^2} \le 1$$, N in tension $\alpha = 2-3\delta/D$ if the dented area acts in compression N_{Sd} = design axial force on the dented section $M_{1,Sd}$ = design bending moment about an axis parallel to the dent $M_{2,Sd}$ = design bending moment about an axis perpendicular to the dent $N_{E,dent}$ = Euler buckling strength of the dented section, for buckling in-line with the dent, $\pi^2 EI_{dent}/(kl)^2$ k = effective length factor, as defined in Table 6-2 I_{dent} = moment of inertia of the dented cross-section, which may be calculated as = ξ_M I I = moment of inertia of undamaged section Δy_1 = member out-of-straightness perpendicular to the dent Δy_2 = member out-of-straightness in-line with the dent C_{m1},C_{m2} = moment reduction, as defined in Table 6-2

ISO 19902: 2007	NORSOK N-004
Stress/ Parameter — Formulation — Limits	Stress/ Parameter — Formulation — Limits
A13.8 Corroded tubular members 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.	**10.6.3 Corroded members** In lieu of refined analysis, the strength of uniformly corroded members cab be assessed by assuming a uniform thickness loass for the entire member. The reduced thickness should be cosistent 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 localised 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. $\delta/D = 0.5(1-\cos\pi A_{Corr}/A)$ δ = equivalent dent depth D = tube diameter A_{Corr} = corroded part of the cross-section A = full cross section area
14.8 Damaged Joints Joints in existing structures sometimes become damaged as a results 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 crakced members with the cracked area loaded in compression can be treated in a similar member to the one discussed for dented tubulars. $\delta/D = 0.5(1-\cos\pi A_{Crack}/A)$ where δ = equivalent dent depth D = tube diameter A_{crack} = crack area A = 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 failure 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 stastic strength of a cracked tubular joint can be calculated by reducing the joint resistances for a 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: $N_{crack,Rd} = F_{AR} N_{Rd}$ $M_{crack,Rd} = F_{AR} M_{Rd}$ where $N_{crack,Rd}$ = axial resistance of the cracked joint $M_{crack,Rd}$ = bending resistance of the cracked joint N_{Rd} = the joint design axial resistance M_{Rd} = the joint design bending moment resistance $F_{AR} = (1-A_c/A)(1/Q_\beta)^{mq}$ A_c = cracked area of the brace /chord intersection A = full area of the brace/chord intersection Q_β = tubular joint geometry factor, given in 6.4.3.3 mq = 0 for part-thickness cracks

ISO 19902: 2007	NORSOK N-004
Stress/ Parameter · **Formulation** · **Limits**	**Stress/ Parameter** · **Formulation** · **Limits**
13.9 Grouted tubular members Subclause 13.9 applies to both fully grouted undamaged members and fully grouted dented tubular members, with the dent depth h limited to either $h \le 0.3D$ or $h \le 10t$. **13.9.2.2 Axial tension** $\sigma_{t,g} \le f_t/\gamma_{R,t,g}$ where $\sigma_{t,g}$ = the axial tensile stress in the steel tubular member due to forces from factored actions, neglecting the grout f_t = the representative axial tensile strength of the steel, $f_t = f_y$ $\gamma_{R,t,g}$ = the partial resistance factor for axial tensile strength of the grouted member, $\gamma_{R,t,g} = 1.05$ Where it can be demonstrated that complete grouting of the tubular has been achieved, f_t may be taken as $1.12f_y$.	**10.8 Repaired and strengthened members and joints** **10.8.2 Grouted tubular members** **10.8.2.1 Axial tension** $N_{Sd} \le N_{tg,Rd} = f_y A_s/\gamma_M$ $\gamma_M = 1.15$ where N_{sd} = design axial force on the grouted section $N_{tg,Rd}$ = design axial tension resistance of the grouted member, composite section A_s = gross steel area = πDt
13.9.2.3 Axial compression $\sigma_{c,g} \le f_{c,g}/\gamma_{R,c,g}$ $\sigma_{c,g}$ = the axial compressive stress in the fully grouted member due to forces from factored actions acting on the transformed area, $\sigma_{c,g} = P/A_{tr}$ $f_{c,g}$ = the representative axial compressive strength of the grouted member, in stress units $= (1.0-0.28\lambda_g^2)f_{ug}$ for $\lambda_g \le 1.34$ $(0.9/\lambda_g^2)\, f_{ug}$ for $\lambda_g > 1.34$ $\lambda_g = (f_{ug}/f_{eg})^{0.5}$ the column slenderness parameter of the grouted member $\gamma_{R,c,g}$ = the partial resistance factor for axial compressive strength of the grouted member, $\gamma_{R,c,g} = 1.18$ P = the axial compressive force in the grouted member due to factored actions A_{tr} = the tranformed area of the fully grouted member = $A_s + A_g/m$ A_s = the cross-sectional area of the steel $= (D-t)\, t\, [\pi - (\alpha_g - \sin\alpha_g)]$ $\alpha_g = 1/\{\cos[1-2h/(D-t)]\}$ $A_g = (D - 2t)^2[\pi - \alpha_g + 0.5\sin(2\alpha_g)]/4$ m = the ratio of elastic moduli of steel and grout, $m = E_s/E_g$ (m=18, in lieu of actual data) E_s = Young's modulus of elasticity for steel E_g = the modulus of elasticity of grout h = the maximum dent depth, if present D = the outer diameter of the steel tubular member t = the thickness of the steel f_{ug} = the axial squash strength of the grouted member, in stress units $= (A_s f_y + 0.67 A_g f_{cu})/A_{tr}$ f_{cu} = the representative unconfined cube strength of the grout, in stress units f_{eg} = the Euler buckling strength of the fully grouted member, in stress units $= \pi^2(E_s I_s + 0.8 E_G I_G)/[A_{tr}(KL)^2]$ K = the effective length factor L = the longer of the unbraced lengths in the y- and z- directions Is = the effective moment of inertia of the steel cross-section $= \{(D-t)^3 t[\pi - \alpha_g - 0.5\sin(2\alpha_g) + 2\sin(\alpha_g)\cos^2(\alpha_g)]/8\} - A_s e_s^2$	**10.8.2.2 Axial Compression** $N_{Sd} \le N_{cg,Rd} = N_{cg}/\gamma_M$ $\gamma_M = 1.15$ $N_{cg} = (1.0-0.28\lambda_g^2)N_{ug}$ for $\lambda_g \le 1.34$ $(0.9/\lambda_g^2)\, N_{ug}$ for $\lambda_g > 1.34$ $\lambda_g = (N_{ug}/N_{eg})^{0.5}$ where N_{sd} = design axial force on the grouted section $N_{cg,Rd}$ = design axial compression resistance of the grouted member N_{ug} = axial yield resistance of the composite cross-section = $A_s f_y + 0.67 A_G f_{cg}$ N_{eg} = elastic Euler buckling load of the grouted member = $\pi^2(E_s I_s + 0.8 E_G I_G)/(kl)^2$ f_{cg} = characteristic cube strength of grout A_s = cross-sectional area of the steel = πDt, for intact sections $= \pi Dt[1-(\alpha-\sin\alpha)/\pi)$, for dented sections A_G = cross-sectional area of the grout = $\pi D^2/4$, for intact sections $= \pi(D^2/4)[1-\alpha/\pi+1/2\sin(2\alpha)/\pi]$, for dented sections Is = effective moment of inertia of grout cross section $= \pi(D^3 t/8)[1-\alpha/\pi-\sin(2\alpha) / (2\pi)+(2\sin\alpha\cos^2\alpha)/\pi]-A_s e_s^2$ I_G = effective moment of inertia of grout cross section $= \pi(D^4/64)[1-\alpha/\pi-\sin(4\alpha) / (4\pi)]-A_G e_G^2$ E_s = modulus of elasticity of the steel E_G = modulus of elasticity of the grout m = modular ration of E_s/E_G e_s = distance from centroid of dented steel section to the centroid of the intact steel section $= D^2 t\sin\alpha(1-\cos\alpha)/(2A_s)$ e_G = distance from centroid of dented grout section to the centroid of the intact grout section $= (D\sin\alpha)^3/(12A_G)$ $\alpha = \cos^{-1}(1-2\delta/D)$ δ = dent depth D = tube diameter

I_g = the effective moment of inertia of the grout cross-section $= \{(D-2t)^4[\pi - \alpha_g + 0.25\sin(4\alpha_g)]/64\} - A_g e_g^2$ $e_s = 0.5(D-t)^2 t \sin(\alpha_g)(1 - \cos\alpha_g)/A_s$ $e_g = (D-2t)^3 \sin^3(\alpha_g)/(12A_g)$	
13.9.2.4 Bending $\sigma_{b,g} = M/Z_e \leq f_{b,g}/\gamma_{R,b,g}$ $\sigma_{b,g}$ = the bending stress due to forces from factored actions and when $\sigma_{b,g} > f_{b,g}$, is to be considered as an equivalent elastic bending stress, $\sigma_{b,g} = M/Z_e$ M = the bending moment in the grouted member due to factored actions $f_{b,g}$ = the representative bending strength of the grouted member, in stress units $= Z_p/Z_e\, f_y\, \delta\, (1+ 0{,}01k)$ $\delta = 1 - 0.5h/D - 1.6(h/D)^2$ $k = 5.5\delta(\rho D/t)^{0.66}$ $\rho = 0.6 f_{cu}/f_y$ $Z_e = \pi/64\,[D^4 - (D-2t)^4]/(D/2)$ elastic section modulus $Z_p = [D^3 - (D - 2t)^3]/6$ plastic section modulus $\gamma_{R,b,g}$ = the partial resistance factor for bending strength of the grouted member, $\gamma_{R,b,g} = 1.05$	**10.8.2.3 Bending** $M_{Sd} \leq M_{g,Rd} = W_{tr}\, f_{bg}/\gamma_M$ where M_{Sd} = design bending moment for the grouted section $M_{g,Rd}$ = design bending resistance of the grouted member W_{tr} = elastic section modulus of the transformed, composite section $\approx 2/D(I_s + I_G/m)$ m = modular ration of $E_s/E_G \approx 18$, in lieu of actual data f_{bg} = charateristic bending strength of grouted member $= 4/\pi\, f_y\, \xi_\delta\, (1+ \xi_m/100)$ $\xi_\delta = 1 - 0.5\,\delta/D - 1.6(\delta/D)^2$ $\xi_m = 5.5\, \xi_\delta\, (0.6 f_{cg}/f_y\, D/t)^{0.66}$
13.9.3.2 Axial compression and bending $(\gamma_{R,c,g}\, \sigma_{c,g})/f_{c,g} + (\gamma_{R,b,g}\, T_1\, \sigma_{c,g})/f_{b,g} + [T_2\, (\gamma_{R,b,g}\, \sigma_{b,g})^2]/f^2_{b,g} \leq 1.0$ for $(\gamma_{R,c,g}\ \sigma_{c,g})/f_{c,g} \geq K_2/K_1$ $(\gamma_{R,b,g}\, \sigma_{b,g}) / f_{b,g} \leq 1.0$ for $(\gamma_{R,c,g}\ \sigma_{c,g})/f_{c,g} < K_2/K_1$ where $T_2 = 4K_3/K_2$ $T_1 = 1 - K_2/K_1 - T_2$ $K_1 = Ncg/Nug$ $1.0-0.28\lambda_g^2$ for $\lambda_g \leq 1.34$ $0.9/\lambda_g^2$ for $\lambda_g > 1.34$ $K_2 = K_{20}\,[115-30(2\beta-1)(1.8- \theta)-100\lambda_g]/[50(2.1-\beta)]$ $0 \leq K_2 \leq K_{20}$ $K_{20} = (0.9\theta^2 + 0.2) \leq 0.75$ $K_3 = K_{30} + \lambda_g\,[0.5\beta+0.4)(\theta^2 - 0.5) +0.15]/(1 + \lambda_g^3)$ $K_{30} = 0.04 - \theta/15 \geq 0$ $\theta = [0.67A_g(f_{cu} + C_1 f_y\, t/D)]/(f_{ug}\, A_{tr})$ β = 1, provided no end moments apply, otherwise it is the ratio of the smaller to the larger end moment $C_1 = 4\, \varphi\varepsilon/(1+\varphi +\varphi^2)^{-0.5}$ $\varphi = 0.02(25-KL/D) \geq 0$ $\varepsilon = 0.25(25-kl/D) \geq 0$ K = effective length factor L = length of member	**10.8.2.5 Combined axial compression and bending** $N_{Sd}/N_{cg,Rd} + T_1\, M_{Sd}/M_{g,Rd} + T_2\, (M_{Sd}/M_{g,Rd})^2 \leq 1$ for $N_{Sd}/N_{cg,Rd} \geq K_2/K_1$ $M_{Sd}/M_{g,Rd} \leq 1$ for $N_{Sd}/N_{cg,Rd} < K_2/K_1$ where $T_2 = 4K_3/K_2$ $T_1 = 1 - K_2/K_1 - T_2$ $K_1 = Ncg/Nug$ = $1.0-0.28\lambda_g^2$ for $\lambda_g \leq 1.34$ $0.9/\lambda_g^2$ for $\lambda_g > 1.34$ $K_2 = K_{20}\,[115-30(2\beta-1)(1.8-\gamma)-100\lambda_g]/[50(2.1-\beta)]$ $0 \leq K_2 \leq K_{20}$ $K_{20} = (0.9\gamma^2 + 0.2) \leq 0.75$ $K_3 = K_{30} + \lambda_g\,[0.5\beta+0.4)(\gamma^2 - 0.5) +0.15]/(1 + \lambda_g^3)$ $K_{30} = 0.04 - \gamma/15 \geq 0$ $\gamma = [0.67A_g(f_{cg} + C_1 f_y\, t/D)]/N_{ug}$ β = 1, provided no end moments apply, otherwise it is the ratio of the smaller to the larger end moment $C_1 = 4\, \varphi\varepsilon/(1+\varphi +\varphi^2)^{-0.5}$ $\varphi = 0.02(25-kl/D) \geq 0$ $\varepsilon = 0.25(25-kl/D) \geq 0$ k = effective length factor l = length of member

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.

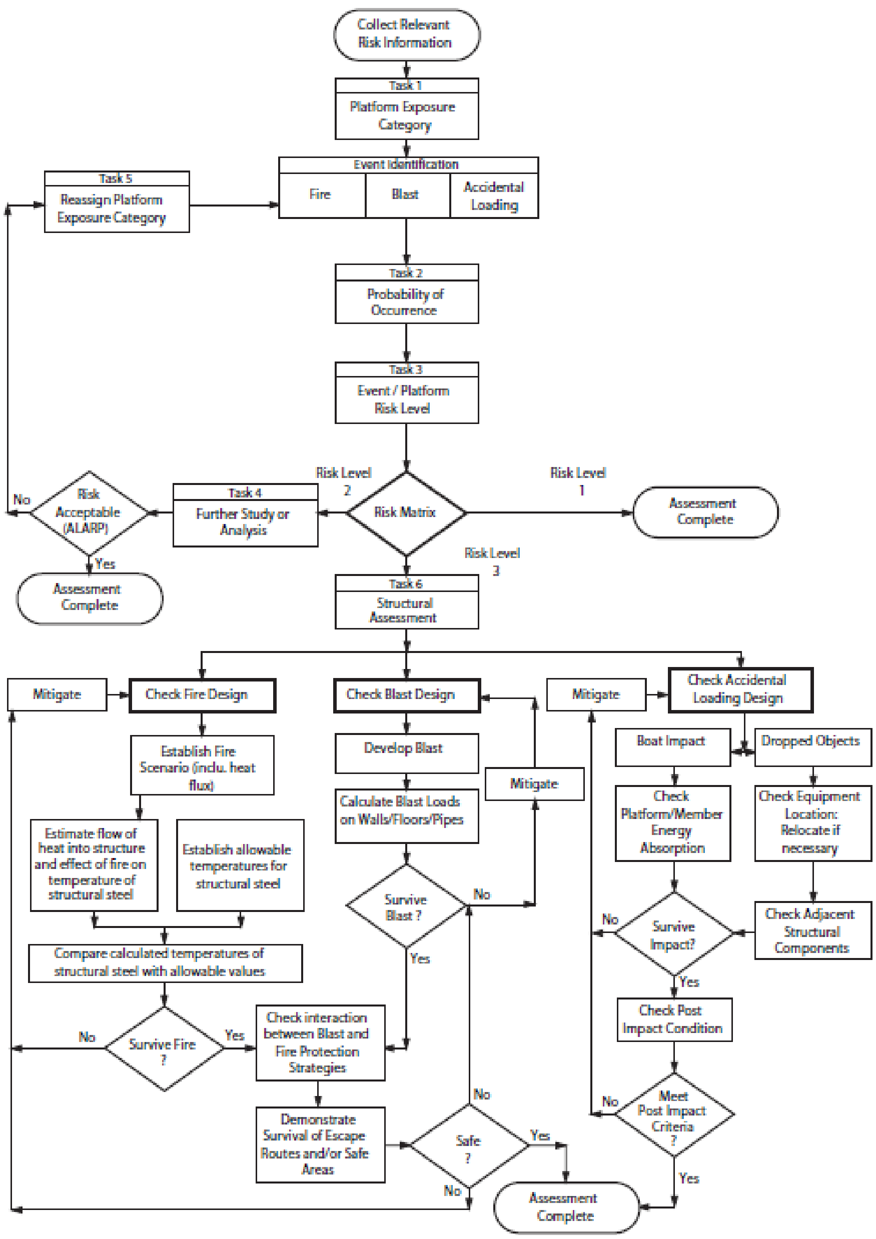


Figure 8-1 Assessment Process (API RP 2A)

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 8-1 Blast Strain Limits

Type of Loading	Strain Limit
Tension	5%
Bending or compression	
Plastic sections	5%
Compact sections	3%
Semi-compact sections	1%
Other sections	< yield strain

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.

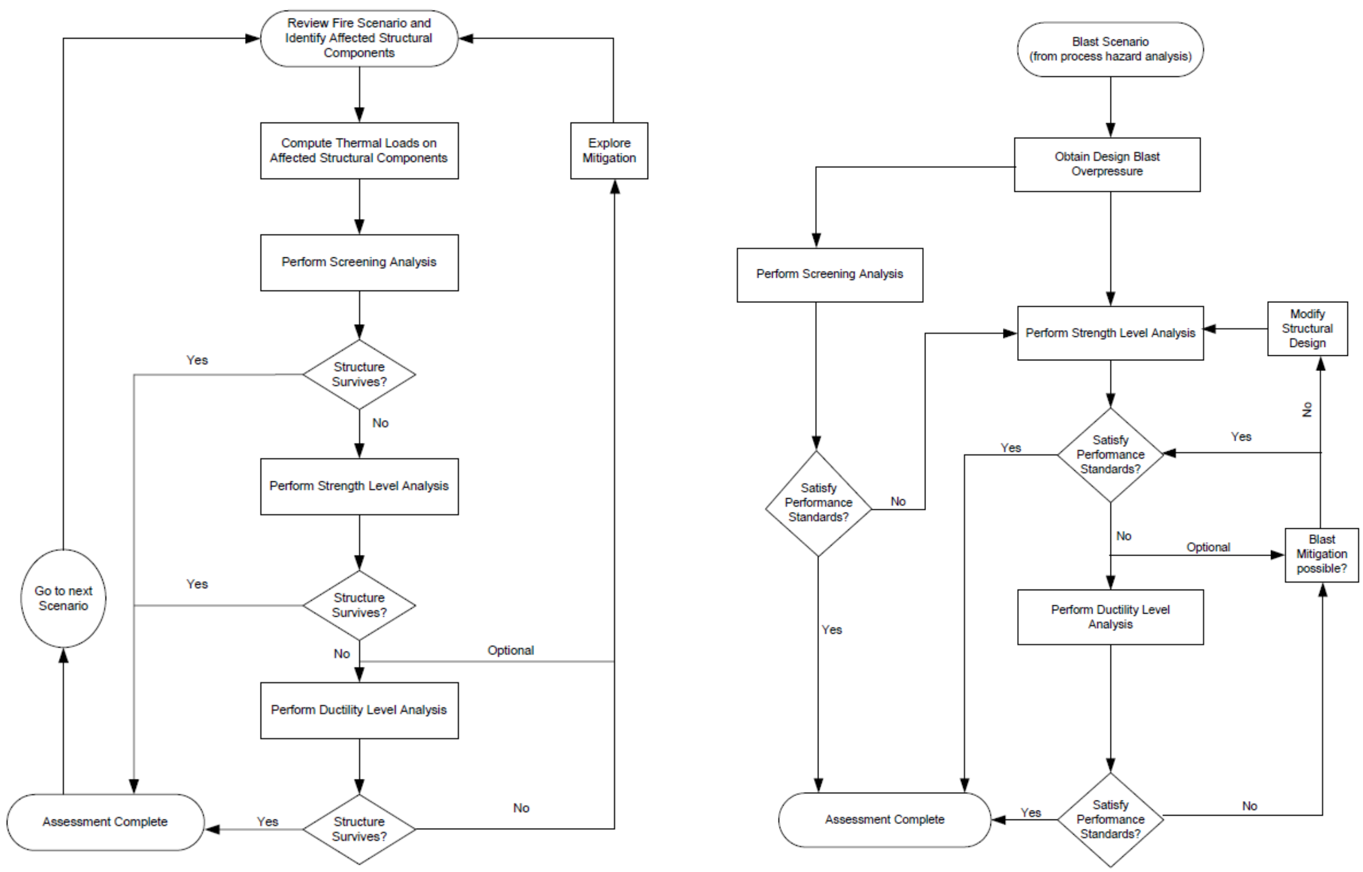


Figure 8-2 Structural Fire and Blast Assessment

ISO 13702 contains requirements and recommendations for control and mitigation of fires and explosions. New ISO 19901-3 (2010) contains more specific requirements for topsides structures.

