

Load Factor Study for API Recommended Practice 2A-LRFD

API TECHNICAL REPORT 2A-LFS
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Executive Summary

The motivation for this study is to evaluate the suitability of the load factors in API 2A-LRFD 1st Edition (1993) given the accumulated experience since the 1980s about hurricane hazards and platform performance. The specific objectives of this study are:

- 1) Review the methodology and analyses used in the 1980s to develop the load factor in the API 2A-LRFD 1st Edition and compare its methodology with the methodology presented in ISO 19902 (2007).
- 2) Analyze the component reliability levels with the improved understanding of hurricane hazards and platform performance and assess the effect of load factors on the component reliability.
- 3) Analyze the system reliability levels for three case study platforms that are being used to guide development of API 2A-LRFD 2nd Edition and assess the effect of load factors on the system reliability.
- 4) Provide design considerations when determining the load factors to be used in API 2A-LRFD 2nd Edition.

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Load Factor Study for API Recommended Practice 2A-LRFD

1 Introduction

1.1 Background

The American Petroleum Institute (API) published the 1st Edition of the Load and