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

	API RP 2A- WSD Section 18	ISO 19902 Clause 10	NORSOK N-004 Annex A
Assessment Process	**- Implementing preventive measures has historically been, and will continue to be, the most effective approach in minimizing the probability of occurrence of an event and the resultant consequences of the event.** **- In U.S. GOM, considerations of preventative measures coupled with established infrastructure, open facilities and relatively benign environment have resulted in a good safety history. Detailed structural assessement should therefore not be necessary for typical U.S. GOM-type structures and environment.** **- Assessment Process** 1. Initially screen those platforms considered to be at low risk, thereby not requiring detailed structural assessment. 2. Evaluate the structural performance of those platforms considered to be at high risk from a life safety and/or consequences of failure point of view, when subjected to fire, blast, and accidental loading events.	**- In this standard, only designing for hazards for structures of exposure level L1 is quatified.** **- The main hazards that faced by an offshore structure include:** a) vessel collisions b) Dropped objects c) fire and explosions d) abnormal environmental actions, including abnormal seismic actions **- When checking accidental limit states (ALS) for accidental events, all partial action and resistance factors may be set to 1.0**	- The overall goal of the design against accidental actions is to achieve a system where the main safety functions of the installation are not impaired. - The material factor to be used for checks of accidental limit states is $\gamma_M = 1.0$
Ship Collisions	**- The platform should survive the initial collision and meet the post-impact criteria.** **- All exposed elements at risk in the collision zone of an installation should be assessed for accidental vessel impact during normal operations.** 1. The collision zone is the area on any side of the platform that a vessel could impact in an accidental situation during normal operations. 2. The vertical height of the collision zone should be determined from the considerations of vessel draft, operational wave height and tidal elevation. 3. Elements carrying substantial dead load, except for platform legs and piles, should not be located in the collision zone. If such elements are located in the collision zone they should be assessed for vessel impact. **- Energy Absorption** An offshore structure will absorb energy primarily from: a. Localized plastic deformation of the tubular wall b. Elastic/platstic bending of the member c. Elastic/platstic elongation of the member d. Fendering device, if fitted e. Global platform deformation (that is, sway) f. Ship deformation and/or rotation **- Damage Assessment** Two cases should be considered: 1. **Impact (energy absorption and survival of platform)** Primary framework should be designed and configured to absorb energy during impact, and to control the consequences of damage after impact. Some permanent deformation of members may be allowable in this energy absorption. The kinetic energy of a vessel: $E = 0.5\ a\ m\ v^2$ Where E = the kinetic energy of the vessel a = added mass factor, (1.4 for broadside collision, 1.1 for bow/stern collision) m = vessel mass v = velocity of vessel at impact For platforms in mild environments and reasonably close to their base of supply, the following minimum requirements should be used, unless other criteria can be demonstrated: Vessel Mass = 1100 short tons (1,000 metric tons) Impact Velocity = 1.64 ft/sec (0.5 m/sec) **The 1100-short-ton vessel is chosen to represent a typical 180-200-foot-long supply vessel in the GoM.** 2. **Post-impact (platform to meet post-impact criteria)** a)The platform should retain sufficient residual strength after impact to withstand the one-year environmental storm loads in addition to normal operating loads. b) Special attention should be given to defensible representation of actual stiffness of damaged members or joints in the post-impact assessment. Damaged members may be considered totally ineffective providing their wave areas are modeled in the analysis.	**- Vessel impact shall be addressed for the structures with exposure levels L1 and L2.** **- Two energy levels shall be considered:** a) low energy level, representing the most frequent condition, based on the type of vessel that would routinely approach alongside the platform (e.g. a supply boat) and that would have a velocity representing normal manoeuvring of the vessel approaching, leaving, or standing alongside the platform This level is a serviceability limit state to which the owner can set his own requirements based on practical and economical considerations. b) high energy level, representing a rare condition, based on the type of vessel that would operate in the platform vicinity, drifting out of control in the worst sea state in which it would be allowed to operate close to the platform This level represents an ultimate limit state in which the structure is damaged but progressive collapse should not occur. **- The kinetic energy of a vessel:** $E = 0.5\ a\ m\ u^2$ Where E = the kinetic energy of the vessel a = added mass factor, (1.4 for broadside collision, 1.1 for bow/stem collision) m = vessel mass u = velocity of vessel at impact a) The added mass coefficients given above are typical values for large (5000 t displacement) supply vessels. For smaller vessels, a value slightly higher than 1.4 should be applied, e.g. 1.6 for a typical 2500 t supply vessel. b) For the northern North Sea, a vessel mass can be 8000t, whereas in the southern North Sea a mass of around 2500 t is more normal. c) For **GoM** structures in mild environments and reasonably close their base of supply, a 1000 t vessel represents a typical 55 m to 60 m (180 ft to 200 ft) supply vessel. For deeper and more remote locations in the GoM the vessel mass can be different.The masses of vessels that could collide with the platform when drifting out-of-control should be specifically considered. d) For low energy impacts, a vessel velocity of 0.5 m/s is commonly used, representing a minor accidental "bump" during normal manoeuvring of the vessel while loading or unloading or while standing alongside the platform. e) For high energy conditions, a vessel velocity of 2 m/s is commonly used, representing a vessel drifting out-of-control in a sea state with significant wave height of approximately 4 m.	**- The load bearing function of the installation shall remain intact with the damages imposed by the ship collision action. In addition, the residual strength requirements shall be complied with.** **- Methods used to determine the structural effects from ship collision:** a) non-linear dynamic finite element analysis b) energy considerations combined with simple elastic-plastic methods **- Three levels for the strain energy dissipation consideration:** 1) local cross-section 2) component/sub-structure 3) total system **- Strain energy** Fixed installations $E_s = \frac{1}{2}(m_s + a_s)v_s^2$ Articulated columns $E_s = \frac{1}{2}(m_s + a_s)\frac{\left(1-\frac{v_i}{v_s}\right)^2}{1+\frac{m_s z^2}{J}}$ Compliant installations $E_s = \frac{1}{2}(m_s + a_s)v_s^2\frac{\left(1-\frac{v_i}{v_s}\right)^2}{1+\frac{m_s + a_s}{m_i + a_i}}$ m_s = ship mass a_s = ship added mass v_s = impact speed m_i = mass of installation a_i = added mass of installation v_i = velocity of installation J = mass moment of inertia of installation (including added mass) with respect to effective pivot point Z = distance from pivot point to point of contact Jacket structures can normally be considered as fixed. Floating platforms (semi-submersibles, TLPs, production vessels) can normally be considered as compliant. Jack-ups may be classified as fixed or compliant. **- More details provided in this provision** A.3.5 Ship Collision Forces A.3.6 Force-deformation relationships for denting of tubular members A.3.7 Force-deformation relationships for beams A.3.8 Strength of connections A.3.9 Strength of adjacent structure A.3.10 Ductility limits A.3.11 Resistance of large diameter, stiffened columns A.3.12 Energy dissipation in floating production vessels A.3.13 Global integrity during impact

Dropped objects

- Certain locations such as crane loading areas are more subject to dropped or swinging objects.
- The probability of occurrence may be reduced by following safe handling practices.
- The consequnces of damage may be minimized by considering the location and protection of facilities and critical platform areas. Operation procedures should limit the exposure of personnel to overhead material transfer.
-The platform should survive the initial impact and meet the post-impact criteria as defined for vessel collision.

- When evaluating the impact risk from dropped objects, the nature of all crane operations in the platform vicinity shall 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.
- Robustness in relation to dropped objects should be incorporated into the design by indirect means such as
a) avoiding weak elements in the structure (particularly at joints)
b) selecting materials with sufficient toughness
c) ensuring that critical components are not placed in vulnerable locations

- Dropped objects are rarely critical to the global integrity of the installation and will mostly cause local damages. The major threat to global integrity is probably puncturing of buoyancy tanks, which could impair the hydrostatic stability of floating installations.
- The structural effects may either be determined by non-linear dynamic finite element analysis or by energy considerations combined with simple elastic-plastic methods.
- Kinetic energy of a falling object:

$E_{kin} = 0.5\, mv^2$ for objects falling in air

$E_{kin} = 0.5\, (m+a)v^2$ for objects falling in water

a = hydrodynamic added mass for considered motion
For impact in air the velocity is given by
$v = (2gs)^{0.5}$
s = travlled distance from drop point
$v = v_0$ at sea surface

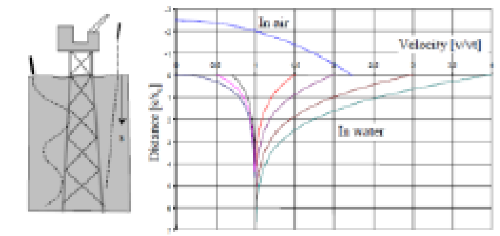


Figure A.4-1 Velocity profile for objects falling in water

Table A.4-1 Terminal velocities for objects falling in water.

Item	Weight [kN]	Terminal velocity [m/s]
Drill collar	28	23 to 24
Winch.	250	
Riser pump	100	
BOP annular preventer	50	16
Mud pump	330	7

$$v_t = \sqrt{\frac{2g(m-\rho_w V)}{\rho_w C_d A_p}}$$ = terminal velocity for the object

$$s_c = \frac{m+a}{\rho_w C_d A_p} = \frac{v_t^2\left(1+\frac{a}{m}\right)}{2g\left(1-\frac{\rho_w V}{m}\right)}$$ = characteristic distance

ρ_w = density of sea water
C_d = hydrodynamic drag coefficient for the object in the considered motion
m = mass of object
A_p = projected cross-sectional area of the object
V = object displacement

- Resistance/Energy dissipation

1) Stiffened plates subjected to drill collar impact

The energy dissipated in the plating subjected to drill collar impact is given by:

$$E_{sp} = \frac{R^2}{2K}\left(1+0.48\frac{m_i}{m}\right)^2 \quad \text{(A.4.6)}$$

$$K = \frac{1}{2}\pi f_y t\left(\frac{1+5\frac{d}{r}-6c^2+6.25\left(\frac{d}{2r}\right)^2}{(1+c)^2}\right)$$: stiffness of plate enclosed by hinge circle

f_y = characteristic yield strength
$c = -e^{-3.5\left(1-\frac{d}{2r}\right)}$
$R = \pi d t \tau$ = contact force for $\tau \le \tau_u$ see A.4.5.1 for τ_u
$m_i = \rho_p \pi r^2 t$ = mass of plate enclosed by hinge circle
m = mass of dropped object
ρ_p = mass density of steel plate
d = smaller diameter at threaded end of drill collar
r = smaller distance from the point of impact to the plate boundary defined by adjacent stiffeners/girders, see Figure A.4-3.

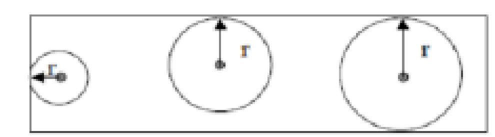


Figure A.4-3 Definition of distance to plate boundary

2) Limits for energy dissipation

a) pipes on plate

The maximum shear stress for plugging of plates due to drill collar impacts may be taken as:

$$\tau_u = f_u\left(0.42+0.41\frac{t}{d}\right) \quad \text{(A.4.7)}$$

f_u = ultimate material tensile strength

b) Blunt objects

For stability of cross-sections and tensile fracture, see A.3.10

Fire	- If the assessment process identified that a significant risk of **fire** exists, fire 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. - If the assessment process identified that a significant risk of **blast** exists, blast should be considered as a load condition; the blast assessment need to demonstrate that the escape routes and safe areas survive. - The fire and blast analyses should be performed together and the effects of one on the other carefully analyzed. **- Fire as a load condition requires that the following be defined:** 1. Fire scenario: fire type, location geometry and intensity 2. Heat flow charateristics from the fire to unprotetcted 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 "view" factor. For engulfed members, a configuration factor of 1.0 is used. 3. Properties of steel at elevated temperatures and where applicable a) thermal properties - required for the calculation of the steel temperature b) mechanical properties - used to verify that original design still meets the strength and serviceablility requirements. 4. Properties of fire protection systems (active and passive) a) They may be required to ensure that the maximum allowable member temperatures are not exceeded for a designated period when fire occur. b) They may also serve to prevent escalation of the fire. c)The designated period of protection is based on either the fire's expected duration or the required evacuation period. **- Design for fire** **There are the following approaches to be used in the design for fire:** **1. Zone method** a) The zone method of design assigns a maximum allowable temperature that can develop in a steel member without reference to the stress level prior to the fire. b) The assumption of this method is that a member utilization ratio calculated using basic (AISC) allowable stress will remian 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. This assumption is valid when the nonlinear stress/strain charateristics of the steel may be linearized such that the yield strength reduction factor is matched by the reduction in Young's modulus (as for a 0.2% strain). c) With an unmatched reduction in both yield strength and Young's modulus, the governing design condition may be affected; thus, the zone method may not be applicable. **2. Linear elastic method (e.g. a working stress code check)** a) A maximum allowable temperature in a steel member is assigned based on the stress level in the member prior to the fire, such that as the temperature increases, the member utilization (UR) remains below 1.00 (the member continues to behave elastically) b) With an unmatched 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. **3. Elastic-plastic method (e.g. a progressive collapse analysis)** a) A maximum allowable temperature in a steel member is assigned based on the stress level in the member prior to the fire. A the temperature increases, the member utilization (UR) may go above 1.00 (the member behavior is elastic palstic). b) A nonlinear analysis to be performed to verify that the structure will not collapse and will still meet the serviceablility criteria. **Notes**: 1) Regardless of the design method, the linearization 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 matched reduction in yield strength and Young's modulus, but has the disadvantage of limiting the allowable temperature of the steel to 400°C.	The industry assoications 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 - UKOOA, which are suited to larger platforms operated in a safety case regime - NORSOK which contains explicit analytical requirements. - ISO 13702 contains requirements and recommendations for fires and explosions	- The assessment of fire load effect and mechanical response shall be based on either a) simple calculation methods applied to individual members - should be based on the provisions given in NS-ENV 1993-1 Eurocode 3: Design of steel structures, Part 1.2. General rules - Structural fire design b) general calculation methods - should be based on the provisions given in NS-ENV 1993 1-1, Part 1.2, Section 4.3 - Assessment of ultimate strength is not needed if the maximum steel temperature is below 400°C., but deformation criteria may have to be checked for impairment of main safety function.

Blast

1. Due to the complexity in predicting blast loads, the pressure-time curves should be generated by an expert in this field.
2. A blast can cause two types of loading:
a) Overpressure - results from increases in pressure due to expanding combustion products It likely to govern the design of structures such as blast walls and floor/roof systems.
b) Drag loading - caused by last-generated wind
Critical piping, equipment, and other items exposed to the blast wind should be designed to resist the predicted drag loads
3. Environmental loads can be neglected in a blast analysis.
4. Structural Resistance
- Strength limit
Failure is defined to occur when the design load or load effects exceed the design strength.
- Deformation limit
1) No part of the structure impinges on critical operational equipment
2) The deformations do not cause collapse of any part of the structure that supports the safe area, escape routes, and embarkation points within the endurance period. A check should be peroformed to ensure that integrity is maintained if subsequent fire occurs.
3) Deformation limits can be based on a maximum allowable strain or an absolute displacement:
a) Strain limit: most types of structural steel used offshore have a minimum strain capacity of approximately 20 percent at low strain rates.

They usually have sufficient toughness against brittle fracture not to limit strain capacity significantly at the high strain rates associated with blast response for nominal U.S. GOM temperature range.
Recommended strain limits for different types of loading are as follows:

Type of Loading	Strain Limit
Tension	5%
Bending or compression	
Plastic sections	5%
Compact sections	2%
Semi-compact sections	1%
Other sections	< yield strain

The strain limits above assume that lateral torsional buckling is prevented.
b) Absolute limits - adopted where there is a risk of a deforming element striking some component, usually process or emergency equipment or key structure
5 Determination of Yield Point
a) Actual yield stress, usually higher than the minimum specific, should be used in the analysis; strain rates and strain hardening effects should be included in determining yield stress and general material behavior.
b) If maximum reaction forces are required, it is necessary to design using an upper bound yield stress. If maximum deflections are required, the design should use a lower bound yield stress.
6 Analysis Methods
a) Static analysis (a long load duration relative to the structure's natural period): The peak pressure should be used to define the loading.
b) Dynamic analysis (load duration is near to the structure's natural period): The actual pressure-time curve can be applied to the structure.
7. Mitigation
The blast effects can generally be minimized by making the vent area as large as possible;
To minimize blast pressure, ven areas should be located as close as possible to likely ignition sources.

The industry assoications 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
- UKOOA, which are suited to larger platforms operated in a safety case regime
- NORSOK which contains explicit analytical requirements.
- ISO 13702 contains requirements and recommendations for fires and explosions

- The reponse to explosion loads may either be determined by
a) non-liear dynamic finite element analysis
b) simple calculation models based on SDOF analogies and elastic-plastic methods of analysis
- Suggested analysis model and reference to applicable resistance function are listed in Table A.6-1

Table A.6-1 Analysis models

Failure mode	Simplified analysis model	Resistance models	Comment
Elastic-plastic deformation of plate	SDOF	A.6.8	
Stiffener plastic – plate elastic	SDOF	Stiffener: A.6.9.1and A.6.9.2 Plate: A.6.8.1	Elastic, effective flange of plate
Stiffener plastic – plate plastic	SDOF	Stiffener: A.6.9.1and A.6.9.2 Plate: A.6.8	Effective width of plate at mid span. Elastic, effective flange of plate at ends.
Girder plastic – stiffener and plating elastic	SDOF	Girder: A.6.9.1and A.6.9.2 Plate: A.6.8	Elastic, effective flange of plate with concentrated loads (stiffener reactions). Stiffener mass included
Girder plastic – stiffener elastic – plate plastic	SDOF	Girder: A.6.9.1and A.6.9.2 Plate: A.6.8	Effective width of plate at girder mid span and ends. Stiffener mass included
Girder and stiffener plastic – plate elastic	MDOF	Girder and stiffener: A.6.9.1and A.6.9.2 Plate: Section A.6.8	Dynamic reactions of stiffeners → loading for girders
Girder and stiffener plastic – plate plastic	MDOF	Girder and stiffener: A.6.9.1and A.6.9.2 Plate: A.6.8	Dynamic reactions of stiffeners → loading for girders

By girder/stiffener plastic is understood that the maximum displacement w_{max} exceeds the elastic limit w_y

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.

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

Mass of lifted object[a] tonnes	Gross weight, W kN	k_{DAF} in air			
		offshore	inshore	onshore[b]	
				moving	static
≤ 100	$W \leq 1\,000$	1,30	1,15	1,15	1,00
from 100 to 1 000	$1\,000 < W \leq 10\,000$	1,20	1,10	1,10	1,00
from 1 000 to 2 500	$10\,000 < W \leq 25\,000$	1,15	1,05	1,05	1,00
from 2 500	$25\,000 < W$	1,10	1,05	1,05	1,00

[a] This column is included to facilitate the comparison with weight reporting.

[b] Lifts by land-based cranes involved with marine operations such as loadouts.

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 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):

$$F_d = k_{DAF}\ \gamma_{f,dl}\ \gamma_{f,lf}\ (\gamma_{f,GT}\ G_T + \gamma_{f,QT}\ Q_T + \gamma_{f,T}\ T) \qquad (9.1)$$

$$S = k_{DAF}\ \gamma_{f,dl}\ \gamma_{f,lf}\ \gamma_{f,Sun}\ S_{un} \qquad (9.2)$$

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 Temporay Conditions Comparision

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>
<tr><th></th><th>API RP 2A- WSD Section 2.4 & Section 12</th><th>ISO 19902 Clause 8 and 22</th><th>NORSOK N-004 Annex K (ISO 19901-6)</th></tr>
<tr><td>General</td>
<td>- For those installation forces that are experienced only during transportation and launch, and which include environmental effects, basic allowable stresses for member design may be increased by 1/3.</td>
<td>- Internal forces due to factored actions (8.2.4.1)
$$F_d = \gamma_{f,GT}\, G_T + \gamma_{f,QT}\, Q_T + \gamma_{f,T}\, T \qquad (8.2\text{-}1)$$
where
G_T is the action imposed either by the weight of the structure in air, or by the submerged weight of the structure in water, during the transient situation being considered, including any permanent equipment or other objects and any piles or conductors installed on the structure, as well as any ballast installed in or carried by the structure;
Q_T is the action imposed by the weight of the temporary equipment or other objects, including any rigging installed or carried by the structure, during the transient situation being considered;
T represents the actions from the transient situation being considered, including:
a) when appropriate, environmental actions,
b) when appropriate, a suitable representation of dynamic effects (see A.8.1 and 8.2.4.2),
c) for lifting, the effects of fabrication tolerances and variances in sling length as detailed in 8.3.3 and for a dual lift as detailed in 8.3.4,
d) for loadout, allowances for misalignment as detailed in 8.5,
e) for transportation, any hydrostatic and hydrodynamic actions on the structure, including any inertial actions resulting from accelerations of the structure (see 8.6), and
f) for installation, the lifting actions and hydrostatic pressure actions on the structure (see 8.7);
$\gamma_{f,GT}$, $\gamma_{f,QT}$ and $\gamma_{f,T}$ are the partial action factors.
The three design situations in Table 8.2-1 shall all be considered.
Table 8.2-1 — Partial action factors for calculating internal forces
<table>
<tr><th rowspan="2">Situation</th><th colspan="3">Partial action factor</th></tr>
<tr><th>$\gamma_{f,GT}$</th><th>$\gamma_{f,QT}$</th><th>$\gamma_{f,T}$</th></tr>
<tr><td>1</td><td>1,3</td><td>1,3</td><td>1,0</td></tr>
<tr><td>2</td><td>1,1</td><td>1,1</td><td>1,35</td></tr>
<tr><td>3</td><td>0,9</td><td>0,9</td><td>1,35</td></tr>
<tr><td colspan="4">NOTE Situation 1 governs for components in which permanent and variable action effects are dominant. Situation 2 governs for components in which transient action effects are dominant and in which the permanent and variable actions increase the magnitudes of the internal forces. Situation 3 governs for components in which transient action effects are dominant but in which the permanent and variable actions decrease the magnitudes of the internal forces.</td></tr>
</table>
- Internal forces due to unfactored actions (8.2.4.2)
$F_{un} = G_T + Q_T + T$
$S = \gamma_{f,Sun}\, S_{un}$
Where F_{un} = total action due to the unfactored actions G_T, Q_T and T defined above;
S_{un} = the internal force resulting from F_{un}
$\gamma_{f,Sun}$ = partial factor to be applied to S_{un}, usually 1.3
- Guidance is also provided in ISO 19901-6.</td>
<td>- NORSOK N-004 Clause K.4.4.6 Installation analysis states that Transport and installation design and operation shall comply with the requirements given in NORSOK J-003. NORSOK J-003 is voided as a consequence of ISO 19901-6 Marine Operations having been issued as DIS.</td></tr>
<tr><td></td>
<td>- Lifting forces on padeyes and on other members of the structure should include both vertical and horizontal components, the latter occurring when lifting slings are other than vertical. Lifting forces on the lift should include buoyancy as well as forces imposed by the lifting equipment.
- To compensate for any side loading on lifting eyes which may occur, in addition to the calculated horizontal and vertical components of the static load for the equilibrium lifting condition, 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 with the static sling load. This horizontal force should be applied perpendicular to the padeye at the center of the pinhole.
-Static Loads (2.4.2.b)
a) When suspended, the lift will occupy a position such that the center of gravity of the lift and the centroid of all upward acting forces on the lift are in static equilibrium. The position in this state should be used to determine forces in the structure and in the slings.
b) The movement of the lift as it is picked up and set down should be taken into account in determining critical combinations of vertical and horizontal forces at all points, including those to which lifting slings are attached.
- Dynamic Load Factors (2.4.2.c)
a) For lifts to be made at open, exposed sea , padeyes and other internal members (and both end 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 the calculated static loads. All other structural members transmitting lifting forces should be designed using a minimum load factor of 1.35.
b) For other marine situations, the selection of load factors should meet the expected local conditions but should not be less than a minimum of 1.5 and 1.15 for the two conditions</td>
<td>-Dynamic Effects (8.3.2)
A dynamic amplication factor (DAF), k_{DAF}, accounting for dynamic effects of the crane taking up the load and for movements of the crane or of the lifted structure, shall be derived from the following:
a) For offshore lifts in air:
1) k_{DAF} = 1.10 for heavy lift by semi-submersible crane vessel
2) k_{DAF} = 1.30 in other cases; the lower DAF vaule may be used based on special investigations, but shall not less than 1.10;
b) For lifts in air, onshore or in sheltered waters,
k_{DAF} = 1.10
c) For lifts partially or fully in water, k_{DAF} shall be specially investigated taking into account factors including the lift arrangement, the orientation of the lifted structure, the ratio of the allowable hook load to the lifted weight, the drag loads on the lifted structure and the motions of the boom tip in the environmental conditions in which the lift is to be made
d) More details see 19901-6 Clause 18.
- Effect of Tolerances (8.3.3)
a) The requirements and partial action factors here are intended to apply to the situations where fabrication misalignments are consistent with the tolerances specified in Annex G 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.
b) The results sling force should be increased by a factor of not less than 1.25 (1.15 for floating spreader beams) to determine the required safe working load of the sling in 8.3.8.
c) The effectof tolerances in a lift analysis of a standard four-point life may be taken into account by the one of the following methods:</td>
<td>- 19901-6, Clause 18 gives requirements and guidance for the design and execution of listing operations (onshore, inshore and offshore). It covers lifting operations by floating crane vessels, including crane barges, crane ships and semi-submersible crane vessels. Onshore lifts by land-based cranes are also included when they form part of a marine operation such as a loadout.
- Additional information on lifting operations can be found in ISO 19902:2007, Clause 8 and 22.</td></tr>
</table>

Lifting	previously listed. c) For typical fabrication yard operations where both the lifting derrick and the structure or components to be lifted are land-based, dynamic load factors are not required. For special procedures where unusual dynamic loads are possible, appropriate load factors may be considered. **- Allowable stresses (2.4.2.d)** a) basic allowable stresses as specified in Section 3.1 b) The AISC increase in allowable stresses for short-term loads should not be used. c) All critical structural connections and primary members should be designed to have adequate reserve strength to ensure structural integrity during lifting. **- Effect of Tolerances (2.4.2.e)** a) The load factors recommened in 2.4.2c are intended to apply to situations where fabrication tolerances do not exceed the requirements of 11.5, and where the variation in length of slings does not exceed plus or minus 1/4 of 1% of nominal sling strength, or 1.5 inches. b) The total variation from the longest to the shortest sling should not be greater than 1/2 of 1% of the sling length or 3 inches. c) If either fabrication tolerance or sling length tolerance exceeds these limits, a detailed analysis taking into account these tolerances should performed to determine the redistribution of forces on both slings and structural members. The same type analysis should also be performed when unusual deflections of particularly stiff structural systems may also affect load distribution. **- Slings, Shackles and Fittings (2.4.2.f)** a) For normal offshore conditions, slings should be selected to have a factor of safety of 4 for the manufacturer's rated minimum breaking strength of the cable compared to static sling load. b) The static sling load should be the maximum load on any individual sling, as calculated in 2.4.2a, b, and e, by taking into account all components of loading and the equilibrium position of the lift. c) This factor of safety should be increased when usually severe conditions are anticipated, and may be reduced to a minimum of 3 for carefully controlled conditions. d) Shackles and fittings should be selected so that the manufacturer's rated working load is equal to or greater than the static sling load, provided the manufaturer's specifications include a minimum factor of safety of 3 compared to the minimum breaking strength.	account by the one of the following methods: 1) an analysis with one pair of opposite slings assumed to carry 75% and the other pair of 25% of the hook force, and vice versa 2) an analysis with modifying sling lengths, e.g. two diagonally opposite slingd with increased length, each by an amount corresponding to the total tolerance, fo each diagonal in turn. **- Member and joint strength (8.3.6)** $F_d = k_{DAF}\, \gamma_{f,dl}\, \gamma_{f,lf} (\gamma_{f,GT}\, G_T + \gamma_{f,QT}\, Q_T + \gamma_{f,T}\, T)$ $S = k_{DAF}\, \gamma_{f,dl}\, \gamma_{f,lf}\, \gamma_{f,Sun}\, S_{un}$ Where $\gamma_{f,dl}$ = rigging factor, specified in 8.3.4; 1.10 for dual lift, and 1.00 for single crane lifts gf,lf = local factor, specified in 8.3.5; a) For lifting attachments (padeyes, trunnions, padears), spreader beams, and internal members (including both end connections) framing into the joint where the lifting attachment is attached and transmitting lift forces: = 1.25 (for a lift in open waters) = 1.15 (for a lift onshore or in sheltered waters) b) For other structural members = 1.00 **- Lifting attachments (8.3.7)** a) Lifting attachments and the connections to the supporting structural members shall be designed for a lateral force of 5% of the sling force, in addition to the calculated horizontal and vertical components of the sling force (including DAF, rigging factor, local factor and partial action factors) for equilibium lifting condition. b) This lateral forces acts simultaneously with the static sling force and shall be applied perpendicular to the lifting attachment at the centre of the pinhole or tubular. Where a spreader bar is directly connected to the padeyes, a lateral force of 8% shall be used. c) Where two slings are connected to one padeye, or where a sling is doubled over a trunnion, the padeye or trunnion should be designed for a 45:55% split of the lift point force between the two slings. **- Slings, Shackles and fitting (8.3.8)** a) For normal offshore conditions, 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. b) The total resistance factor should be increased when unusually severe conditions are anticipated. Conversely, the total resistance factor may be reduced to a minimum of 3.0 for carefully controlled conditions. c) Where two slings are connected to one padeye, or where a sling is doubled over a trunnion, the padeye or trunnion should be designed for a 45:55% split of the lift point force between the two slings. d) Shackles and fittings should be such 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 1.0 on minimum breaking strength.	
Loadout	**- Direct Lift (2.4.3.a)** Lifting forces for a structure loaded out by direct lift onto the transportation barge should be evaluated only if the lifting arrangement differs from that to be used in the installation, since lifting in open water will impose more severe conditions. **- Horizontal Movement Onto Barge(2.4.3.b)** Structures skidded onto transportation barges are subject to load conditions resulting from movement of the barge due to tidal fluctuations, nearby marine traffic and/or change in draft, load conditions imposed by location, slope and/or settlement of supports at all stages of the skiding operation. Since movement is normally slow, impact need not be considered.	**- Direct Lift (8.5.1)** Action on a structure that is lifted onto the transportation barge shall be evaluated in accordance with 8.3. If the lifting arrangement is the same as that used to offload the structure from the transportation barge at sea, it will suffice to check the latter load cases only. **- Horizontal movement onto barge (8.5.2)** Structures skidded onto transportation barges are subject to actions resulting from movement of the barge due to tidal fluctuations, nearby marine traffic and/or change in draft, load conditions imposed by location, slope and/or settlement of supports at all stages of the skiding operation. Since movement is normally slow, impact need not be considered. **- Self-floating structures (8.5.3)** Self-floating structures skidde directly into the water at the fabrication yard shall be analysed to determine the actions on the structures as they move down the slipways and into the floating position. Consideration should be given to local enviromental conditions and dynamically induced forces.	- 19901-6, Clause 11 applied to the loadout of various types of structure, including, but not limited to, steel and concrete structures, TLPs, spars, FPSs, modules, components and bridges onto floating or grounded barges ans ships. Additional information can be found in ISO 19902:2007, Clauses 8 and 22. - 19901-6, Clause 11 applied particularly to skidded and trailer-transported floating loadouts in tidal waters. Recommendations for grounded loadouts or loadouts accomplished by lifting are also included.
	- Environmental Criteria (2.4.4.b) The selection of environemntal conditions to be used should consider the following: 1. Previous experience along the tow route 2. Exposure time and reliability of predicted "weather windows" 3. Accessibility of safe havens	- For long ocean tows where the structure and barge are unmanned, the extreme enviornmental 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	- ISO 19906-1, Clause 12 applies to offshore transportation, inshore transportation and transportation in sheltered areas, using either wet tow or dry tow. Additional information can be found in ISO 19902:2007, Clause 8 and 22, and ISO 19903:2006, Clause 11.

Transportation	4. Seasonal weather system 5. Appropriateness of the recurrence interval used to determine maximum design wind, wave and current conditions and consider the characteristics of the tow, such as size, structure, sensitivity, and cost **- Determination of Forces (2.4.4.c)** a) Beam, head and quartering wind and seas should be considered to determine maximum transportation forces in the tow structural elements. b) Tows may be analyzed based on gravitational and inertial forces resulting from the tow's rigid body motions using appropriate period and amplitude by combining roll with heave and pitch with heave. c) Submerged members should be investigated for slamming, buoyancy and collapse force. d) Large buoyant overhanging members also may affect motions and should be considered. e) The effects on long slender members of wind-induced vertex shedding vibrations should be investigated. f) For long transocean tows, repetitive member stresses may become significant to the fatigue life of certain member connections or details and should be investigated.	of not less than 1 year for season in which the tow takes place. **- Environmental Criteria (8.6.2)** The selection of environemntal conditions to be used should consider the following: 1. Previous experience along the tow route 2. Exposure time and reliablity of predicted "weather windows" 3. Accessibility of safe havens 4. Seasonal weather system 5. Appropriateness of the recurrence interval used to determine maximum design wind, wave and current conditions and consider the characteristics of the tow, such as size, structure, sensitivity, and cost. **- Determination of Forces (8.6.3)** a) Beam, head and quartering wind and seas should be considered to determine maximum transportation responses due to the environmental actions on the overall system. In case of large barge- transported structures, the stiffness of both the structures and the barge shall be included in the structural analysis. b) Tows may be analyzed based on a combination of permanent and inertia actions resulting from the tow's rigid body motions using appropriate period and amplitude by combining roll with heave and pitch with heave. c) Large buoyant overhanging members also may affect motions and should be considered. d) The effects on long slender members of wind-induced vertex shedding vibrations should be investigated.	
Launching Forces and Uprighting Forces	**- Guyed Tower and Template Type (2.4.5.a)** a) Forces supporting the jacket on the ways should be evaluated for the full travel of the jacket. b) Deflection of the rocker beam and the effect on loads throughout the jackets should be considered. C) Horizontal forces required to initiate movement of the jacket should be evaluated. d) Consideration should be given to wind, wave, current and dynamic forces expected on the structure and barge during launching and uprighting. **- Tower Type (2.4.5.b)** Forces should be evaluated for the full travel of the tower down the ways. **- Hook Load (2.4.5.c)** Floating jackets for which lifting equipment is employed for turning to a vertical position should be designed to resist the gravitational and internal forces required to upright the jacket.	**- Launched structures (8.7.2)** a) A structure shall not be lauched from a 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. 'b) Barge-launched structures shall be analysed to determine the actions on the structure throughout the launch. Consideration shall be given to hydrostatic pressure, wind and current actions, and the development of dynamically induced actions resulting from the launch. c) Horizontal actions required to initiate movement of the structure should also be evaluated. Expected actions o both the structure and the barge during launching should be considered. **- Crane assisted uprighting of structures (8.7.3)** The requirements of 8.3 apply to this situation.	- ISO 19901-1, Clause 9.9.3 and Clause 17.5
On-Bottom Stability	- The factors of safety against bearing capacity failure recommended are 2.0 for on bottom gravity loads alone and 1.5 for the design environmental condition applicable for the installation period. - At the operators 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. - In the event of rough seas or if the installation equipment must leave the site for other reasons before the jacket has been adequately secured with piles, the effective weight on bottom may require adjustment to minimize the possibility of jacket movement due to skidding, overturning, or soil failure.	The design shall ensure the followings: a) the footings or mudmats have adequate capacity against sliding and bearing failure, and that pin-piles, if any, have adequate strength to avoid being damaged b) the footings, mudmats, or other bearing components and structural members supporting these, have adequate strength to avoid being damaged c) the safety margins against overturning of the structure are adequate, with the recommendation that the structure be checked in a piled condition but without the permanent action of the topsides if placement of the topsides does not follow shortly after structure installation.	**NORSOK N-004, K.6.4** The foundation system for the jacket temporary on-bottom condition prior to installation of the permanent foundation system shall be documented to have the required foundation stability for the governing environmental conditions as specified, and for all relevant limit states.

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 10-1 Site Seismic Zone in ISO

$S_{a,\,map}(1.0)$	< 0.03g	0.03g to 0.10 g	0.11g to 0.25g	0.26g to 0.45g	> 0.45g
Seismic zone	0	1	2	3	4

$S_{a,\,map}$ (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 10-2 Seismic Zone In API

Z	0	1	2	3	4	5
G	0.00	0.05	0.10	0.20	0.25	0.40

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

G = 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 10-3 Site Class

Average properties in top 30m of effective seabed		
Site class (ISO)	**Soil profile name**	**Soil shear wave velocity, ν_s, m/s**
A/B	Hard rock/Rock, thickness of sediment < 5m	$\nu_s > 750$ (API Soil A)
C	Very dense hard soil and soft rock	$350 < \nu_s \leq 750$
D	Stiff to very stiff soil	$180 < \nu_s \leq 350$ (API Soil B)
E	Soft to firm soil	$120 < \nu_s \leq 180$ (API Soil C)
F	-	Any profile, including those otherwise classified as A to E.

10.2.5 Earthquake Response Spectrum

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

$T<0.05$ s	$S_a/G = 1.0$
0.05 sec $< T < 0.125$ s	$S_a/G = 20T$
API soil type A :	
0.125 sec $< T < 0.32$ s	$S_a/G = 2.5$
$T > 0.32$ s	$S_a/G = 0.8/T$
API soil type B :	
0.125 sec $< T < 0.48$ s	$S_a/G = 2.5$
$T > 0.48$ s	$S_a/G = 1.2/T$
API soil type C :	
0.125 sec $< T < 0.72$ s	$S_a/G = 2.5$
$T > 0.72$ s	$S_a/G = 1.8/T$

where G = effective horizontal ground acceleration

The response spectrum defined in ISO 19901-2 is:

$S_{a,site}(T) = (3T+0.4)(C_a)S_{a,map}(0.2)$ for $T \leq 0.2$s

$S_{a,site}(T) = C_v S_{a,map}(1.0)/T$ for $T > 0.2$s

except that

$S_{a,site}(T) \leq C_a S_{a,map}(0.2)$

$S_{a,site}(T) = 4C_v S_{a,map}(1.0)/T^2$ for $T > 4$ s

Where

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

C_a, C_v = site coefficients

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

$S_{a,map}(0.2)$ = 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

$S_{a,map}(1.0)$ = 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 $F_yD/E.t \leq 0.069$ 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

<u>ELE Requirements</u>

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:

$$F_d = 1.1G_1 + 1.1\ G_2 + 1.1Q_1 + 0.9\ E \qquad (10.1a)$$

$$\text{Or } F_d = 0.9G_1 + 0.9\ G_2 + 0.8Q_1 + 0.9\ E \qquad (10.1b)$$

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 + 1.3 E

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

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

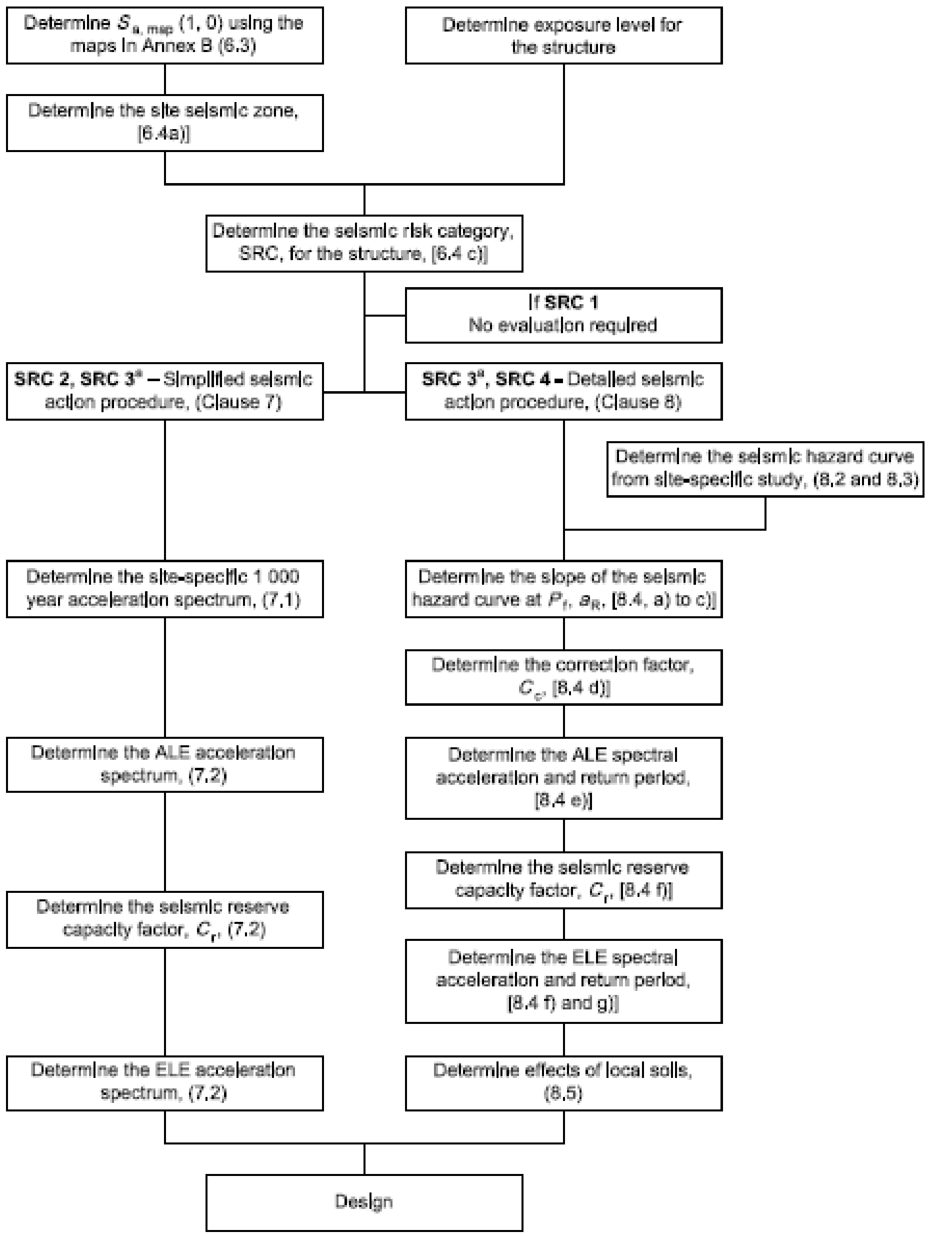
NORSOK also gives the guideline for action combination for ALS:

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

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.



[a] SRC 3 structures may be designed using either a simplified or detailed seismic action procedure, see Table 4.

Figure 10-1 Seismic design procedures in ISO 19901-2 (Figure 1of ISO 19901-2)

Table 10-4 Seismic Design Requirements (Table in ISO 19901-2)

SRC	Seismic action procedure	Evaluation of seismic activity	Non-linear ALE analysis
1	None	None	None
2	Simplified	ISO maps or regional maps	Permitted
3[a]	Simplified	Site-specific, ISO maps or regional maps	Recommended
	Detailed	Site-specific	Recommended
4	Detailed	Site-specific	Required

[a] For an SRC 3 structure, a simplified seismic action procedure is in most cases more conservative than a detailed seismic action procedure. For evaluation of seismic activity, results from a site-specific probabilistic seismic hazard analysis (PSHA), see 8.2, are preferred and should be used, if possible. Otherwise regional or ISO seismic maps may be used. A detailed seismic action procedure requires results from a PSHA whereas a simplified seismic action procedure may be used in conjunction with either PSHA results or seismic maps (regional or ISO maps).

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 10-5 Target annual probability of failure, p_f

Exposure Level	p_f
L1	$4 \times 10^{-4} = 1/2500$
L2	$1 \times 10^{-3} = 1/1000$
L3	$2.5 \times 10^{-3} = 1/400$

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

Site seismic zone	Exposure level		
	L3	L2	L1
0	SRC1	SRC1	SRC1
1	SRC2	SRC2	SRC3
2	SRC2	SRC2	SRC4
3	SRC2	SRC3	SRC4
4	SRC3	SRC4	SRC4

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)

Site class	$S_{a,map}(0,2)$				
	≤ 0,25 g	0,50 g	0,75 g	1,0 g	≥ 1,25 g
A/B	1,0	1,0	1,0	1,0	1,0
C	1,2	1,2	1,1	1,0	1,0
D	1,6	1,4	1,2	1,1	1,0
E	2,5	1,7	1,2	0,9	0,9
F	a	a	a	a	a

a A site-specific geotechnical investigation and dynamic site response analyses shall be performed.

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

Site class	$S_{a,map}(1,0)$				
	≤ 0,1 g	0,2 g	0,3 g	0,4 g	≥ 0,5 g
A/B	1,0	1,0	1,0	1,0	1,0
C	1,7	1,6	1,5	1,4	1,3
D	2,4	2,0	1,8	1,6	1,5
E	3,5	3,2	2,8	2,4	2,4
F	a	a	a	a	a

a A site-specific geotechnical investigation and dynamic site response analyses shall be performed.

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

Site class	C_a	C_v
A/B	1,0	0,8
C	1,0	1,0
D	1,0	1,2
E	1,0	1,8
F	a	a

a A site-specific geotechnical investigation and dynamic site response analyses shall be performed.

c) Determine site 1000-year horizontal acceleration spectrum $S_{a,site}$ (T) for different oscillator periods (T), see /2/.

d) The site vertical spectral acceleration at a period T shall be taken as half the corresponding horizontal spectral acceleration. The vertical spectrum shall not be reduced further due to water depth effects.

e) A modal damping corresponding to 5% of critical can be used to obtain the acceleration spectra. For other damping value, the ordinates may be scaled by applying a correction factor D:

$$D = \frac{\ln(\frac{100}{\eta})}{\ln(20)}$$ where η is the per cent of critical damping

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.

$$S_{a,ALE}(T) = NALE * S_{a,site}(T) \qquad (10.2)$$

Table 10-10 Scale factors for ALE spectra

Exposure Level	ALE scale factor
L3	0.85
L2	1.15
L1	1.60

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

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

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

To avoid return periods for the ELE that are too short, Cr 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 in normally performed by specialists using probabilistic seismic hazard analysis (PSHA) and/or with deterministic seismic hazard analysis (DSHA) as a complement to PHSA. 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, pf), and determine the site-specific spectral acceleration at P_f, $S_{a,pf}(T_{dom})$.
c) Determine the slope of the seismic hazard curve (α_R) in the region close to P_f by drawing a tangent line to the seismic hazard curve at P_f. The slope α_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

α_R	1.75	2.0	2.5	3.0	3.5
C_c	1.20	1.15	1.12	1.10	1.10

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

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

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

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

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

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

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

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

10.3.4 Seismic Analysis Methods

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

- Linear methods
  - i) Response spectrum analysis
  - ii) Time history method (modal analysis method, or direct time integration numerical analysis method)
- Non-linear methods
  - i) 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:

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

  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.

$$C_{dr} = \sqrt{1 + \frac{A_d}{F_u \Delta_u}} \qquad (10.8)$$

Where A_d is the area under the action-deformation curve starting from Δ_u and ending with Δ_{CAP}, the deformation capacity of the structure. Δ_{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

	API RP 2A			ISO 19901-2		
Earthquake Design	SLE (Strength Level Eqrthquake) DLE (Ductility Level Earthquake)			ELE (Extreme Level Eqrthquake) ALE (Abnormal Level Earthquake)		
Seismic Risk Map	Figure C2.3.6.1 - Seismic Risk of United States Continental Shelves			Worldwide Seismic Maps (Appendix B) - The return period selected for the development of the ground motion maps is 1000 year. - The maps give generic 5% damped spectral accelerations, expressed in g, for bedrock outcrop for a 1.0 s oscillator period and for a 0.2 s oscillator period respectively.		
Seismic Zones	Zones 0 0.0g 1 0.05g 2 0.10g 3 0.20g 4 0.25g 5 0.40g Based on 200-year earthquake			Zones Sa, map (1.0) 0 <0.03g 1 0.03g - 0.10g 2 0.11g - 0.25g 3 0.26g - 0.45g 4 >0.45g Sa,map(1.0) is the rock outcrop 1.0 second horizontal spectral acceleration corresponding to 1000-year earthquake.		
Foundation Soil	Soil Class	Soil Profile	Soil shear wave velocity, ft/sec	Soil class	Soil profile name	Soil shear wave velocity, v_s, m/s
	A	Rock - crystalline, conglomerate, or shale-like material	> 3000 ft/sec (914 m/sec)	A/B	Hard rock/Rock, thickness of sediments < 5 m	$v_s > 750$
				C	Very dense hard soil and soft rock	$350 < v_s \le 750$
	B	Shallow strong alluvium - component sands, silts and stiff clays with shear strengths in excess of about 1500psf (72 kPa), limited to depths of less than about 200 ft (61m), and overlying rock-like materials		D	Stiff to very stiff soil	$180 < v_s \le 350$
	C	Deep strong alluvium - components sands, silts and stiff clays with thickness in excess of about 200 ft (61m) and overlying rock-like materials		E	Soft to firm soil	$120 < v_s \le 180$
				F	-	Any profile, including those otherwise classified as A to E
Earthquake Directional Loads (Actions)	1.0:1.0 (two horizontal orthogonal dir.) and 0.5 (vertical), acted simultaneuously)			1.0:1.0 (two horizontal orthogonal dir.) and 0.5 (vertical), acted simultaneuously		
Earthquake Directional Combinations	root of the sum of the squares method(SRSS)			SRSS or 1 component 100%, and 40% of its maximum values in other 2 components combined linearly		
Time History Analysis	Minimum 3 sets of time history records			Minimum 4 sets of time history records		
Response Spectrum Shape	T≥4.0 seconds, Sa(T) proportional to 1/T			T≥4.0 seconds, Sa(T) proportional to $1/T^2$		
Structural Slenderness (DLE or ALE)	kl/r ≤ 80 (primary diagonal bracing)			kl/r ≤ 80 (primary diagonal bracing)		
Tubular D/t Ratio (DLE or ALE)	$D/t \le 1900/F_y$			$D/t \le 2000/F_y$		
Pile-Soil Performance for ELE	ϕ_{PE} = 0.80 (axial) (1/0.8 = 1.25)			Partial resistance factor - 1.25 (axial) Partial resistance factor - 1.00 (p-y curves)		
Pile-Soil Performance for ALE	ϕ_{PE} = 1.0 (axial)			Partial resistance factor - 1.00 (axial) Partial resistance factor - 1.00 (p-y curves)		
Pile Axial Capacity Requirements (General)	API-LRFD ϕ_{PE} = 0.80 (axial) (extreme conditions) (1/0.8 = 1.25) ϕ_{PO} = 0.70 (axial) (operating conditions) (1/0.7 = 1.429) API-WSD Factor of Safety = 1.50 (extreme conditions) Factor of Safety = 2.00 (operating conditions)			Partial resistance factor - 1.25 (extreme conditions) Partial resistance factor - 1.50 (operating conditions)		

	API RP 2A		ISO 19901-2		NORSOK N-003 & N-004
Provisions Purpose	1.The strength requirements are presented to provide resistance to moderate earthquake, which have a reasonable likelihood of not being exceeded during the life of the platform, without significant structural damage. 2. To prevent collapse of the platform in the event of rare intense earthquake ground motions.	Provisions 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 (abnormal level earthquake) requirements are intended to ensure that the structure and foundation have sufficient reserve strength, displacement and/or energy dissipation capacity to maintain the overall structural integrity and avoid structural collapse.	Provisions Purpose	1. ULS (strength) check of components based on earthquakes with an annual probability of occurrence of 10^{-2} and appropriate action and material factors; 2. ALS 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 3. This provisions mainly focus on Norwegian continental shelf.
Strength Requirement — Structural Modeling (section 2.3.6.c2)	1.The analysis model should include the three dimentsional distribution of platform stiffness and mass; 2. Earthquake loading should be combined with other simultaneous loadings such as gravity, buoyancy, and hydrostatic pressure; Gravity loading should include the platform dead weight (comprised of the weight of the structure, equipment, appurtenances), actual live loads and 75% of the maximum supply and storage loads. 3. In computing the dynamic characteristics of braced, pile supported steel strcutures, uniform modal damping ratios of 5% critical should be used for an elastic analysis.	Extreme Level Earthquake Design — Action Combinations	1. Design Action $F_d = 1.1G_1 + 1.1G_2 + 1.1Q_1 + 0.9E$ where E: the inertia action induced by the ELE ground-motion and determined using dynamic analysis procedures such as response spectrum or tme-history analysis G_1 and G_2: permanent actions; Q_1: variable action; and shall include actions that are likely to be present during earthquake. When contributions to the action effects due to weight oppose the inertia actions due to the earthquake, $F_d = 0.9G_1 + 0.9G_2 + 0.8Q_1 + 0.9E$ where G_1, G_2 and Q_1 shall include only actions that are reasonably certain to be present during an earthquake. 2. The mass used in the dynamic analysis: - the permanent actions G1 and G2 - 75% of the variable actions Q1 - the mass of entrapped water, and the added mass 3. A modal damping ratio of 5% of critical may be used in the dynamic analysis of the ELE event.	ULS — Action Combinations	1. The number of vibration modes in the analysis should represent at least 90% of the total response energy of all modes. 2. In the 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 for ULS: ULS (a): 1.3G + 1.3Q + 0.7E ULS (b): 1.0G + 1.0Q + 1.3E
Strength Requirement — Response Analysis (section 2.3.6.c3)	1. **Response spectrum method** - one design spectrum is applied equally in both horizontal directions. An acceleration spectrum of one-half that for the given zone should be applied in the vertical direction. The complete quadratic combination (CQC) method may be used for combining modal responses and the square root of the sum of the squares (SRSS) may be used for combining the directional response. At least two modes having the highest overall response should be included for each of the three principal directions plus significant torsional modes. 2. **Time history method** - the design response should be calculated as the average of the maximum values for each of the time histories considered.	Extreme Level Earthquake Design — Response Analysis	1. In both methods, the base excitations shall be composed of three motions, i.e. two orthogonal horizontal motions and the vertical motions. 2. **Response spectrum method** - When responses due to each directional component of an earthquake are calculated separately, the responses due to the three earthquake directions may be combined using the root of the sum of the squares method. Alternatively, 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. 3. **Time history method** - a minimum of 4 sets of time history records shall be used to capture the randomness in seismic motions. The ELE design is satisfactory if the code utilization maxima are less than 1.0 for half or more of the records; a scale factor of 1.05 shall be applied to the records if less than 7 sets of records are used.	ULS — Response Analysis	1.Earthquake motion can be described by two orthogonal horizontal motions and the vertical motion action 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 2/3 of the major component and vertical component equal to 2/3 of the major component, referred to bedrock. 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.
Strength Requirement — ...ction 2.3.6.c4 & 2.3.6e)	1. The structural members should not exceed yielding of the complete section or buckling. 2. For strength requirement, the basic AISC allowable stresses and those presented in Section 3.2 (Allowable Stresses for Cylindrical Members) may be increased by 70 percent. 3. For combined earthquake loading and hydrostatic pressure, the suggested safety factors for local buckling and interaction formula listed in Section 3.2 are as follows: Axial Tension 1.0 Axial Compression 1.0-1.2 Hoop Compression 1.2 4. Addtional Guidelines	Extreme Level Earthquake Design — ...mance	1. All primary, secondary structural and foundation components shall sustain little or no damage to the structure. Limited non-linear behaviour (e.g. yielding in steel) is permitted, but brittle degradation (e.g. local buckling in steel) shall be avoided. 2. The internal forces in joints shall stay below the joint strengths, using the calculated (elastic) forces and moments. 3. Masts, derricks and flare structures shall be capable of sustaining the motions transmitted via the structure with little or no damage. 4. For the design of piles for ELE event, a partial resistance factor of 1.25 shall be used to determine the axial pile capacity and a partial resistance factor of p-y curves of 1.0 shall be used to determine the lateral pile performance.	ULS — ...sessment	Material Factor γ_M =1.15

API RP 2A			ISO 19901-2			NORSOK N-003 & N-004		
	Response Assessment (se	a) Tubular joints are sized for the yield or buckling capacity of incoming members, so that premature failure of the joints will be avoided and the ductility of the overall structure can be fully developed. b) Joint capacity may be determined in accordance with Section 4.3 except that Equations 4.3-1, 4.3-2, and 4.3-3 should all have the safety factor (FS) set equal to 1.0. See Commentary for the influence of chord load and other detailed considerations. c) Deck-supported structures, and equipment tie-downs, should be designed with a **one-third increase** in basic allowable stresses. This lower increase in design allowables for strength level earthquake loads compared to a full yield stress allowable typically used for jackets is intended to provide a margin of safety in lieu of performing an explicit ductility level analysis.		ELE Perfo			Response As	
Ductility Requirements	Limitations (section 2.2.6.d2)	1. The intensity ratio of the rare, intense earthquake ground motions to strength level earthquake ground motions is 2 or less. 2. Systems are jacket type structures with 8 or more legs.	Abnormal Level Earthquake Design	Structural Modeling	1.Structural and foundation models shall include possible stiffness and strength degradation of componets under cyclic action reversals. 2. The ALS analysis shall be based on best estimate values of modelling parameters such as material strength, soil strength and soil stiffness. 3. A modal damping ratio of 5% of critical may be used in the dynamic analysis of the ALE event.	ALS	Structural Modeling	1. The number of vibration modes in the analysis should represent at least 90% of the total response energy of all modes. 2. In the 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 for ALS (abnormal effect). ALS (a): 1.0G + 1.0Q + 1.0E
	Desgin Practice (section 2.3.6.d2)	1. Jacket legs, including any enclosed piles, are designed to meet the requirements of 2.3.6c4, using twice the strength level seimic loads; 2. Diagonal bracing in the vertical frames are configured such that shear forces between horizontal frames or in vertical runs between legs are distributed approximately equally to both tension and compression diagonal braces, and that "K" bracing is not used where the ability of a panel to transmit shear is lost if the compression brace buckles. Where these conditions are not met, including areas such as the portal frame between the jacket and the deck, the structural components should be designed to meet the requirements of Section 2.3.6c4 using twice the strength level seismic loads. 3. Horizontal members are provided between all 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. 4. The slenderness ratio (Kl/r) of primary diagonal bracing in vertical frames is limited to 80 and their ratio of diameter to thickness is limited to 1900/Fy where Fy is in ksi (13100/Fy for Fy in MPa). 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 2.3.6c4 using twice the strength level seismic loads.		Response Analysis	1. In both methods, the base excitations shall be composed of three motions, i.e. two orthogonal horizontal motions and the vertical motions. 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. 3. A minimum of 4 sets of time history records shall be used to capture the randomness in seismic motions. If 7 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 demostrated in at least 4 time history analyses.		Response Analysis	1.Earthquake motion can be described by two orthogonal horizontal motions and the vertical motion action 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 2/3 of the major component and vertical component equal to 2/3 of the major component, referred to bedrock. 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.
	Structural Analysis	1. Structure-foundation systems which do not meet the conditions listed in 2.3.6d2 should be analyzed to demonstrate their ability to withstand the rare, intense earthquake without collapsing. 2. The time history method of analysis is recommended. 3. At least three sets of representative earthquake ground motion time histories should be analyzed.		ALE Performance	1. Structural elements are allowed to exhibit plastic degrading behaviour (e.g. local buckling in steel), but catastrophic failures such as global collapse or failure of a cantilevered section of the deck should be avoided. 2. Stable plastic mechanisms in foundations are allowed, but catastrophic failure modes sych as instability and collapse should be avoided. 3. 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.		Response Analysis	Material Factor γ_M =1.0

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.

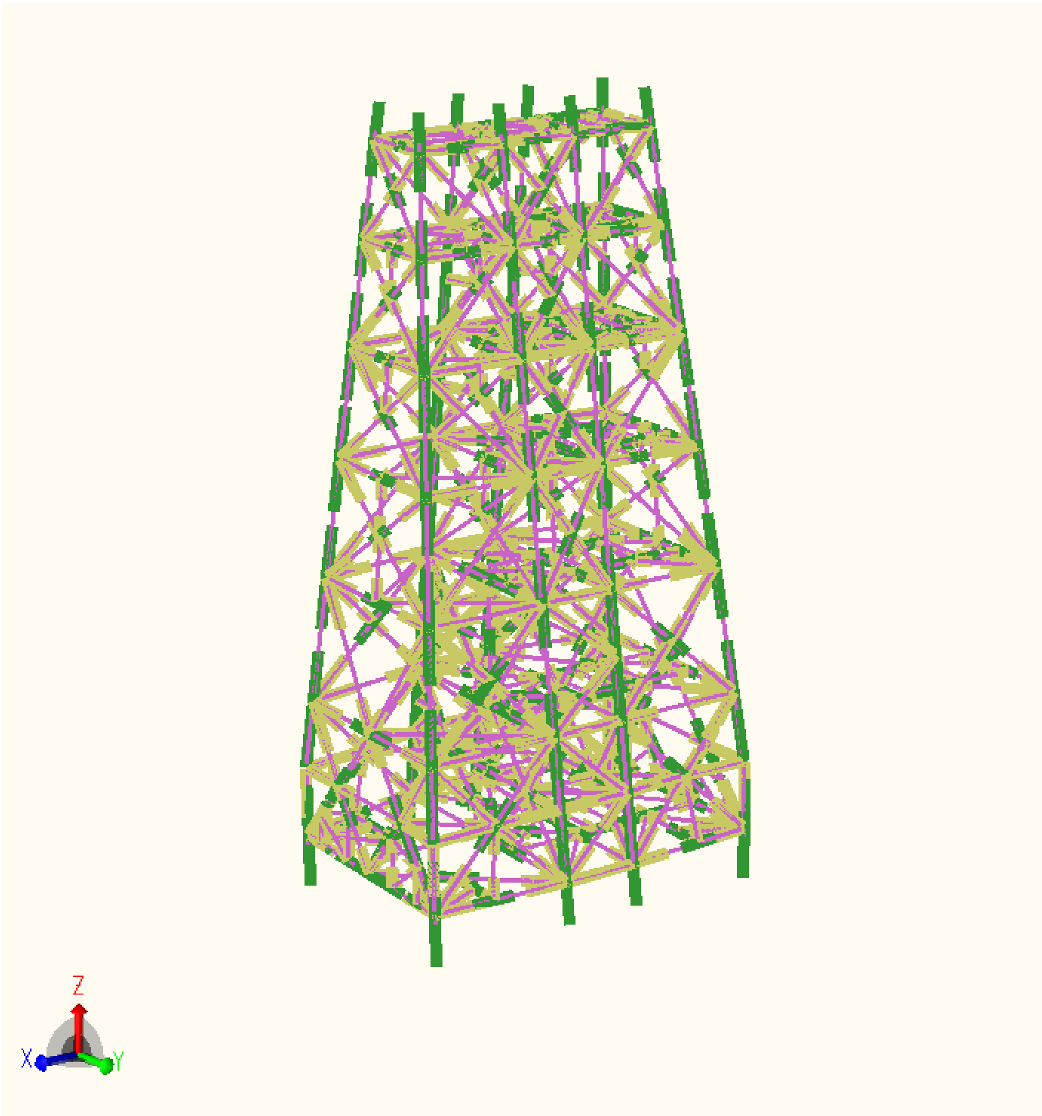


Figure 11-1 Capacity Model in GeniE

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

Layer	Soil type	From	To	Submerged Unit Weight	Undrained Shear Strength	Angle of Internal Friction	API-J Factor
	-	[m]	[m]	[kN/m3]	[kPa]	[deg]	-
1	clay	0	5.0	8	5 to 20		0.25
2	clay	5.0	22.0	10	80		0.25
3	sand	22.0	27.5	9.5		30	
4	clay	27.5	36.0	10	100		0.25
5	sand	36.0	54.0	9.5		35	
6	sand	54.0	100.9	10		37	
7	clay	100.9	110.5	11	290		0.25
8	sand	110.5	140.0	10.5		35	

The pile capacity evaluation was excluded from the scope Scope of work Work of this study, 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 WAJAC 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.

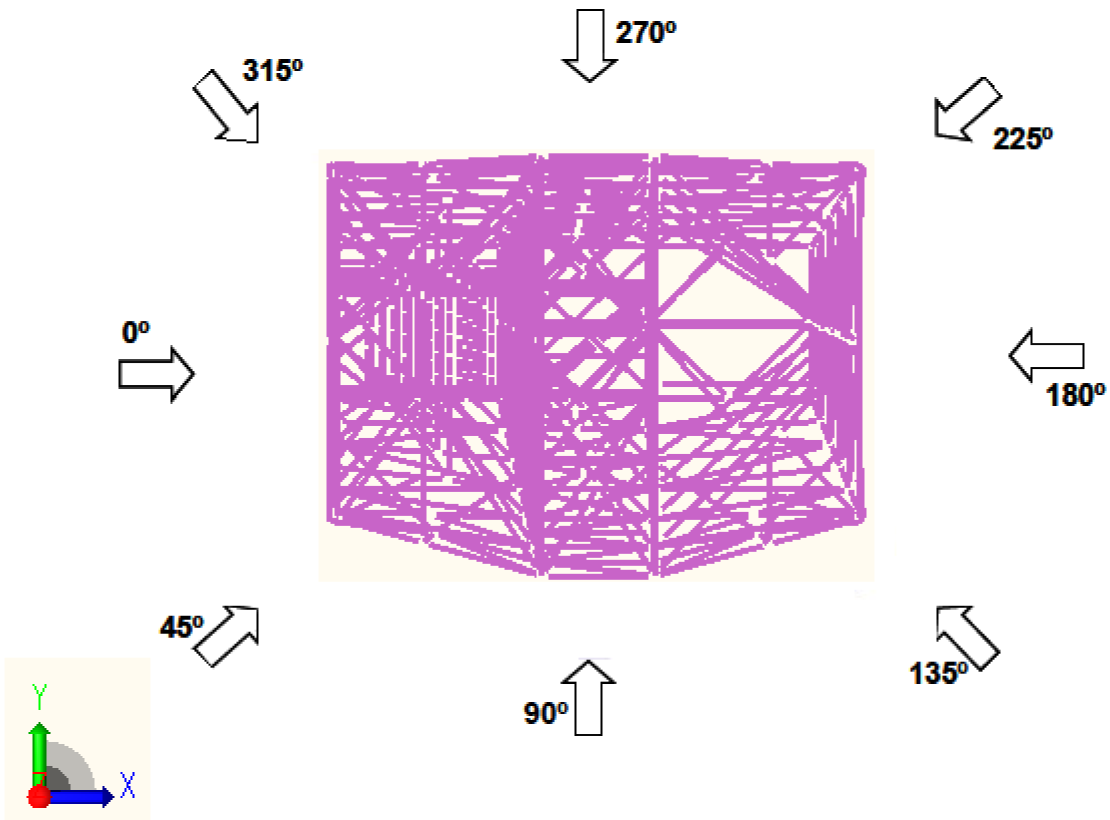


Figure 11-2 Analyzed Wave Headings

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

		API	ISO	NORSOK
Water Depth (Including Surge)	m	110	110	110
Max Wave Height [1]	m	26.0	24.3	26.0
Max Wave Period	s	13.9	13.2	13.9
Wind Direction (Relative to Wave)	deg	-15	0	0
Avg. Wind Speed	m/s	45.6	46.1	48
Current Direction (Relative to Wave)	deg	15	0	0
Current Speed [1]@ Surface	m/s	1.68	2.1	1.32
Current Speed [1]@ Middle of Profile (Elev.)	m/s	1.46 (32)	1.76 (35)	1.11 (35)
Current Speed [1]@ Bottom of Profile (Elev.)	m/s	0.00 (64)	0.09 (70)	0.00 (70)
Current Speed @ Mudline	m/s	0.00	0.09	0.00

[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

	Heading	Max Base Shear		Max Overturning Mom.[1]	
	deg	**MN**	**Phase**	**MNm**	**Phase**
API	**0**	70.5	340	6829	350
	45	67.7	350	6872	350
	90	71.2	350	7551	350
	135	67.8	350	7002	0
	180	67.8	0	6541	0
	225	66.8	350	6221	0
	270	71.4	350	6908	350
	315	70.0	350	6711	350
	MAX	**71.4**	-	**7551**	-
ISO	**0**	68.0	340	6707	340
	45	67.3	350	6982	350
	90	70.7	350	7634	350
	135	66.3	350	6794	0
	180	65.2	0	6403	0
	225	66.5	350	6322	0
	270	70.8	350	6990	350
	315	67.5	350	6516	350
	MAX	**70.8**	-	**7634**	
NORSOK	**0**	69.0	340	6725	350
	45	66.6	350	6849	350
	90	70.7	350	7592	350
	135	65.7	350	6660	0
	180	66.3	350	6438	0
	225	65.8	350	6183	0
	270	70.8	350	6948	350
	315	66.8	350	6382	350
	MAX	**70.8**	-	**7592**	-

[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 UFs 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 11-4 Maximum Member Utilization Results – Base Case (API)

	Member	API			Corresponding Utilization				Ratio of Total Utilization	
					ISO		NORSOK			
		UF [1]	Formula	LC	UF	Formula	UF	Formula	UF_{API}/UF_{ISO}	UF_{API}/UF_{Norsok}
1	513	1.00	3.3.4-3	8	0.93	13.2-31	0.80	6.15	1.08	1.25
2	505	0.99	3.3.4-3	1	0.93	13.2-31	0.79	6.15	1.06	1.25
3	1651	0.98	3.3.4-3	15	0.90	13.2-31	0.80	6.15	1.09	1.23
4	1622	0.97	3.3.4-3	13	0.90	13.2-31	0.80	6.15	1.08	1.21
5	96	0.89	3.3.3-1	2	0.81	13.2-31	0.62	6.15	1.10	1.44
6	342	0.77	3.3.3-1	15	0.72	13.2-31	0.52	6.15	1.07	1.48
7	2707	0.75	3.3.3-1	7	0.54	13.2-31	0.50	6.42	1.39	1.50
8	350	0.73	3.3.3-1	3	0.54	13.2-31	0.48	6.42	1.35	1.52
9	348	0.72	3.3.3-1	11	0.54	13.2-31	0.47	6.42	1.33	1.53
10	343	0.72	3.3.3-1	5	0.72	13.2-31	0.51	6.15	1.00	1.41
11	2708	0.72	3.3.3-1	11	0.54	13.2-31	0.47	6.42	1.33	1.53
12	351	0.71	3.3.3-1	7	0.54	13.2-31	0.48	6.42	1.31	1.48
13	3083	0.68	3.3.3-1	15	0.54	13.2-31	0.47	6.42	1.26	1.45
14	448	0.68	3.3.3-1	8	0.51	13.2-12	0.51	6.42	1.33	1.33
15	346	0.67	3.3.3-1	13	0.54	13.2-31	0.44	6.42	1.24	1.52

[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

	ISO			LC	NORSOK			Corresponding Utilization (API)	
	Member	**UF**	**Formula**		**Member**	**UF**	**Formula**	**UF [1]**	**Formula**
1	740	**1.46**	13.6-21	13	740	**1.36**	6.71	0.62	API Cone
2	462	**1.41**	13.6-21	13	462	**1.31**	6.71	0.60	API Cone
3	1690	**1.36**	13.6-21	5	1690	**1.26**	6.71	0.58	API Cone
4	461	**1.30**	13.6-21	5	461	**1.21**	6.71	0.56	API Cone
5	41	**1.28**	13.6-21	5	41	**1.16**	6.71	0.61	API Cone
6	36	**1.18**	13.6-21	13	36	**1.06**	6.71	0.56	API Cone
7	31	**1.14**	13.6-21	7	31	**1.04**	6.71	0.46	API Cone
8	749	**1.04**	13.6-21	15	749	0.96	6.71	0.61	API Cone
9	10	**1.02**	13.6-21	5	10	0.93	6.71	0.47	API Cone
10	21	**1.01**	13.6-21	3	21	0.93	6.71	0.38	API Cone
11	647	0.96	13.6-21	3	647	0.89	6.71	0.56	API Cone
12	646	0.95	13.6-21	8	646	0.88	6.71	0.57	API Cone
13	2	0.94	13.6-21	13	2	0.86	6.71	0.43	API Cone
14	505	0.93	13.6-31	1 / 12 [2]	748	0.85	6.71	0.99 / 0.59 [2]	3.3.4-3 / Cone [2]
15	513	0.93	13.6-31	1 / 13 [2]	1647	0.83	6.71	0.98 / 0.44 [2]	3.3.4-3 / Cone [2]

[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 utilizations (due to axial force and bending moment) are reported.

Table 11-6 Member Utilization Results – Chosen Members

Member			Utilization Factor								
			UF_{API}			UF_{ISO}			UF_{NORSOK}		
ID	Location	Type	TOTAL	Axial	Bending	TOTAL	Axial	Bending	TOTAL	Axial	Bending
1674	Top	Leg	**0.29**	0.26	0.03	**0.40**	0.33	0.07	**0.37**	0.30	0.07
1678		Brace	**0.56**	0.39	0.18	**0.71**	0.42	0.29	**0.70**	0.39	0.30
43	Middle	Leg	**0.47**	0.43	0.04	**0.50**	0.47	0.03	**0.48**	0.43	0.05
662		Brace	**0.64**	0.59	0.05	**0.73**	0.66	0.07	**0.69**	0.62	0.06
1286	Bottom	Leg	**0.49**	0.42	0.07	**0.61**	0.48	0.13	**0.58**	0.44	0.14
442		Brace	**0.41**	0.36	0.05	**0.49**	0.42	0.07	**0.49**	0.41	0.08

Table 11-7 Ratio of Total Utilization – Chosen Members

Member			Ratio of Total Utilization	
ID	Location	Type	UF_{API}/UF_{ISO}	UF_{API}/UF_{NORSOK}
1674	Top	Leg	0.74	0.79
1678		Brace	0.79	0.81
43	Middle	Leg	0.94	0.97
662		Brace	0.88	0.93
1286	Bottom	Leg	0.81	0.85
442		Brace	0.83	0.83

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 re-sizing 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 11-8 Maximum Joints Utilization Results – Base Case (API)

	Joint	API		Corresponding Utilization		Ratio of Total Utilization	
		UF_{API} [1]	LC	UF_{ISO}	UF_{NORSOK}	UF_{API}/UF_{ISO}	UF_{API}/UF_{Norsok}
1	33	**2.48**	13	**2.76**	**3.02**	0.90	0.82
2	321	**1.88**	10	**2.06**	**2.25**	0.91	0.84
3	373	**1.37**	2	**1.52**	**1.58**	0.90	0.87
4	335	1.00	15	**1.03**	**1.07**	0.97	0.93
5	341	0.91	13	0.98	**1.02**	0.93	0.89
6	324	0.88	10	0.96	0.97	0.92	0.91
7	339	0.83	11	0.84	0.87	0.98	0.95
8	35	0.68	5	0.78	0.8	0.88	0.86
9	320	0.68	2	0.75	0.78	0.90	0.87
10	16	0.63	13	0.79	0.82	0.80	0.77
11	298	0.61	15	0.81	0.84	0.75	0.73
12	319	0.58	2	0.69	0.72	0.84	0.80
13	366	0.56	3	0.75	0.78	0.74	0.71
14	188	0.56	1	0.68	0.72	0.82	0.77
15	117	0.56	15	0.65	0.67	0.86	0.83

[1]) 33% increase of the allowable stresses included

Table 11-9 Joint Utilization Results – Chosen Joints

Joint			Utilization Factor								
			UF_{API}			UF_{ISO}			UF_{NORSOK}		
ID	**Location**	**Elev. [m]**	**TOTAL**	**Axial**	**Bending**	**TOTAL**	**Axial**	**Bending**	**TOTAL**	**Axial**	**Bending**
22 @ Mem1629	Top	111	**0.29**	0.07	0.22	**0.29**	0.07	0.22	**0.30**	0.07	0.23
195 @ Mem728	Middle	72.3	**0.46**	0.44	0.02	**0.51**	0.49	0.02	**0.54**	0.52	0.02
298 @ Mem432	Bottom	2.7	**0.61**	0.55	0.07	**0.81**	0.75	0.06	**0.84**	0.77	0.07

Table 11-10 Ratio of Total Utilization – Chosen Joints

Member		Ratio of Total Utilization	
ID	**Location**	UF_{API}/UF_{ISO}	UF_{API}/UF_{NORSOK}
22 @ Mem1629	Top	1.02	0.98
195 @ Mem728	Middle	0.90	0.85
298 @ Mem432	Bottom	0.75	0.73

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.

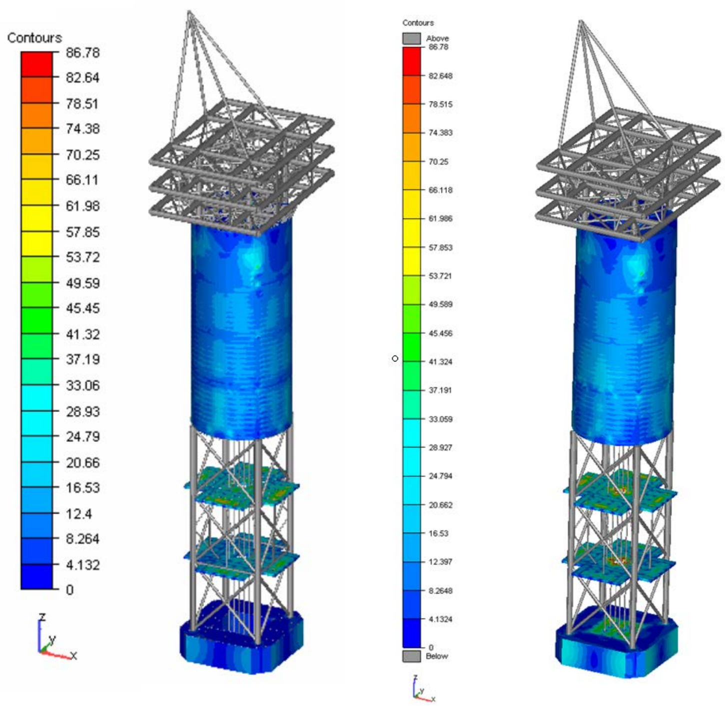


Figure 11-3 Comparison of the Designer's (left) and DNV's (right) Global Stress Results, (ksi, Nodal Von Mises Stress, 100 year Hurricane)

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 11-11 Design Waves for 100-year hurricane

Design Wave Case	Wave Amplitude [m]	Wave Period [s]
Max Shear	10.2	12
Max Moment	11.1	14
Max Axial	4.3	20

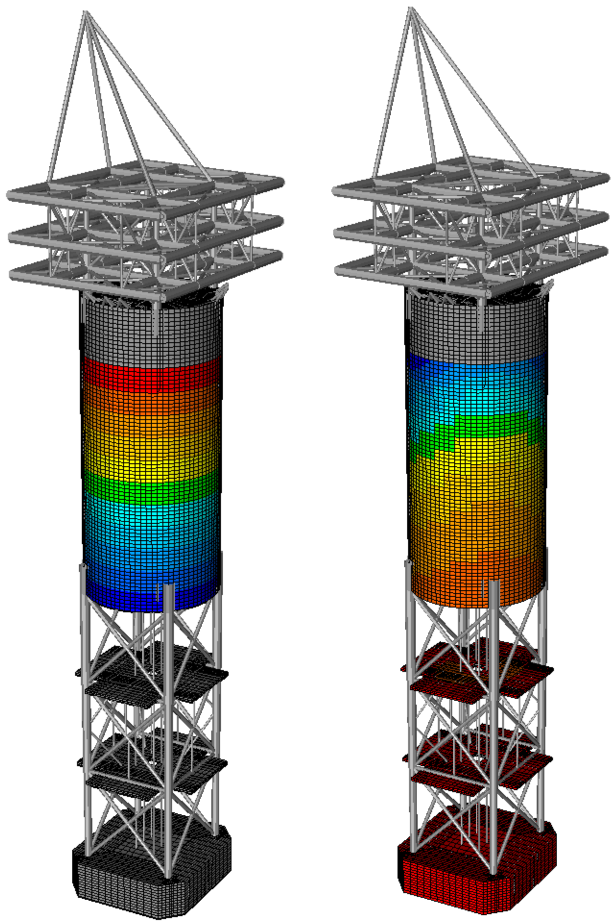


Figure 11-4 Static and Dynamic Pressure Distribution on the Structure

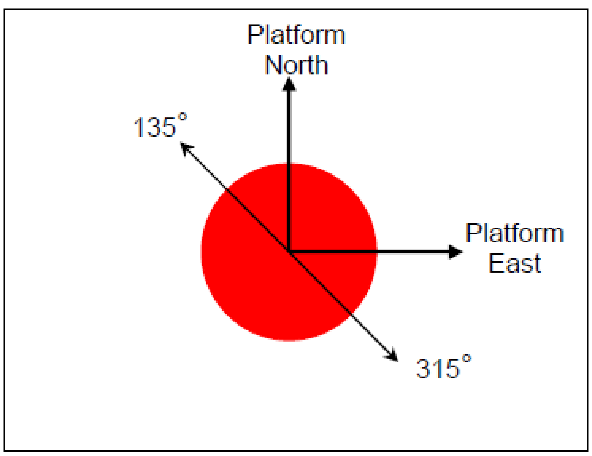


Figure 11-5 Analyzed Wave Headings

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/m^3 with a Poison's ratio of 0.3 and a Young's modulus of $2.1x10^5$ 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 6.6º 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 γ_M=1.15.

Table 11-12 Action Factors Applied to the ULS-b Limit State

Action Category		
Permanent (G)	**Variable (Q)**	**Environmental (E)**
1.0	1.0	1.3

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

	API	**ISO**
Usage Factor	$\frac{1}{1.67} \cdot 1.33$	0.8*

*) Based on DNV OS-C201

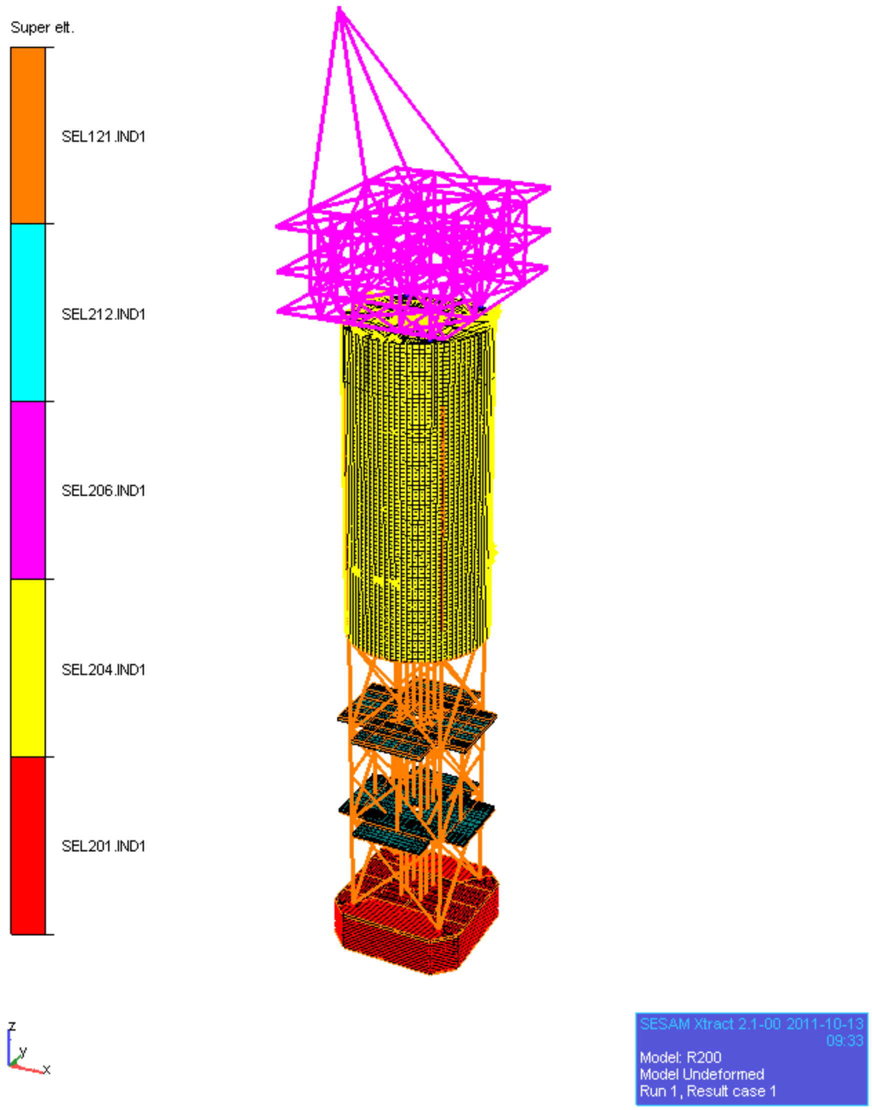


Figure 11-6 Structural Model Overview

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

Region	Panel	WSD (API & ISO)		LRFD – ULS-b (ISO & Norsok)		Ratio of Utilization	Stress Location	
		Stress [Mpa]	UF_{WSD} [-]	Stress [Mpa]	UF_{LRFD} [-]	UF_{WSD} / UF_{LRFD}	Elev. (ABL) [m]	Description
Deck 3-4	Radial Bhd	251	0.91	260	0.87	1.05	93.7	Intersection with OS
	Moonpool Bhd	205	0.74	212	0.71	1.04	88.7	Moonpool CL
	OS	577	2.09	683	2.28	0.92	108.0	Radial bulkhead @ Deck 4
Deck 4-5	Radial Bhd	380	1.38	442	1.47	0.94	108.7	Intersection with OS
	Moonpool Bhd	127	0.46	147	0.49	0.94	110.9	Moonpool corner
	OS	409	1.48	486	1.62	0.91	108.9	Radial bulkhead @ Deck 4
Deck 5-6	Radial Bhd	161	0.58	201	0.67	0.87	128.5	Intersection with OS
	Moonpool Bhd	88	0.32	101	0.34	0.94	138.4	Moonpool corner
	OS	170	0.62	209	0.70	0.89	128.5	Radial bulkhead @ Deck 5
Deck 6-7	Radial Bhd	411	1.49	485	1.62	0.92	154.7	Intersection with OS
	Moonpool Bhd	105	0.38	118	0.39	0.97	168.7	Moonpool CL
	OS	408	1.48	477	1.59	0.93	154.7	Radial bulkhead

Table 11-15 Von Mises Stress for the Centre of Analyzed Panels

Region	Panel	WSD (API & ISO)		LRFD – ULS-b (ISO & Norsok)		Ratio of Utilization	Elev. (ABL)
		Stress [Mpa]	UF_{WSD} [-]	Stress [Mpa]	UF_{LRFD} [-]	UF_{WSD} / UF_{LRFD}	
Deck 3-4	Radial Bhd	155.1	0.56	164	0.55	1.02	101.0
	Moonpool Bhd	114.4	0.41	119	0.40	1.03	
	OS	126.2	0.46	133.9	0.45	1.02	
Deck 4-5	Radial Bhd	107.1	0.39	126.9	0.42	0.93	118.8
	Moonpool Bhd	77.3	0.28	82.7	0.28	1.00	
	OS	140.1	0.51	155.7	0.52	0.98	
Deck 5-6	Radial Bhd	101.1	0.37	120.6	0.40	0.93	138.5
	Moonpool Bhd	74.5	0.27	85	0.28	0.96	
	OS	103.4	0.37	119	0.40	0.93	
Deck 6-7	Radial Bhd	102.7	0.37	110.8	0.37	1.00	161.2
	Moonpool Bhd	71.4	0.26	90.1	0.30	0.87	
	OS	53.6	0.19	69	0.23	0.83	

Where:

$$UF_{WSD} = \frac{\sigma_{V-M}}{\left(\frac{\sigma_Y}{1.67}\right)\cdot 1.33} \qquad (11.1a)$$

$$UF_{LRFD} = \frac{\sigma_{V-M}}{\left(\frac{\sigma_Y}{1.15}\right)} \qquad (11.1b)$$

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

DNV-RP-C201 Oct. 2010		Plate			Stiffener		Stresses				Unity Check	
Region	Panel	Breadth S (m)	Length L (m)	Thickness T (mm)	Type	Spacing s (m)	X σ_X (MPa)	Y σ_Y (MPa)	Shear τ_{XY} (MPa)	Pressure P_{HYD} (MPa)	Utilization η_{PLATE} (–)	Utilization η_{STIFF} (–)
Deck 3-4	Radial blk	8.05	1.98	21	HP370x 13	0.75	-55	-122	65	0.000	0.56	0.63
	Moonpool blk	14.02	1.98	24	HP340x 12	0.79	-40	-115	30	0.504	0.45	0.60
	Outer shell	28.21	1.98	23	HP400x 14	0.76	-55	-98	70	0.543	0.54	0.64
Deck 4-5	Radial blk	8.05	1.98	20	HP370x 13	0.75	-48	-115	51	0.000	0.48	0.56
	Moonpool blk	14.02	1.98	21	HP340x 12	0.79	-37	-92	22	0.321	0.37	0.48
	Outer shell	28.21	1.98	21	HP340x 12	0.76	-60	-142	28	0.379	0.48	0.71
Deck 5-6	Radial blk	8.05	1.98	15	HP280x 12	0.75	-39	-99	39	0.000	0.40	0.55
	Moonpool blk	14.02	1.98	16	HP260x 10	0.79	-30	-79	19	0.119	0.28	0.48
	Outer shell	28.21	1.98	20	HP300x 12	0.76	-48	-99	16	0.185	0.33	0.50
Deck 6-7	Radial blk	8.05	2.13	15	HP280x 12	0.75	-14	-75	55	0.000	0.43	0.46
	Moonpool blk	14.02	2.13	15	HP260x 10	0.79	-15	-26	23	0.000	0.17	0.16
	Outer shell	28.2	2.13	20	HP280x 12	0.76	-15	-75	67	0.000	0.49	0.48

Table 11-17 Results of the Buckling Check – ISO/NORSOK LRFD Run

DNV-RP-C201		Plate			Stiffener		Stresses				Unity Check	
Oct. 2010		Breadth	Length	Thickness	Type	Spacing	X	Y	Shear	Pressure	Utilization	Utilization
Region	Panel	S	L	T		s	σ_X	σ_Y	τ_{XY}	P_{HYD}	η_{PLATE}	η_{STIFF}
		m	m	mm		m	MPa	MPa	MPa	MPa	–	–
Deck 3-4	Radial blk	8.05	1.98	21	HP370x 13	0.75	-62	-141	73	0.000	0.51	0.71
	Moonpool blk	14.02	1.98	24	HP340x 12	0.79	-48	-123	31	0.504	0.42	0.61
	Outer shell	28.21	1.98	23	HP400x 14	0.76	-59	-129	68	0.557	0.51	0.71
Deck 4-5	Radial blk	8.05	1.98	20	HP370x 13	0.75	-54	-128	45	0.000	0.39	0.56
	Moonpool blk	14.02	1.98	21	HP340x 12	0.79	-44	-96	24	0.321	0.34	0.47
	Outer shell	28.21	1.98	21	HP340x 12	0.76	-62	-157	30	0.398	0.43	0.72
Deck 5-6	Radial blk	8.05	1.98	15	HP280x 12	0.75	-45	-106	50	0.000	0.37	0.58
	Moonpool blk	14.02	1.98	16	HP260x 10	0.79	-35	-86	26	0.119	0.25	0.49
	Outer shell	28.21	1.98	20	HP300x 12	0.76	-63	-24	17	0.205	0.32	0.57
Deck 6-7	Radial blk	8.05	2.13	15	HP280x 12	0.75	-16	-59	64	0.000	0.36	0.41
	Moonpool blk	14.02	2.13	15	HP260x 10	0.79	-15	-24	40	0.000	0.21	0.23
	Outer shell	28.2	2.13	20	HP280x 12	0.76	-8	-74	50	0.000	0.32	0.37

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 utilizations 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

/1/ Gerhard Ersdal (2005). “Assessment of Existing Offshore Structures for Life Extension”. Doctorial Thesis, University of Stavanger.

/2/ B. Chang and B.-F. Peng (2005). “Comparison of ISO and API Seismic Design Guidelines Using 3 Existing Offshore Platforms”. OTC paper 17285.

/3/ N.S. Potty, Z. Nizamani and A.B. Idrus (2010). “Strength of Tubular Members – Numerical Comparison of API RP 2A to ISO Codes”. Proceedings of the Ninth (2010) ISOPE Pacific/Asia Offshore Mechanics Symposium

/4/ MSL Engineering Limited for the Health and Safety Executive (2000). “Load Factor Calibration for ISO 13819 Regional Annex: Component Resistance”. Technology Report 2000/072

/5/ Bomel Limited for the Health and Safety Executive (2001). “Comparison of Fatigue Provisions in Codes and Standards”. Offshore Technology Report 2001/083

/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”.

/7/ API RP 2A (WSD) - Recommended Practice for Planning, Designing and Constructing Fixed Offshore Platforms – Working Stress Design, 21st Edition, December, 2000, with Supplements in December, 2002, September, 2005, and October 2007.

/8/ API RP 2FB - Recommended Practice for the Design of Offshore Facilities Against Fire and Blast Loading, 1st Edition, April, 2006.

/9/ API RP 2T - Planning, Designing, and Construction Tension Leg Platforms, third edition, July 2010.

/10/ API RP 2FPS - Recommended Practice for Planning, Designing, and Constructing Floating Production Systems, 1st edition, March 2001

/11/ API Bulletin 2INT-MET - Interim Guidance on Hurricane Conditions in the Gulf of Mexico, May 2007

/12/ API RP 2A (LRFD) - Recommended Practice for Planning, Designing and Constructing Fixed Offshore Platforms – Load and Resistance Factor Design, 1st edition reaffirmed, May 2003

/13/ ISO 19900 – Petroleum and natural gas industries - General Requirements for Offshore Structures, 1st edition, December 2002

/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

/15/ ISO 19901-2 - Petroleum and natural gas industries - Specific requirements for offshore structures – Part 2: Seismic Design Procedures and Criteria, 1st edition, November 2004

/16/ ISO 19901-6 - Petroleum and natural gas industries - Specific requirements for offshore structures – Part 6: Marine Operations, 1st edition, December 2009

/17/ ISO 19902 - Petroleum and natural gas industries - Fixed Steel Offshore Structures, 1st edition, December 2007

/18/ ISO 19904-1- Petroleum and natural gas industries - Floating offshore structures – Part 1: Monohulls, Semi-submersibles and Spars, 1st edition, November 2006

/19/ NORSOK Standard N-001 - Integrity of Offshore Structures, 7th edition, June 2010

/20/ NORSOK Standard N-003 - Action and Action Effects, 2nd edition, September 2007

/21/ NORSOK Standard N-004 - Design of Steel Structures, 2nd edition, October 2004

/22/ NORSOK Standard N-006 - Assessment of Structure Integrity for Existing Offshore Load-bearing Structures, 1st edition, March 2009

/23/ http://www.texaspelagics.com/GOMocean.html

/24/ DNV-RP-C203, "Fatigue Design of Offshore Steel Structures", April 2010.

/25/ DNV-RP-C206, "Fatigue Methodology of Offshore Ships", October 2010.

/26/ DNV CN 30.7, "Fatigue Assessment Of Ship Structures", June 2010.

/27/ NORSOK Standard N-005 – Condition Monitoring of Loadbearing Structures, Rev. 1, December, 1997.

/28/ DNV-RP-C201, "Buckling Strength of Plated Structures", October 2010

/29/ DNV-OS-C101, "Design of Offshore Structures, General (LRFD Method)", April 2011

/30/ Ghoneim, G.A., "Recent Developments in Offshore Codes, Rules and Regulations for E&P Systems", ISOPE-CCC-01, Vancouver, 2008.

/31/ Ghoneim, G.A., "The Balance between Risk and Reliability for Gulf of Mexico Production Systems", Deep Offshore Technology Conference, Houston, Texas, 2006.

/32/ API Bulletin 2INT-DG - Interim Guidance for Design of Offshore Structures for Hurricane Conditions, May 2007

/33/ API Bulletin 2INT-EX - Interim Guidance for Assessment of Existing Offshore Structures for Hurricane Conditions, May 2007

APPENDIX A: MATHCAD UNITY CHECK SHEETS

A.1 MATHCAD - API RP 2A

API RP 2A
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/mm^2)
Young's Modulus, E (N/mm^2)
Allowable tension stress (N/mm^2)
Allowable bending Stress F_b (N/mm^2)
Tension stress due to actual loads, f_t (N/mm^2)
Axial compression stress due to actual loads f_a (N/mm^2)
Bending stress due to actual loads about y axis, f_{by} (N/mm^2)
Bending stress due to actual loads about z axis, f_{bz} (N/mm^2)
Hoop stress due to actual loads, f_h (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)

INPUT DATA

$D := 1066.8$ $\qquad$ $D = 1.067 \times 10^3$

$t := 17.8$ $\qquad$ $t = 17.8$ $\qquad$ $\frac{D}{t} = 59.933$

$F_y := 345$ $\qquad$ $F_y = 345$

$E := 210000$ $\qquad$ $E = 2.1 \times 10^5$

$f_t := 0$ $\qquad$ $f_t = 0$

$f_a := 29$

$f_{by} := 3.93734375$ $\qquad$ $f_{by} = 3.937$

$f_{bz} := 11.016950$ $\qquad$ $f_{bz} = 11.017$

$f_h := 29.462564$ $\qquad$ $f_h = 29.463$

$l := 16178$ $\qquad$ $l = 1.618 \times 10^4$

$k := 0.8$

$v := 27922.17$ $\qquad$ $v = 2.792 \times 10^4$

$$I := \pi \cdot \frac{\left[D^4 - (D - 2 \cdot t)^4\right]}{64} \qquad I = 8.071 \times 10^9$$

$$area := \frac{\pi}{4} \cdot \left[D^2 - (D - 2 \cdot t)^2\right] \qquad area = 5.866 \times 10^4$$

$$r := \sqrt{\frac{I}{area}} \qquad r = 370.931$$

$$\frac{k \cdot l}{r} = 34.892$$

3.2.1 - Axial tension, , f_t <= F_y

$$F_t := 0.60 \cdot F_y \qquad F_t = 207$$

$$UR_{3.2.1\_1} := \frac{f_t}{F_t} \qquad UR_{3.2.1\_1} = 0$$

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:

$$C_c := \left(2 \cdot \pi^2 \cdot \frac{E}{F_y}\right)^{\frac{1}{2}} \qquad C_c = 109.614$$

$$F_a := \begin{cases} \dfrac{\left[1 - \dfrac{\left(\frac{k \cdot l}{r}\right)^2}{2 \cdot C_c^{\,2}}\right] \cdot F_y}{\dfrac{5}{3} + \dfrac{3\left(\frac{k \cdot l}{r}\right)}{8 \cdot C_c} - \dfrac{\left(\frac{k \cdot l}{r}\right)^3}{8 \cdot C_c^{\,3}}} & \text{if } \dfrac{k \cdot l}{r} < C_c \qquad (3.2.2 - 1) \\[2ex] \dfrac{12 \cdot \pi^2 \cdot E}{23 \cdot \left(\frac{k \cdot l}{r}\right)^2} & \text{if } \dfrac{k \cdot l}{r} \geq C_c \qquad (3.2.2 - 2) \end{cases}$$

$$F_a = 183.794$$

3.2.2.b Local Buckling

1. Elastic Load Buckling Stress

$$C_{crt} := 0.3$$

$$F_{xe} := 2 \cdot C_{crt} \cdot E \cdot \frac{t}{D} \qquad (3.2.2-3)$$

$$F_{xe} = 2.102 \times 10^3$$

2. Inelastic Local Buckling Stress

$$F_{xc} := \begin{vmatrix} F_y \cdot \left[1.64 - 0.23\left(\frac{D}{t}\right)^{\frac{1}{4}}\right] \\ F_y \ \text{ if } \left(\frac{D}{t}\right) \le 60 \end{vmatrix}$$

equal or less than F_{xe}

(3.2.2 − 4)

$$F_{xc} = 345$$

3.2.3 Bending

$$F_b := \begin{vmatrix} (0.75F_y) \ \text{ if } \frac{D}{t} \le \frac{10340}{F_y} & (3.2.3-1a) \\ \left[\left(0.84 - 1.74F_y \frac{D}{E \cdot t}\right)F_y\right] \ \text{ if } \frac{10340}{F_y} < \frac{D}{t} \le \frac{20680}{F_y} & (3.2.3-1b) \\ \left[\left(0.72 - 0.58 \cdot F_y \cdot \frac{D}{E \cdot t}\right)F_y\right] \ \text{ if } \frac{20680}{F_y} < \frac{D}{t} \le 300 & (3.2.3-1c) \end{vmatrix}$$

$$F_b = 230.694$$

Combined bending stress is given by, $f_b = (f_{by}^2 + f_{bz}^2)^{0.5}$

$$f_b := \left(f_{by}^2 + f_{bz}^2\right)^{0.5} \qquad f_b = 11.699$$

$$UR_{3.2.3} := \frac{f_b}{F_b} \qquad UR_{3.2.3} = 0.051$$

3.2.4 Shear

3.2.4.a Beam Shear

The maximum beam shear stress, f_v , for cylindrical member is:

$$f_v := \frac{v}{0.5 \cdot area} \qquad f_v = 0.952 \qquad (3.2.4-1)$$

The allowable beam shear stress:

$$F_v := 0.4 \cdot F_y \qquad F_v = 138 \qquad (3.2.4-2)$$

$$UR_{3.2.4.a} := \frac{f_v}{F_v} \qquad UR_{3.2.4.a} = 6.899 \times 10^{-3}$$

3.2.4.b Torsional Shear

Input the following data:

$M_t := 32827.6$ $\qquad M_t = 3.283 \times 10^4$ Torsional Moment

$I_p := \frac{\pi}{2} \cdot \left[\left(\frac{D}{2} \right)^4 - \left[\frac{(D - 2 \cdot t)}{2} \right]^4 \right]$ $\qquad I_p = 1.614 \times 10^{10}$ polar moment of inertia

The maximum torsional shear stress, F_v , for cylindrical members caused by torsion is:

$$f_{vt} := M_t \cdot \frac{\left(\frac{D}{2}\right)}{I_p} \qquad (3.2.4-3)$$

The allowable torsional shear stress:

$$F_{vt} := 0.4 \cdot F_y \qquad F_{vt} = 138 \qquad (3.2.4-4)$$

$$UR_{3.2.4.b} := \frac{f_{vt}}{F_{vt}} \qquad UR_{3.2.4.b} = 7.861 \times 10^{-6}$$

3.2.5 Hydrostatic Pressure (Stiffened and Unstiffened Cylinders)

3.2.5.a Design Hydrostatic Head

Input the following data:

$\rho := 1025$ density of the sea water which may be taken as 1025 kg/m^3

$g := 9.810$ the acceleration due to gravity (m/s^2)

$d := 110.000$ the still water depth to the sea floor (m)

$H_w := 26$ wave height (m)

$T_p := 13.9$ wave period (s)

$z := -95.38$ the depth of the member relative to still water level (measured positive upwards) (m)

$L_{wave} := \frac{g \cdot T_p^{\,2}}{2 \cdot \pi}$ $L_{wave} = 301.661$

$$H_z := -z + \frac{H_w}{2} \cdot \left[\frac{\cosh\left[2 \cdot \frac{\pi}{L_{wave}} (d + z)\right]}{\cosh\left(2 \cdot \frac{\pi}{L_{wave}} d\right)} \right] \qquad (3.2.5-3)$$

$H_z = 98.105$

Hydrostatic Pressure (N/mm²):

$p_0 := \frac{\rho \cdot g \cdot H_z}{1000000}$ $p_0 = 0.986$

$f_{hp} := p_0 \cdot \frac{D}{2 \cdot t}$ $f_{hp} = 29.561$

3.2.5.b Hoop Buckling Stress

1. Elastic Hoop Buckling Stress

Length of cylinder between stiffening rings, diaphragms, or end connections:

$L_{stiff} := 16178$ $L_{stiff} = 1.618 \times 10^4$ (Input)

The geomtric parameter, M, is defined as:

$M := \frac{L_{stiff}}{D} \cdot \left(2 \cdot \frac{D}{t}\right)^{\frac{1}{2}}$ $M = 166.031$

The critical hoop buckling coefficient C_h includes the effect of initial geometric imperfections within API Spec 2B tolerance limits.

$$C_h := \begin{cases} 0.44 \cdot \frac{t}{D} & \text{if } M \geq 1.6 \cdot \frac{D}{t} \\ 0.44 \cdot \frac{t}{D} + 0.21 \cdot \frac{\left(\frac{D}{t}\right)^3}{M^4} & \text{if } 0.825 \cdot \frac{D}{t} \leq M < 1.6 \cdot \frac{D}{t} \\ [0.736 \cdot (M - 0.636)] & \text{if } 3.5 \leq M \leq 0.825 \cdot \frac{D}{t} \\ [0.755 \cdot (M - 0.559)] & \text{if } 1.5 \leq M < 3.5 \\ 0.8 & \text{if } M < 1.5 \end{cases}$$

$$C_h = 7.342 \times 10^{-3}$$

Elastic hoop buckling stress:

$$F_{he} := 2 \cdot C_h \cdot E \cdot \frac{t}{D} \qquad F_{he} = 51.449$$

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 approriate formula.

Elastic buckling (3.2.5-6)

$$F_{hc} := \begin{cases} F_{he} & \text{if } F_{he} \leq 0.55 \cdot F_y \\ (0.45 \cdot F_y + 0.18 \cdot F_{he}) & \text{if } 0.55F_y < F_{he} \leq 1.6 \cdot F_y \\ \frac{1.31 \cdot F_y}{1.15 + \left(\frac{F_y}{F_{he}}\right)} & \text{if } 1.6 \cdot F_y < F_{he} < 6.2 \cdot F_y \\ F_y & \text{if } F_{he} > 6.2 \cdot F_y \end{cases}$$

Elastic Buckling

Inelastic Buckling

$$F_{hc} = 51.449$$

$$F_{hp} := \min(F_{hc}, F_{he}) \qquad F_{hp} = 51.449$$

$$UR_{3.2.5} := \frac{f_{hp}}{F_{hp}} \qquad UR_{3.2.5} = 0.575$$

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

$f_a = 29$ Compression Stress due to axial loads

$C_{mx} := 0.85$ based on Section 3.3.1.d

$C_{my} := 0.85$ based on Section 3.3.1.d

$k = 0.8$

$l_{by} := 1$ Actual unbraced length in the plane of bending

$l_{bz} := 1$ Actual unbraced length in the plane of bending

$r_y := r$ Corresponding radius of gyration

$r_z := r$ Corresponding radius of gyration

$$F_{ey} := \frac{12 \cdot \pi^2 \cdot E}{23 \cdot \left(k \cdot \frac{l_{by}}{r_y} \right)^2}$$ Euler stress divided by a factor of Safety

$$F_{ez} := \frac{12 \cdot \pi^2 \cdot E}{23 \cdot \left(k \cdot \frac{l_{bz}}{r_z} \right)^2}$$ Euler stress divided by a factor of Safety

$F_{ey} = 888.237$ $F_{ez} = 888.237$

$$UR_{3.3.1a} := \left| \begin{array}{ll} \left(\frac{f_a}{F_a} + \frac{\sqrt{f_{by}^2 + f_{bz}^2}}{F_b} \right) \text{ if } \frac{f_a}{F_a} \le 0.15 & (3.3.1-3) \\ \frac{f_a}{F_a} + \frac{\sqrt{\left(\frac{C_{mx} \cdot f_{by}}{1 - \frac{f_a}{F_{ey}}} \right)^2 + \left(\frac{C_{my} \cdot f_{bz}}{1 - \frac{f_a}{F_{ez}}} \right)^2}}{F_b} & (3.3.1-4) \end{array} \right.$$

$UR_{3.3.1a} = 0.202$

3.3.2 Combined Axial Tension and Bending

$$UR_{3.3.2} := \frac{f_a}{0.6 \cdot F_y} + \frac{\sqrt{f_{by}^2 + f_{bz}^2}}{F_b} \qquad UR_{3.3.2} = 0.191$$

3.3.3 Axial Tension and Hydrostatic Pressure

$SF_x := 1.67$ safety factor for axial tension (see 3.3.5)

$SF_h := 2$ safety factor for axial tension (see 3.3.5)

$f_b = 11.699$ abosulte value of acting resultant bending stress

$f_h = 29.463$ abosulte value of acting resultant Compression stress

$\nu := 0.3$ Poissons ratio

$$AA := \frac{f_a + f_b - 0.5 \cdot f_h}{F_y} \cdot SF_x$$

"A" should reflect the maximum tensile stress combination

$AA = 0.126$

$$BB := \frac{f_h}{F_{hc}} \cdot SF_h \qquad BB = 1.145$$

$$UR_{3.3.3.1} := \left| \begin{array}{l} 0 \\ \left(AA^2 + BB^2 + 2 \cdot \nu \cdot |AA| \cdot BB\right) \ \text{if} \ f_t > 0 \end{array} \right. \qquad (3.3.3-1)$$

$UR_{3.3.3.1} = 0$

3.3.4 Axial Compression and Hydrostatic Pressure

$f_x := f_a + f_b + 0.5 \cdot f_h$ should reflect the maximum compressive stress combination

$f_x = 55.431 \qquad f_a = 29 \qquad f_b = 11.699 \qquad f_h = 29.463$

$$F_{aa} := \frac{F_{xe}}{SF_x} \qquad F_{aa} = 1.259 \times 10^3$$

$$F_{ha} := \frac{F_{he}}{SF_h} \qquad F_{ha} = 25.724$$

$$SF_b := \frac{F_y}{F_b} \qquad SF_b = 1.495$$

$$UR_{3.3.4.1} := \frac{f_a + 0.5 \cdot f_h}{F_{xc}} \cdot SF_x + \frac{f_b}{F_y} \cdot SF_b \qquad UR_{3.3.4.1} = 0.262$$

$$UR_{3.3.4.2} := \frac{SF_h \cdot f_h}{F_{hc}} \qquad UR_{3.3.4.2} = 1.145$$

$$UR_{3.3.4.3} := \frac{f_x - 0.5 \cdot F_{ha}}{F_{aa} - 0.5 \cdot F_{ha}} + \left(\frac{f_h}{F_{ha}}\right)^2 \qquad UR_{3.3.4.3} = 1.346 \qquad \text{when } f_x > 0.5F_{ha}$$

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

$0.2 \le \beta \le 1.0$

$10 \le \gamma \le 50$

$30^{o} \le \theta \le 90^{o}$

$F_y \le 72\text{ksi}$

$\frac{g_b}{D} > -0.6(\text{for K Joints})$

INPUT: Basic Geometric Parameters for Simple Tubular Joints

$F_y := 345$

$F_{yb} := 345$

$\theta := 49.6°$	Brace included angle
$g_b := 1058$	Gap between braces
$t := 22.2$	Brace wall thickness at intersection
$T_c := 53.3$	Chord wall thickness at intersection
$d := 853$	Brace outside diameter
$D := 2560$	Chord outside diameter

$A_c := \frac{\pi}{4} \cdot \left[D^2 - \left(D - 2T_c\right)^2\right] = 4.197 \times 10^5$ Chord cross sectional area

$\beta := \frac{d}{D}$ $\beta = 0.333$

$\gamma := \frac{D}{2 \cdot T_c}$ $\gamma = 24.015$

$\tau := \frac{t}{T_c}$ $\tau = 0.417$

$\phi := \frac{t \cdot F_{yb}}{T_c \cdot F_y}$ $\phi = 0.417$

$\frac{g_b}{D} = 0.413$

4.3.3. Strength Factor Q_u

K joint only with positive gap

$$Q_{g1} := \begin{cases} 1 + 0.2\cdot\left(1 - 2.8\cdot\frac{g_b}{D}\right)^3 & \text{if } \frac{g_b}{D} > 0.05 \\ \text{"gap too small"} & \text{otherwise} \end{cases} = 0.999$$

Gap factor

$$Q_g := \max\left(1, Q_{g1}\right)$$

$$Q_g = 1$$

Q_u varies with the joint and load type, as given in Table 4.3-1.

- **Q_u for Axial Loads**

$$\gamma = 24.015$$

$$Q_{u\_a1} := (16 + 1.2\cdot\gamma)\cdot\beta^{1.2}\cdot Q_g$$

$$Q_{u\_a} := \min\left(Q_{u\_a1}, 40\cdot\beta^{1.2}\cdot Q_g\right) \qquad \beta = 0.333$$

$$Q_{u\_a} = 10.698$$

- **Q_u for In-Plane Bending**

$$Q_{u\_ipb} := (5 + 0.7\cdot\gamma)\cdot\beta^{1.2}$$

$$Q_{u\_ipb} = 5.833$$

- **Q_u for Out-Of-Plane Bending**

$$Q_{u\_opb} := 2.5 + (4.5 + 0.2\cdot\gamma)\cdot\beta^{2.6}$$

$$Q_{u\_opb} = 3.034$$

4.3.4 Chord Load Factor Q_f

Q_f is a factor to account for the presence of nominal loads in the chord.

$$Z_p := \frac{1}{6}\cdot\left[D^3 - \left(D - 2T_c\right)^3\right]$$

Plastic section modulus

$M_p := F_y \cdot Z_p = 1.156 \times 10^{11}$ Plastic moment capacity of the chord

$P_y := F_y \cdot A_c = 1.448 \times 10^{8}$ Yield axial capacity of the chord

$FS := 1.6$ Safety factor

$P_c := -1462340$ Chord axial force

$M_{c\_ipb} := 2472810000$

$M_{c\_opb} := -75653000$

$$M_c := \sqrt{M_{c\_ipb}^2 + M_{c\_opb}^2} = 2.474 \times 10^{9}$$

$$AA := \left[\left(\frac{FS \cdot P_c}{P_y}\right)^2 + \left(\frac{FS \cdot M_c}{M_p}\right)^2\right]^{0.5} \qquad (4.3-3)$$

$AA = 0.038$

INPUT the following coefficients:

Coefficients depending on joint and load type as given in Table 4.3-2.

$C_{1\_a} := 0.2$

$C_{2\_a} := 0.2$

$C_{3\_a} := 0.3$

$$Q_{f\_a} := \left[1 + C_{1\_a} \cdot \left(\frac{FS \cdot P_c}{P_y}\right) - C_{2\_a} \cdot \left(\frac{FS \cdot \left|M_{c\_ipb}\right|}{M_p}\right) - C_{3\_a} \cdot AA^2\right] \qquad (4.3-2)$$

$Q_{f\_a} = 0.989$

$C_{1\_b} := 0.2$

$C_{2\_b} := 0$

$C_{3\_b} := 0.4$

$$Q_{f\_ipb} := \left[1 + C_{1\_b} \cdot \left(\frac{FS \cdot P_c}{P_y}\right) - C_{2\_b} \cdot \left(\frac{FS \cdot M_{c\_ipb}}{M_p}\right) - C_{3\_b} \cdot AA^2\right]$$

$Q_{f\_ipb} = 0.996$

$$Q_{f\_opb} := \left[1 + C_{1\_b}\cdot\left(\frac{FS\cdot P_c}{P_y}\right) - C_{2\_b}\cdot\left(\frac{FS\cdot M_{c\_opb}}{M_p}\right) - C_{3\_b}\cdot AA^2\right]$$

$$Q_{f\_opb} = 0.996$$

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.

$$P_a := Q_{u\_a}\cdot Q_{f\_a}\cdot\frac{F_y\cdot T_c^2}{FS\cdot\sin(\theta)}$$ allowable capacity for brace axial load

$$P_a = 8.515\times10^6$$

$$M_{a\_ipb} := Q_{u\_ipb}\cdot Q_{f\_ipb}\cdot\frac{F_y\cdot T_c^2\cdot d}{FS\cdot\sin(\theta)}$$ allowable capacity for brace BM

$$M_{a\_ipb} = 3.987\times10^9$$

$$M_{a\_opb} := Q_{u\_opb}\cdot Q_{f\_opb}\cdot\frac{F_y\cdot T_c^2\cdot d}{FS\cdot\sin(\theta)}$$ allowable capacity for brace BM

$$M_{a\_opb} = 2.074\times10^9$$

4.3.6 Strength Check

$$P := 4720646$$

$$M_{ipb} := 194146000$$

$$M_{opb} := -56799000$$

$$IR := \left|\frac{P}{P_a}\right| + \left(\frac{M_{ipb}}{M_{a\_ipb}}\right)^2 + \left|\frac{M_{opb}}{M_{a\_opb}}\right|$$

$$IR = 0.584$$

$$\left|\frac{P}{P_a}\right| = 0.554 \qquad \left(\frac{M_{ipb}}{M_{a\_ipb}}\right)^2 = 2.371\times10^{-3} \qquad \left|\frac{M_{opb}}{M_{a\_opb}}\right| = 0.027$$

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/mm^2)

Young's Modulus, E (N/mm^2)

Axial tension stress due to factored loads, σ_t (N/mm^2)

Axial compression stress due to factored loads, σ_c (N/mm^2)

Bending stress due to factored loads about y axis, f_{by} (N/mm^2)

Bending stress due to factored loads about z axis, f_{bz} (N/mm^2)

Hoop stress due to factored loads, f_h (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)

$D := 1066.8$	$D = 1.067 \times 10^3$	mm
$t := 17.8$	$t = 17.8$	mm
$f_y := 345$	$f_y = 345$	$\frac{N}{mm^2}$
$E := 210000$	$E = 2.1 \times 10^5$	$\frac{N}{mm^2}$
$\sigma_c := 22.4$	$\sigma_c = 22.4$	$\frac{N}{mm^2}$
$\sigma_t := 0$	$\sigma_t = 0$	$\frac{N}{mm^2}$
$\sigma_{by} := 5.1$	$\sigma_{by} = 5.1$	$\frac{N}{mm^2}$
$\sigma_{bz} := 11.8$	$\sigma_{bz} = 11.8$	$\frac{N}{mm^2}$
$L_y := 16178$	$L_y = 1.618 \times 10^4$	mm

$L_z := 16178$ $\qquad L_z = 1.618 \times 10^4$ mm

Combined bending stress is given by, $\sigma_b = (\sigma_{by}^2 + \sigma_{bz}^2)^{0.5}$

$\sigma_b := \left(\sigma_{by}^2 + \sigma_{bz}^2\right)^{0.5}$ $\qquad \sigma_b = 12.855$ $\frac{N}{mm^2}$

13.2.2 Axial tension, $\sigma_{t <=} f_{t\,/}\,\gamma_{Rt}$ (13.2-1)

$\gamma_{Rt} := 1.05$

$f_t := f_y$

$UR_{13.2.2} := \frac{\sigma_t}{\frac{f_t}{\gamma_{Rt}}}$ $\qquad UR_{13.2.2} = 0$

13.2.3.3 Local buckling

$C_x := 0.3$ elastic critical buckling coefficient - account for the effect of initial geometric imperfections within the tolerance limits given in Clause 21.

$f_{xe} := 2 \cdot C_x \cdot E \cdot \frac{t}{D}$ $\qquad f_{xe} = 2.102 \times 10^3$ $\qquad (13.2 - 10)$

$$f_{yc} := \begin{cases} f_y \ \text{ if } \frac{f_y}{f_{xe}} \le 0.17 & (13.2 - 8) \\ \left[\left(1.047 - 0.274 \cdot \frac{f_y}{f_{xe}}\right) \cdot f_y\right] \ \text{ if } \frac{f_y}{f_{xe}} > 0.17 & (13.2 - 9) \end{cases}$$

$f_{yc} = 345$

13.2.3.2 Column buckling

INPUT following parameter:

$k := .7$ effective length factor, in y or z direction, see 13.5

$L_y = 1.618 \times 10^4$

$$L_z = 1.618 \times 10^4$$

$$I := \frac{\pi}{64} \cdot \left[D^4 - (D - 2 \cdot t)^4\right] \qquad I = 8.071 \times 10^9$$

$$Area := \frac{\pi}{4} \cdot \left[D^2 - (D - 2 \cdot t)^2\right] \qquad Area = 5.866 \times 10^4$$

$$r := \sqrt{\frac{I}{Area}} \qquad r = 370.931$$

Column slenderness parameter:

$$\lambda := \frac{k \cdot L_y}{\pi \cdot r} \sqrt{\frac{f_{yc}}{E}} \qquad \lambda = 0.394 \qquad 13.2 - 7$$

$$f_c := \begin{cases} \left[\left(1.0 - 0.278 \cdot \lambda^2\right) \cdot f_{yc}\right] & \text{if } \lambda \le 1.34 \qquad 13.2 - 5 \\ \left(\frac{0.9}{\lambda^2} \cdot f_{yc}\right) & \text{if } \lambda > 1.34 \qquad 13.2 - 6 \end{cases}$$

$$f_c = 330.119$$

13.2.3.1 General, $\sigma_c <= f_c / \gamma_{Rc}$ (13.2-3)

$$\sigma_c = 22.4$$

$$\gamma_{Rc} := 1.18$$

$$UR_{13.2.4} := \frac{\sigma_c}{\frac{f_c}{\gamma_{Rc}}} \qquad UR_{13.2.4} = 0.08$$

13.2.4 Bending, $\sigma_b = M/Z_e <= f_b / \gamma_{Rb}$ (13.2-11)

$$Z_p := \frac{1}{6} \cdot \left[D^3 - (D - 2 \cdot t)^3\right] \qquad Z_p = 1.959 \times 10^7 \qquad \text{plastic section modulus}$$

$$Z_e := \frac{\pi}{64} \cdot \frac{\left[D^4 - (D - 2 \cdot t)^4\right]}{\frac{D}{2}} \qquad Z_e = 1.513 \times 10^7 \qquad \text{elastic section modulus}$$

$$f_b := \begin{cases} \left(\frac{Z_p}{Z_e} \cdot f_y\right) & \text{if } \frac{f_y \cdot D}{E \cdot t} \le 0.0517 & (13.2-13) \\ \left[\left(1.13 - 2.58 \cdot \frac{f_y \cdot D}{E \cdot t}\right) \cdot \frac{Z_p}{Z_e} \cdot f_y\right] & \text{if } 0.0517 < \frac{f_y \cdot D}{E \cdot t} \le 0.1034 & (13.2-14) \\ \left(0.94 - 0.76 \cdot \frac{f_y \cdot D}{E \cdot t}\right) \cdot \frac{Z_p}{Z_e} \cdot f_y & \text{if } 0.1 < \frac{f_y \cdot D}{E \cdot t} \le 120 \cdot f_y & (13.2-15) \end{cases}$$

$f_b = 391.24$

$\gamma_{Rb} := 1.05$ partial resistance factor for bending stress

$\sigma_b = 12.855$

$$UR_{13.2.11} := \frac{\sigma_b}{\frac{f_b}{\gamma_{Rb}}} \qquad UR_{13.2.11} = 0.034$$

13.2.5.1 Beam shear $\tau_b = 2V/A <= f_v/\gamma_{Rv}$

$\gamma_{Rv} := 1.05$

$V_{shear} := 38529$

$$f_v := \frac{f_y}{\sqrt{3}} \qquad f_v = 199.186$$

$$UR_{13.2.17} := \frac{\frac{2 \cdot V_{shear}}{Area}}{\frac{f_v}{\gamma_{Rv}}} \qquad UR_{13.2.17} = 6.925 \times 10^{-3}$$

13.2.5.2 Torsional shear

$I_p := \frac{\pi}{32} \cdot \left[D^4 - (D - 2 \cdot t)^4\right]$ $I_p = 1.614 \times 10^{10}$ polar moment of inertia

$M_{vt} := 41438000$ Torsional moment due to factored actions

$$UR_{13.2.19} := \frac{\frac{M_{vt} \cdot D}{2 \cdot I_p}}{\frac{f_v}{\gamma_{Rv}}} \qquad UR_{13.2.19} = 7.218 \times 10^{-3}$$

13.2.6 Hydrostatic pressure

$\gamma_{fG1} := 1.3$ partial action factor for permanent actions 1, see Table 9.10-1

$\rho_w := 1025$ density of the sea water which may be taken as 1025 kg/m^3

$g_c := 9.810$ the acceleration due to gravity (m/s^2)

$d := 110.000$ the still water depth to the sea floor (m)

$H_w := 26$ wave height (m)

$T_p := 13.9$ wave period (s)

$z := -95.38$ the depth of the member relative to still water level (measured positive upwards) (m)

$$L_{wave} := \frac{g_c \cdot T_p^{\,2}}{2 \cdot \pi} \qquad L_{wave} = 301.661$$

Effective hydrostatic head (m):

$$H_z := -z + \frac{H_w}{2} \cdot \frac{\cosh\left[\frac{2 \cdot \pi}{L_{wave}} (d + z)\right]}{\cosh\left(\frac{2 \cdot \pi}{L_{wave}} \cdot d\right)} \qquad H_z = 98.105$$

The factored hydrostatic pressure (p, N/mm^2) shall be calculated:

$$p := \frac{\gamma_{fG1} \cdot g_c \cdot \rho_w \cdot H_z}{1000000}$$

$$p = 1.282 \qquad \frac{N}{mm^2} \qquad (13.2 - 20)$$

13.2.6.2 Hoop buckling

Tubular members subjected to external pressure shall be designed to satisfy the following condition:

$\sigma_h = pD/2t <= f_h/\gamma_{Rh}$ (13.2-22)

$\gamma_{Rh} := 1.25$

Hoop stress due to the forces from factored hydrostatic pressure:

$\sigma_h := \frac{p \cdot D}{2 \cdot t}$ $\sigma_h = 38.429$ $\frac{N}{mm^2}$

Geometric Parameter:

$L_r := 16178$ the length of tubular between stiffening rings, diaphragms, or end connections

$$\mu := \frac{L_r}{D} \cdot \sqrt{\frac{2 \cdot D}{t}}$$

Elastic critical hoop buckling coefficient C_h:

$$C_h := \begin{cases} \left(0.44 \cdot \frac{t}{D}\right) \text{ if } \mu \geq 1.6 \cdot \frac{D}{t} & (13.2-27) \\ \left[0.44 \cdot \frac{t}{D} + 0.21 \cdot \left(\frac{D}{t}\right)^3 \cdot \mu^4\right] \text{ if } 0.825 \cdot \frac{D}{t} \leq \mu < 1.6 \cdot \frac{D}{t} & (13.2-28) \\ \frac{0.737}{\mu - 0.579} \text{ if } 1.5 \leq \mu < 0.825 \cdot \frac{D}{t} & (13.2-29) \\ 0.80 \text{ if } \mu < 1.5 & (13.2-30) \end{cases}$$

$f_{he} := 2 \cdot C_h \cdot \frac{E \cdot t}{D}$ $f_{he} = 51.449$ (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:

$$f_h := \begin{cases} f_y \text{ if } f_{he} > 2.44 \cdot f_y & (13.2-23) \\ \left[0.7 \cdot \left(\frac{f_{he}}{f_y}\right)^{0.4} \cdot f_y\right] \text{ if } 0.55 f_y < f_{he} \leq 2.44 \cdot f_y & (13.2-24) \\ f_{he} \text{ if } f_{he} \leq 0.55 \cdot f_y & (13.2-25) \end{cases}$$

$f_h = 51.449$

The utilization of a member under external pressure shall be calculated:

$$UR_{13.2.31} := \frac{\sigma_h}{\frac{f_h}{\gamma_{Rh}}} \qquad (13.2-31)$$

$UR_{13.2.31} = 0.934$

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:

$$UR_{13.3.2} := \frac{\gamma_{Rt} \cdot \sigma_t}{f_t} + \frac{\gamma_{Rb} \cdot \sqrt{\sigma_{by}^2 + \sigma_{bz}^2}}{f_b} \qquad (13.3-2)$$

$UR_{13.3.2} = 0.034$

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:

$k_y := .7$ $\qquad C_{my} := 0.85$

$k_z := .7$ $\qquad C_{mz} := 0.85$

$$f_{ey} := \frac{\pi^2 \cdot E}{\left(k_y \cdot \frac{L_y}{r}\right)^2} \qquad f_{ey} = 2.224 \times 10^3 \qquad (13.3-5)$$

$$f_{ez} := \frac{\pi^2 \cdot E}{\left(k_z \cdot \frac{L_z}{r}\right)^2} \qquad f_{ez} = 2.224 \times 10^3 \qquad (13.3-6)$$

$$UR_{13.3.7} := \begin{cases} 0 & \text{if } p > 0 \\ \dfrac{\gamma_{Rc} \cdot \sigma_c}{f_c} + \dfrac{\gamma_{Rb}}{f_b} \cdot \left[\left(\dfrac{C_{my} \cdot \sigma_{by}}{1 - \dfrac{\sigma_c}{f_{ey}}} \right)^2 + \left(\dfrac{C_{mz} \cdot \sigma_{by}}{1 - \dfrac{\sigma_c}{f_{ez}}} \right)^2 \right]^{0.5} & \text{if } p = 0 \end{cases} \qquad (13.3-7)$$

$$UR_{13.3.7} = 0$$

$$UR_{13.3.8} := \begin{cases} 0 & \text{if } p > 0 \\ \left(\dfrac{\gamma_{Rc} \cdot \sigma_c}{f_c} + \dfrac{\gamma_{Rb} \cdot \sqrt{\sigma_{by}^2 + \sigma_{bz}^2}}{f_b} \right) & \text{if } p = 0 \end{cases} \qquad (13.3-8)$$

$$U_{13.3.3} := \max(UR_{13.3.7}, UR_{13.3.8}) \qquad U_{13.3.3} = 0$$

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 Equ (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

$$\sigma_{nc} := \begin{cases} (\sigma_q - \sigma_{tc}) & \text{if } \sigma_{tc} < \sigma_q \qquad (13.4-5) \\ (\sigma_{cc} - \sigma_q) & \text{if } \sigma_{cc} > \sigma_q \qquad (13.4-6) \end{cases}$$

13.4.2 Axial Tension, Bending and Hydrostatic Pressure

$$\eta := 5 - 4 \cdot \frac{f_h}{f_y} \qquad (13.4-11) \qquad \eta = 4.403$$

$$B := \frac{\gamma_{Rh} \cdot \sigma_h}{f_h} \qquad (13.4-10) \qquad B = 0.934$$

The representive bending strength in the presence of external hydrostatic pressure:

$$f_{bh} := f_b \cdot \left(\sqrt{1 + 0.09 \cdot B^2 - B^{2 \cdot \eta}} - 0.3B\right) \qquad (13.4-9) \qquad f_{bh} = 175.796$$

The representive axial tensile strength in the presence of external hydrostatic pressure:

$$f_{th} := f_y \cdot \left(\sqrt{1 + 0.09 \cdot B^2 - B^{2 \cdot \eta}} - 0.3 \cdot B\right) \qquad (13.4-8) \qquad f_{th} = 155.019$$

$$UR_{13.4.12} := \frac{\gamma_{Rt} \cdot \sigma_{tc}}{f_{th}} + \frac{\gamma_{Rb} \cdot \sqrt{\sigma_{by}^2 + \sigma_{bz}^2}}{f_{bh}} \qquad (13.4-12)$$

$$UR_{13.4.12} = 0.077$$

13.4.3 Axial compression, bending and hydrostatic pressure

$\lambda = 0.394$

$\sigma_q = 19.215$

$\sigma_x := \sigma_b + \sigma_{cc}$ $\qquad \sigma_x = 35.255$ $\qquad$ Maximum combined compressive stress

$f_{xe} = 2.102 \times 10^3$ $\qquad f_{he} = 51.449$

If the maximum combined compressive stress, σ_x, and the representative elastic local buckling strength, f_{xe}, exceed the limits given below ($\sigma_{13.4.17.1} > \sigma_{13.4.17.2}$),

$$\sigma_{13.4.17.1} := \frac{0.5 \cdot f_{he}}{\gamma_{Rh}} \qquad \sigma_{13.4.17.1} = 20.58$$

$$\sigma_{13.4.17.2} := \max\left(\sigma_x, \frac{f_{xe}}{\gamma_{Rc}}\right) \qquad \sigma_{13.4.17.2} = 1.782 \times 10^3$$

then Equation (13.4-8) shall also be satisfied (less than or equal to 1)

$$UR_{13.4.18} := \frac{\sigma_x - \dfrac{0.5 \cdot f_{he}}{\gamma_{Rh}}}{\dfrac{f_{xe}}{\gamma_{Rc}} - \dfrac{0.5 \cdot f_{he}}{\gamma_{Rh}}} + \left(\frac{\gamma_{Rh} \cdot \sigma_h}{f_{he}}\right)^2 \qquad (13.4 - 18)$$

$$UR_{13.4.18} = 0.88$$

$$f_{ch} := \left| \begin{array}{l} \left[\dfrac{f_{yc}}{2} \cdot \left[\left(1.0 - 0.278 \cdot \lambda^2\right) - \dfrac{2 \cdot \sigma_q}{f_{yc}} + \sqrt{\left(1.0 - 0.278\lambda^2\right)^2 + 1.12 \cdot \dfrac{\lambda^2 \cdot \sigma_q}{f_{yc}}}\right]\right] \text{ if } \lambda \le 1.34 \cdot \sqrt{\left(1 - \dfrac{2 \cdot \sigma_q}{f_{yc}}\right)^{-1}} \\ \left(\dfrac{0.9}{\lambda^2} \cdot f_{yc}\right) \text{ if } \lambda > 1.34 \cdot \sqrt{\left(1 - \dfrac{2 \cdot \sigma_q}{f_{yc}}\right)^{-1}} \end{array} \right.$$

$$f_{ch} = 311.775$$

$$UR_{13.4.19} := \frac{\gamma_{Rc} \cdot \sigma_c}{f_{yc}} + \frac{\gamma_{Rb} \cdot \sqrt{\sigma_{by}^2 + \sigma_{bz}^2}}{f_{bh}} \qquad (13.4 - 19)$$

$$UR_{13.4.20} := \frac{\gamma_{Rc} \cdot \sigma_c}{f_{ch}} + \frac{\gamma_{Rb}}{f_{bh}} \cdot \left[\left(\frac{C_{my} \cdot \sigma_{by}}{1 - \dfrac{\sigma_c}{f_{ey}}}\right)^2 + \left(\frac{C_{mz} \cdot \sigma_{by}}{1 - \dfrac{\sigma_c}{f_{ez}}}\right)^2\right]^{0.5} \qquad (13.4 - 20)$$

$$UR_{13.4.21} := \frac{\sigma_x - 0.5 \cdot \dfrac{f_{he}}{\gamma_{Rh}}}{\dfrac{f_{xe}}{\gamma_{Rc}} - \dfrac{0.5 \cdot f_{he}}{\gamma_{Rh}}} + \left(\frac{\gamma_{Rh} \cdot \sigma_h}{f_{he}}\right)^2 \qquad (13.4 - 21)$$

$UR_{13.4.19} = 0.153$ $\qquad UR_{13.4.20} = 0.122$ $\qquad UR_{13.4.21} = 0.88$

$U_m := \max\left(UR_{13.4.19}, UR_{13.4.20}, UR_{13.4.21}\right)$ $\qquad U_m = 0.88$

14. Strength of tubular joints

14.3.1 General

Validity Range

$0.2 \le \beta \le 1.0$

$10 \le \gamma \le 50$

$30^{o} \le \theta \le 90^{o}$

$\tau \le 1.0$

$F_y \le 500 \quad \frac{N}{mm^2}$

$g_b \cdot T_c > -1.2\gamma$ For K-joints

INPUT: Basic Geometric Parameters for Simple Tubular Joints

$F_y := 345$	Yield strength, chord
$F_{yb} := 345$	Yield strength, brace
$\theta := 49.6°$	Brace included angle
$g_b := 1058$	Gap between braces
$t := 22.2$	Brace wall thickness at intersection
$T_c := 53.3$	Chord wall thickness at intersection
$d := 853$	Brace outside diameter
$D := 2560$	Chord outside diameter

$$A_c := \frac{\pi}{4}\cdot\left[D^2 - \left(D - 2T_c\right)^2\right] = 4.197 \times 10^5$$ Chord cross sectional area

$$\beta := \frac{d}{D} \qquad \beta = 0.333$$

$$\gamma := \frac{D}{2\cdot T_c} \qquad \gamma = 24.015$$

$$\tau := \frac{t}{T_c} \qquad \tau = 0.417$$

$$\phi := \frac{t\cdot F_{yb}}{T_c\cdot F_y}$$

14.3.3. Strength Factor Q_u

K joint only with positive gap

$$Q_{g1} := \begin{cases} 1.9 - 0.7\gamma^{-0.5} \cdot \left(\frac{g_b}{T_c}\right)^{0.5} & \text{if } \frac{g_b}{T_c} \geq 2 \\ \text{"gap too small"} & \text{otherwise} \end{cases} \qquad (14.3\text{-}7)$$

$$Q_g := \max(1, Q_{g1})$$

Gap factor

$$Q_g = 1.264$$

$$Q_\beta := \begin{cases} \frac{0.3}{\beta \cdot (1 - 0.8333 \cdot \beta)} & \text{if } \beta > 0.6 \\ 1.0 & \text{otherwise} \end{cases} \qquad \begin{matrix}(14.3\text{-}5) \\ (14.3\text{-}6)\end{matrix}$$

$$Q_\beta = 1$$

Geometrical factor

Q_u varies with the joint and load type, as given in Table 14.3-1.

- **Q_u for Axial Loads**

$$Q_{u\_a} := (1.9 + 19 \cdot \beta) \cdot Q_\beta^{0.5} \cdot Q_g$$

$$Q_{u\_a} = 10.4$$

- **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**

$$Q_{u\_opb} := 3.2 \cdot \gamma^{\left(0.5 \cdot \beta^2\right)}$$

$$Q_{u\_opb} = 3.818$$

14.3.4 Chord Load Factor Q_f

Q_f is a factor to account for the presence of nominal loads in the chord.

$$Z_p := \frac{1}{6}\left[D^3 - \left(D - 2T_c\right)^3\right]$$ Plastic section modulus

$$M_p := F_y \cdot Z_p = 1.156 \times 10^{11}$$ Plastic moment capacity of the chord

$$P_y := F_y \cdot A_c = 1.448 \times 10^{8}$$ 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 resisitance factor for yield strength

INPUT the following coefficients:

Coefficients depending on joint and load type as given in Table 14.3-2.

$$C_{1\_a} := 14$$
$$C_{2\_a} := 43$$

$$q_{A\_a} := \left[C_{1\_a} \cdot \left(\frac{P_c}{P_y}\right)^2 + C_{2\_a} \cdot \left(\frac{M_{c\_ipb}}{M_p}\right)^2 + C_{2\_a} \cdot \left(\frac{M_{c\_opb}}{M_p}\right)^2\right]^{0.5} \cdot \gamma_{Rq} \qquad (14.3\text{-}10)$$

$$q_{A\_a} = 0.379$$

$$Q_{f\_a} := \left(1 - \lambda_a \cdot q_{A\_a}^{\ 2}\right) \qquad (14.3\text{-}9)$$

Chord force factor, axial force

$$Q_{f\_a} = 0.996$$

$C_{1\_b} := 25$

$C_{2\_b} := 43$

$$q_{A\_b} := \left[C_{1\_b} \cdot \left(\frac{P_c}{P_y} \right)^2 + C_{2\_b} \cdot \left(\frac{M_{c\_ipb}}{M_p} \right)^2 + C_{2\_b} \cdot \left(\frac{M_{c\_opb}}{M_p} \right)^2 \right]^{0.5} \cdot \gamma_{Rq} \qquad (14.3\text{-}10)$$

$q_{A\_b} = 0.497$

$$Q_{f\_ipb} := 1.0 - \lambda_{ipb} \cdot q_{A\_b}^{\ 2} \qquad (14.3\text{-}9)$$

Chord force factor, in-plane bending

$Q_{f\_ipb} = 0.989$

$$Q_{f\_opb} := 1.0 - \lambda_{opb} \cdot q_{A\_b}^{\ 2} \qquad (14.3\text{-}9)$$

Chord force factor, out-of-plane bending

$Q_{f\_opb} = 0.995$

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.

$$P_{uj} := \frac{F_y \cdot T_c^{\ 2}}{\sin(\theta)} \cdot Q_{u\_a} \cdot Q_{f\_a}$$

Representative joint axial strength (14.3-1)

$P_{uj} = 1.333 \times 10^7$

$$M_{uj\_ipb} := \frac{F_y \cdot T_c^{\ 2} \cdot d}{\sin(\theta)} \cdot Q_{u\_ipb} \cdot Q_{f\_ipb}$$

Representative joint moment strength, in-plane bending (14.3-2)

$M_{uj\_ipb} = 7.977 \times 10^9$

$$M_{uj\_opb} := \frac{F_y \cdot T_c^{\ 2} \cdot d}{\sin(\theta)} \cdot Q_{u\_opb} \cdot Q_{f\_opb}$$

Representative joint moment strength, out-of-plane bending (14.3-2)

$M_{uj\_opb} = 4.169 \times 10^9$

$\gamma_{Rj} := 1.05$ Partial resistance factor for tubular joints

$$P_d := \frac{P_{uj}}{\gamma_{Rj}} = 1.269 \times 10^7$$ Joint axial design strength

$$M_{d\_ipb} := \frac{M_{uj\_ipb}}{\gamma_{Rj}} = 7.597 \times 10^9$$ Design joint bending moment strength, in-plane bending

$$M_{d\_opb} := \frac{M_{uj\_opb}}{\gamma_{Rj}} = 3.971 \times 10^9$$ 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, σ_y (N/mm^2)

Young's Modulus, E (N/mm^2)

Design Axial Tension Forces due to factored loads, N_{Sdt} (N)

Design Axial compression Forces due to factored loads, N_{Sdc} (N)

Design Bending Moment due to factored loads about y axis, M_{ySd} (N-mm)

Design Bending Moment due to factored loads about z axis, M_{zSd} (N-mm)

Design Shear Force V_{Sd} (N)

Design Hydrostatic Pressure, p_{Sd} (N/mm^2)

Hoop stress due to factored loads, f_h (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)

$D := 1066.8$ $\qquad D = 1.067 \times 10^3$ mm

$t := 17.8$ $\qquad t = 17.8$ mm

$f_y := 345$ $\qquad f_y = 345$ $\frac{N}{mm^2}$

$E := 210000$ $\qquad E = 2.1 \times 10^5$ $\frac{N}{mm^2}$

$N_{Sdc} := 2330506$ $\qquad N_{Sdc} = 2.331 \times 10^6$ N

$N_{Sdt} := 0$ $\qquad N_{Sdt} = 0$ N

$M_{ySd} := 88794000$ $\qquad M_{ySd} = 8.879 \times 10^7$ N − mm

$M_{zSd} := 230831000$ $\qquad M_{zSd} = 2.308 \times 10^{8}$ N − mm

$V_{Sd} := 39479$ $\qquad V_{Sd} = 3.948 \times 10^{4}$ N

$p_{Sd} := 0.985$ $\qquad p_{Sd} = 0.985$ $\frac{N}{mm^2}$

$l_y := 16178$ $\qquad l_y = 1.618 \times 10^{4}$ mm

$l_z := 16178$ $\qquad l_z = 1.618 \times 10^{4}$ mm

$\frac{D}{t} = 59.933$

6.3.2 Axial tension

$Area := \frac{\pi}{4}\cdot\left[D^2 - (D - 2\cdot t)^2\right]$ $\qquad$ Cross Section Area $\quad Area = 5.866 \times 10^{4}$

$\gamma_{Mt} := 1.15$

$N_{tRd} := \frac{Area\cdot f_y}{\gamma_{Mt}}$ $\qquad N_{tRd} = 1.76 \times 10^{7}$

$UR_{6.1} := \frac{N_{Sdt}}{N_{tRd}}$ $\qquad$ (6.1)

$UR_{6.1} = 0$

6.3.3 Axial Compression

INPUT the following parameters:

$k := .7$ $\qquad$ See Table 6-2

$L_t := 16178$ $\qquad$ Length of tubular between stiffening rings, diaphragms, or end connections

$C_e := 0.3$ $\qquad$ critial elastic buckling coefficient

$I := \frac{\pi}{64}\cdot\left[D^4 - (D - 2\cdot t)^4\right]$ $\qquad$ Moment of inertia $\qquad I = 8.071 \times 10^{9}$ mm^4

$i := \sqrt{\frac{I}{Area}}$ $\qquad$ radius of gyration $\qquad i = 370.931$ mm

$f_{cle} := 2\cdot C_e\cdot E\cdot\frac{t}{D}$ $\qquad$ characteristic elastic local buckling strength

$f_{cle} = 2.102 \times 10^3$

The characteristic local buckling strength should be determined from:

$$f_{cl} := \begin{cases} f_y \ \text{ if } \dfrac{f_y}{f_{cle}} \le 0.170 & (6.6) \\ \left[\left(1.047 - 0.274 \cdot \dfrac{f_y}{f_{cle}}\right) \cdot f_y\right] \ \text{ if } 0.170 < \dfrac{f_y}{f_{cle}} \le 1.911 & (6.7) \\ f_{cle} \ \text{ if } \dfrac{f_y}{f_{cle}} > 1.911 & (6.8) \end{cases}$$

$f_{cl} = 345$

$\lambda_y := \dfrac{k \cdot l_y}{\pi \cdot i} \cdot \sqrt{\dfrac{f_{cl}}{E}}$ column slenderness parameter (6.5) $\lambda_y = 0.394$

$\lambda_z := \dfrac{k \cdot l_z}{\pi \cdot i} \cdot \sqrt{\dfrac{f_{cl}}{E}}$ column slenderness parameter (6.5) $\lambda_z = 0.394$

$$f_{cy} := \begin{cases} \left[\left(1.0 - 0.28 \cdot {\lambda_y}^2\right) \cdot f_y\right] \ \text{ if } \lambda_y \le 1.34 & (6.3) \\ \left(\dfrac{0.9}{{\lambda_y}^2} \cdot f_y\right) \ \text{ if } \lambda_y > 1.34 & (6.4) \end{cases}$$

$f_{cy} = 330.012$

$$f_{cz} := \begin{cases} \left[\left(1.0 - 0.28 \cdot {\lambda_z}^2\right) \cdot f_y\right] \ \text{ if } \lambda_z \le 1.34 & (6.3) \\ \left(\dfrac{0.9}{{\lambda_z}^2} \cdot f_y\right) \ \text{ if } \lambda_z > 1.34 & (6.4) \end{cases}$$

$f_{cz} = 330.012$

$f_c := \min\left(f_{cy}, f_{cz}\right)$ $f_c = 330.012$

Define the characteristic elastic hoop buckling strength (Clause 6.3.6.1):

Geometric parameter, μ, :

$$\mu := \frac{L_t}{D} \cdot \sqrt{\frac{2 \cdot D}{t}} \qquad \mu = 166.031$$

$$C_h := \begin{cases} \left(0.44 \cdot \frac{t}{D}\right) & \text{if } \mu \ge 1.6 \cdot \frac{D}{t} \\ \left[0.44 \cdot \frac{t}{D} + \frac{0.21 \cdot \left(\frac{D}{t}\right)^3}{\mu^4}\right] & \text{if } 0.825 \cdot \frac{D}{t} \le \mu < 1.6 \cdot \frac{D}{t} \\ \frac{0.737}{\mu - 0.579} & \text{if } 1.5 \le \mu < 0.825 \cdot \frac{D}{t} \\ 0.80 & \text{if } \mu < 1.5 \end{cases} \qquad C_h = 7.342 \times 10^{-3}$$

Then elastic hoop buckling strength, $f_{he,}$ is determined:

$$f_{he} := 2 \cdot C_h \cdot E \cdot \frac{t}{D} \qquad (6.20) \qquad f_{he} = 51.449$$

Charateristic elastic buckling strength:

$$f_h := \begin{cases} f_y & \text{if } f_{he} > 2.44 \cdot f_y & (6.17) \\ \left[0.7 \cdot f_y \cdot \left(\frac{f_{he}}{f_y}\right)^{0.4}\right] & \text{if } 2.44 \cdot f_y \ge f_{he} > 0.55 \cdot f_y & (6.18) \\ f_{he} & \text{if } f_{he} \le 0.55 \cdot f_y & (6.19) \end{cases} \qquad f_h = 51.449$$

Define Material Factor (Clause 6.3.7)

$$W_e := \frac{2 \cdot I}{D} \qquad \text{Elastic Section Modulus} \qquad W_e = 1.513 \times 10^7$$

$N_{sd} := N_{Sdc}$ $\qquad N_{sd} = 2.331 \times 10^6$ $\qquad N_{sd}$ is negative if in tension

$$\sigma_{cSd} := \frac{N_{sd}}{Area} + \frac{\sqrt{M_{ySd}^2 + M_{zSd}^2}}{W_e} \qquad (6.25) \qquad \sigma_{cSd} = 56.074$$

$$\sigma_{psd} := \frac{p_{Sd} \cdot D}{2 \cdot t} \qquad (6.16) \qquad \sigma_{psd} = 29.517$$

$$\lambda_c := \sqrt{\frac{f_y}{f_{cle}}} \quad \text{and} \quad \lambda_h := \sqrt{\frac{f_y}{f_{he}}} \qquad (6.24) \qquad \lambda_c = 0.405$$

$$\lambda_h = 2.59$$

$$\lambda_s := \frac{\left|\sigma_{cSd}\right|}{f_{c1}} \cdot \lambda_c + \left(\frac{\sigma_{psd}}{f_h}\right)^2 \cdot \lambda_h \qquad (6.23) \qquad \lambda_s = 0.918$$

$$\gamma_M := \begin{vmatrix} 1.15 \ \text{ if } \lambda_s < 0.5 \\ \left(0.85 + 0.60 \cdot \lambda_s\right) \ \text{ if } 0.5 \le \lambda_s \le 1.0 \\ 1.45 \ \text{ if } \lambda_s > 1.0 \end{vmatrix} \qquad (6.22)$$

$$\gamma_M = 1.401$$

$$N_{cRd} := \frac{Area \cdot f_c}{\gamma_M} \qquad N_{cRd} = 1.382 \times 10^7$$

$$UR_{6.2} := \frac{N_{sd}}{N_{cRd}} \qquad (6.2) \qquad UR_{6.2} = 0.169$$

6.3.4 Bending

$$Z := \frac{1}{6} \cdot \left[D^3 - (D - 2 \cdot t)^3\right]$$ plastic section modulus

$$M_{Sd} := \sqrt{M_{ySd}^2 + M_{zSd}^2}$$ design bending moment

Characteristic bending strength:

$$f_m := \begin{vmatrix} \left(\frac{Z}{W_e} \cdot f_y\right) \ \text{ if } \frac{f_y \cdot D}{E \cdot t} \le 0.0517 & (6.10) \\ \left[\left(1.13 - 2.58 \cdot \frac{f_y \cdot D}{E \cdot t}\right) \cdot \frac{Z}{W_e} \cdot f_y\right] \ \text{ if } 0.0517 < \frac{f_y \cdot D}{E \cdot t} \le 0.1034 & (6.11) \\ \left[\left(0.94 - 0.76 \cdot \frac{f_y \cdot D}{E \cdot t}\right) \cdot \frac{Z}{W_e} \cdot f_y\right] \ \text{ if } 0.1034 < \frac{f_y \cdot D}{E \cdot t} \le 120 \cdot \frac{f_y}{E} & (6.12) \end{vmatrix}$$

$f_m = 391.24$ $\gamma_M = 1.401$

$M_{Rd} := \frac{f_m \cdot W_e}{\gamma_M}$ $M_{Rd} = 4.226 \times 10^9$ Moment Resistance

$UR_{6.9} := \frac{M_{Sd}}{M_{Rd}}$ $UR_{6.9} = 0.059$

6.3.5 Shear

$V_{Sd} = 3.948 \times 10^4$

$\gamma_{Ms} := 1.15$

$V_{Rd} := \frac{Area \cdot f_y}{2 \cdot \sqrt{3} \cdot \gamma_{Ms}}$ $V_{Rd} = 5.08 \times 10^6$ Shear Resistance

$UR_{6.13} := \frac{V_{Sd}}{V_{Rd}}$ $UR_{6.13} = 7.771 \times 10^{-3}$

$M_{TSd} := 45301000$

$I_p := 2 \cdot I$

$M_{TRd} := \frac{2 \cdot I_p \cdot f_y}{D \cdot \sqrt{3} \cdot \gamma_{Ms}}$ $M_{TRd} = 5.242 \times 10^9$

$UR_{6.14} := \frac{M_{TSd}}{M_{TRd}}$ $UR_{6.14} = 8.642 \times 10^{-3}$

6.3.6.1 Hoop Buckling

$\sigma_{psd} = 29.517$ $f_h = 51.449$

$f_{hRd} := \frac{f_h}{\gamma_M}$ Hoop Stress Resistance

$UR_{6.15} := \frac{\sigma_{psd}}{\frac{f_h}{\gamma_M}}$ $UR_{6.15} = 0.804$

6.3.8 Tubular members subjected to combined loads without hydrostatic pressure

6.3.8.1 Axial Tension and Bending

$N_{Sdt} = 0$ $\quad M_{ySd} = 8.879 \times 10^{7}$ $\quad M_{zSd} = 2.308 \times 10^{8}$

$N_{tRd} = 1.76 \times 10^{7}$ $\quad M_{Rd} = 4.226 \times 10^{9}$

$$UR_{6.26} := \begin{cases} 0 & \text{if } p_{Sd} > 0 \\ \left[\left[\left(\dfrac{N_{Sdt}}{N_{tRd}}\right)^{1.75} + \dfrac{\sqrt{M_{ySd}^{2} + M_{zSd}^{2}}}{M_{Rd}}\right]\right] & \text{if } p_{Sd} = 0 \end{cases}$$

$UR_{6.26} = 0$

6.3.8.2 Axial Compression and Bending

INPUT the following parameters:

$C_{my} := 0.85$ $\quad C_{mz} := 0.85$ reduction factors corresponding to the member y and z axes

$$N_{ey} := \frac{\pi^{2} \cdot E \cdot Area}{\left[\left(\frac{k \cdot l_y}{i}\right)^{2}\right]} \qquad N_{ey} = 1.304 \times 10^{8}$$

$$N_{ez} := \frac{\pi^{2} \cdot E \cdot Area}{\left[\left(\frac{k \cdot l_z}{i}\right)^{2}\right]} \qquad N_{ez} = 1.304 \times 10^{8}$$

$$N_{clRd} := \frac{f_{cl} \cdot Area}{\gamma_M}$$ Design axial local buckling resistence

$$N_{clRd} = 1.445 \times 10^{7}$$

$$UR_{6.27} := \begin{cases} 0 & \text{if } p_{Sd} > 0 \\ \left[\dfrac{N_{sd}}{N_{cRd}} + \dfrac{1}{M_{Rd}} \cdot \left[\left(\dfrac{C_{my} \cdot M_{ySd}}{1 - \dfrac{N_{sd}}{N_{ey}}}\right)^2 + \left(\dfrac{C_{mz} \cdot M_{zSd}}{1 - \dfrac{N_{sd}}{N_{ez}}}\right)^2\right]^{0.5}\right] & \text{if } p_{Sd} = 0 \end{cases} \qquad (6.27)$$

$$UR_{6.28} := \begin{cases} 0 & \text{if } p_{Sd} > 0 \\ \left(\dfrac{N_{sd}}{N_{clRd}} + \dfrac{\sqrt{M_{ySd}^{\,2} + M_{zSd}^{\,2}}}{M_{Rd}}\right) & \text{if } p_{Sd} = 0 \end{cases} \qquad (6.28)$$

$UR_{6.27} = 0$ and $UR_{6.28} = 0$

6.3.8.3 Interaction Shear and Bending Moment

$$\frac{V_{Sd}}{V_{Rd}} = 7.771 \times 10^{-3}$$

$$UR_{6.3.8.3} := \begin{cases} \dfrac{\dfrac{M_{Sd}}{M_{Rd}}}{\sqrt{1.4 - \dfrac{V_{Sd}}{V_{Rd}}}} & \text{if } \dfrac{V_{Sd}}{V_{Rd}} \ge 0.4 \qquad (6.31) \\ \dfrac{M_{Sd}}{M_{Rd}} & \text{if } \dfrac{V_{Sd}}{V_{Rd}} < 0.4 \qquad (6.32) \end{cases}$$

$$UR_{6.3.8.3} = 0.059$$

6.3.8.4 Interaction Shear, Bending Moment and Torsional Moment

$$M_{TSd} = 4.53 \times 10^{7}$$

$\gamma_M = 1.401$

$\text{Radius} := \frac{D}{2}$ $\qquad$ $\text{Radius} = 533.4$

$f_d := \frac{f_y}{\gamma_M}$ $\qquad$ $f_d = 246.27$

$\tau_{TSd} := \frac{M_{TSd}}{2 \cdot \pi \cdot \text{Radius}^2 \cdot t}$ $\qquad$ $\tau_{TSd} = 1.424$

$f_{mRed} := f_m \cdot \sqrt{1 - 3 \cdot \left(\frac{\tau_{TSd}}{f_d}\right)^2}$ $\qquad$ $f_{mRed} = 391.22$

$M_{RedRd} := \frac{W_e \cdot f_{mRed}}{\gamma_M}$

$$UR_{6.33} := \begin{cases} \dfrac{\dfrac{M_{Sd}}{M_{RedRd}}}{\sqrt{1.4 - \dfrac{V_{Sd}}{V_{Rd}}}} & \text{if } \dfrac{V_{Sd}}{V_{Rd}} \ge 0.4 \\ \dfrac{M_{Sd}}{M_{RedRd}} & \text{if } \dfrac{V_{Sd}}{V_{Rd}} < 0.4 \end{cases}$$

$UR_{6.33} = 0.059$

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:

$\sigma_{aSd} := -39.7$	design axial stress that excludes the effect of capped-end axial compression arising from the external hydrostatic pressure (tension positive)
$\sigma_{qSd} := 0.5 \cdot \sigma_{psd}$	capped-end design axial compression stress due to external hydrostatic pressure (compression positive)
$\sigma_{mySd} := 5.9$	design in plane bending stress

$\sigma_{mzSd} := 15.2$ design out-of plane bending stress

$\sigma_{qSd} = 14.758$ $f_{hRd} = 36.726$ $f_m = 391.24$

Method A ($\sigma_{a.Sd}$ is in Tension)

a). For $\sigma_{aSd} >= \sigma_{qSd}$ (net axial tension condition)

$$\eta := 5 - 4 \cdot \frac{f_h}{f_y} \qquad \eta = 4.403 \tag{6.38}$$

$$B := \min\left(1, \frac{\sigma_{psd}}{f_{hRd}}\right) \qquad B = 0.804 \tag{6.37}$$

Design bending resistance in the presence of external hydrostatic pressure:

$$f_{mhRd} := \frac{f_m}{\gamma_M} \cdot \left(\sqrt{1 + 0.09 \cdot B^2 - B^{2 \cdot \eta}} - 0.3 \cdot B\right) \tag{6.36}$$

$$f_{mhRd} = 199.394$$

Design axial tensile resistance in the presence of external hydrostatic pressure:

$$f_{thRd} := \frac{f_y}{\gamma_M} \cdot \left(\sqrt{1 + 0.09 \cdot B^2 - B^{2 \cdot \eta}} - 0.3 \cdot B\right) \tag{6.35}$$

$$f_{thRd} = 175.828$$

$$UR_{6.34} := \left| \begin{array}{l} 0 \ \text{ if } \sigma_{aSd} < 0 \\ \left(\dfrac{\sigma_{aSd} - \sigma_{qSd}}{f_{thRd}} + \dfrac{\sqrt{\sigma_{mySd}^2 + \sigma_{mzSd}^2}}{f_{mhRd}}\right) \ \text{ if } \sigma_{aSd} > 0 \end{array} \right. \tag{6.34}$$

$$UR_{6.34} = 0$$

b). For $\sigma_{aSd} < \sigma_{qSd}$ (net axial compression condition)

$$f_{clRd} := \frac{f_{cl}}{\gamma_M} \quad (6.40) \qquad f_{clRd} = 246.27$$

$$UR_{6.39} := \frac{\left|\sigma_{aSd} - \sigma_{qSd}\right|}{f_{clRd}} + \frac{\sqrt{\sigma_{mySd}^2 + \sigma_{mzSd}^2}}{f_{mhRd}}$$

$UR_{6.39} = 0.303$

When $\sigma_{cSd} > 0.5f_{he}/\gamma_M$ and $f_{cle} > 0.5f_{he,}$ then

$$\sigma_{mSd} := \frac{\sqrt{M_{zSd}^2 + M_{ySd}^2}}{W_e}$$

$\sigma_{csd} := \sigma_{mSd} + \sigma_{qSd} - \sigma_{aSd}$ Reflect the maximum combined compressive stress

$\sigma_{csd} = 70.803$

$$UR_{6.41} := \frac{\sigma_{csd} - 0.5\cdot\frac{f_{he}}{\gamma_M}}{\frac{f_{cle}}{\gamma_M} - 0.5\cdot\frac{f_{he}}{\gamma_M}} + \left(\frac{\sigma_{psd}}{\frac{f_{he}}{\gamma_M}}\right)^2 \qquad (6.41)$$

$UR_{6.41} = 0.681$

Method B ($\sigma_{ac.Sd}$ is in Tension)

$\sigma_{acSd} := -39.7$ design axial stress that includes the effect of the capped-end compression arising from the external hydrostatic pressure (tension positive)

$$UR_{6.42} := \frac{|\sigma_{acSd}|}{f_{thRd}} + \frac{\sqrt{\sigma_{mySd}^2 + \sigma_{mzSd}^2}}{f_{mhRd}} \qquad (6.42)$$

$UR_{6.42} = 0.308$

6.3.9.2 Axial Compression, Bending, and Hydrostatic Pressure

Method A ($\sigma_{a.Sd}$ is in Compression)

$\lambda := \lambda_y$ $\lambda = 0.394$

$\xi := 1 - 0.28\cdot\lambda^2$ $\xi = 0.957$

$$f_{chRd} := \begin{cases} \left[\frac{1}{2}\cdot\frac{f_{c1}}{\gamma_M}\cdot\left(\xi - \frac{2\cdot\sigma_{qSd}}{f_{c1}} + \sqrt{\xi^2 + 1.12\cdot\lambda^2\cdot\frac{\sigma_{qSd}}{f_{c1}}}\right)\right] & \text{if } \lambda < 1.34\cdot\sqrt{\left(1 - \frac{2\cdot\sigma_{qSd}}{f_{c1}}\right)^{-1}} \quad (6.47) \\ \frac{0.9\cdot f_{c1}}{\lambda^2\cdot\gamma_M} & \text{if } \lambda \ge 1.34\cdot\sqrt{\left(1 - \frac{2\cdot\sigma_{qSd}}{f_{c1}}\right)^{-1}} \quad (6.48) \end{cases}$$

$$f_{chRd} = 225.514$$

$$f_{Ey} := \frac{\pi^2\cdot E}{\left[\left(\frac{k\cdot l_y}{i}\right)^2\right]} \qquad (6.45) \qquad f_{Ey} = 2.224 \times 10^3$$

$$f_{Ez} := \frac{\pi^2\cdot E}{\left[\left(\frac{k\cdot l_z}{i}\right)^2\right]} \qquad (6.46) \qquad f_{Ez} = 2.224 \times 10^3$$

$$UR_{6.43} := \frac{\left|\sigma_{aSd}\right|}{f_{chRd}} + \frac{1}{f_{mhRd}}\cdot\left[\left(\frac{C_{my}\cdot\sigma_{mySd}}{1 - \frac{\left|\sigma_{aSd}\right|}{f_{Ey}}}\right)^2 + \left(\frac{C_{mz}\cdot\sigma_{mzSd}}{1 - \frac{\left|\sigma_{aSd}\right|}{f_{Ez}}}\right)^2\right]^{0.5} \qquad (6.43)$$

$$UR_{6.44} := \frac{\left|\sigma_{aSd}\right| + \sigma_{qSd}}{f_{clRd}} + \frac{\sqrt{\sigma_{mySd}^2 + \sigma_{mzSd}^2}}{f_{mhRd}} \qquad (6.44)$$

$UR_{6.43} = 0.247$ $\qquad$ $UR_{6.44} = 0.303$

Method B ($\sigma_{ac.Sd}$ is in Compression)

$\sigma_{aacSd} := 39.7$ $\qquad$ design axial stress that includes the effect of the capped-end compression arising from the external hydrostatic pressure (compression positive)

$$UR_{6.50} := \frac{\sigma_{aacSd} - \sigma_{qSd}}{f_{chRd}} + \frac{1}{f_{mhRd}}\cdot\left[\left(\frac{C_{my}\cdot\sigma_{mySd}}{1 - \frac{\sigma_{aacSd} - \sigma_{qSd}}{f_{Ey}}}\right)^2 + \left(\frac{C_{mz}\cdot\sigma_{mzSd}}{1 - \frac{\sigma_{aacSd} - \sigma_{qSd}}{f_{Ez}}}\right)^2\right]^{0.5} \quad (6 - 50).$$

$UR_{6.50} = 0.181$

$$UR_{6.51} := \frac{\sigma_{aacSd}}{f_{clRd}} + \frac{\sqrt{\sigma_{mySd}^2 + \sigma_{mzSd}^2}}{f_{mhRd}} = 0.243 \qquad (6-51)$$

$$U_m := \begin{cases} \max\left(UR_{6.50}, UR_{6.51}\right) & \text{if } \sigma_{aacSd} > \sigma_{qSd} \\ UR_{6.51} & \text{if } \sigma_{aacSd} \le \sigma_{qSd} \end{cases}$$

$$U_m = 0.243$$

6.4. Tubular joints

6.4.3.1 General

Validity Range

$0.2 \le \beta \le 1.0$

$10 \le \gamma \le 50$

$30^{o} \le \theta \le 90^{o}$

$\tau \le 1.0$

$F_y \le 500 \quad \frac{N}{mm^2}$

$\frac{g_b}{D} \ge -0.6$ For K-joints

INPUT: Basic Geometric Parameters for Simple Tubular Joints

$F_y := 345$ Yield strength, chord

$F_{yb} := 345$ Yield strength, brace

$\theta := 49.6°$ Brace included angle

$g_b := 1058$ Gap between braces

$t := 22.2$ Brace wall thickness at intersection

$T_c := 53.3$ Chord wall thickness at intersection

$d := 853$ Brace outside diameter

$D := 2560$ Chord outside diameter

$A_c := \frac{\pi}{4} \cdot \left[D^2 - \left(D - 2T_c\right)^2\right] = 4.197 \times 10^5$ Chord cross sectional area

$\beta := \frac{d}{D}$ $\beta = 0.333$

$\gamma := \frac{D}{2 \cdot T_c}$ $\gamma = 24.015$

$\tau := \frac{t}{T_c}$ $\tau = 0.417$

6.4.3.3. Strength Factor Q_u

K joint only with positive gap

$$Q_{g1} := \begin{cases} 1.9 - \left(\dfrac{g_b}{D}\right)^{0.5} & \text{if } \dfrac{g_b}{T_c} \geq 2 \\ \text{"gap too small"} & \text{otherwise} \end{cases}$$

$$Q_g := \max(1, Q_{g1})$$

$$Q_g = 1.257$$

Gap factor

$$Q_\beta := \begin{cases} \dfrac{0.3}{\beta \cdot (1 - 0.833 \cdot \beta)} & \text{if } \beta > 0.6 \\ 1.0 & \text{otherwise} \end{cases}$$

(14.3-5)

(14.3-6)

$$Q_\beta = 1$$

Geometrical factor

Q_u varies with the joint and load type, as given in Table 6-3.

- **Q_u for Axial Loads**

$$Q_{u\_a} := (1.9 + 19 \cdot \beta) \cdot Q_\beta^{0.5} \cdot Q_g$$

$$Q_{u\_a} = 10.347$$

- **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_f

Q_f is a factor to account for the presence of nominal loads in the chord.

$$Z_E := \frac{\pi}{32} \cdot \left[\frac{D^4 - (D - 2 \cdot T_c)^4}{D} \right]$$ Elastic section modulus

$$Z_E = 2.577 \times 10^8$$

$P_c := -12525200$ Chord axial force

$M_{c\_ipb} := 1827400000$ Chord bending moment, in-plane bending

$M_{c\_opb} := 115418000$ Chord bending moment, out-of-plane bending

$$\sigma_{aSd} := \frac{P_c}{A_c} = -29.84$$ Design axial stress in chord

$$\sigma_{mySd} := \frac{M_{c\_ipb}}{Z_E} = 7.092$$ Design in-plane bending stress in chord

$$\sigma_{mzSd} := \frac{M_{c\_opb}}{Z_E} = 0.448$$ Design out-of-plane bending stress in chord

$\lambda_a := 0.03$
$\lambda_{ipb} := 0.045$
$\lambda_{opb} := 0.021$

INPUT the following coefficients:

Coefficients depending on joint and load type as given in Table 6-4.

$C_{1\_a} := 20$
$C_{2\_a} := 22$

$$A\_a := \left[C_{1\_a} \cdot \left(\frac{\sigma_{aSd}}{F_y} \right)^2 + C_{2\_a} \cdot \left(\frac{\sigma_{mySd}^2 + \sigma_{mzSd}^2}{1.62 \cdot F_y^2} \right) \right]^{0.5} \quad (6.55)$$

$A\_a = 0.394$

$$Q_{f\_a} := \left(1 - \lambda_a \cdot A\_a^2\right) \qquad (6.54)$$

$Q_{f\_a} = 0.995$ Chord action factor, axial force

$C_{1\_b} := 25$
$C_{2\_b} := 30$

$$A\_b := \left[C_{1\_b} \cdot \left(\frac{\sigma_{aSd}}{F_y}\right)^2 + C_{2\_b} \cdot \left(\frac{\sigma_{mySd}^2 + \sigma_{mzSd}^2}{1.62 \cdot F_y^2}\right)\right]^{0.5} \qquad (6.55)$$

$A\_b = 0.441$

$$Q_{f\_ipb} := 1.0 - \lambda_{ipb} \cdot A\_b^2 \qquad (6.54)$$

$Q_{f\_ipb} = 0.991$ Chord action factor, in-plane bending

$$Q_{f\_opb} := 1.0 - \lambda_{opb} \cdot A\_b^2 \qquad (6.54)$$

$Q_{f\_opb} = 0.996$ 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

$$N_{Rd} := \frac{F_y \cdot T_c^2}{\gamma_M \cdot \sin(\theta)} \cdot Q_{u\_a} \cdot Q_{f\_a} \qquad (6.52)$$

$N_{Rd} = 1.153 \times 10^7$ Joint design axial resistance

$$M_{Rd\_ipb} := \frac{F_y \cdot T_c^2 \cdot d}{\gamma_M \cdot \sin(\theta)} \cdot Q_{u\_ipb} \cdot Q_{f\_ipb} \qquad (6.53)$$

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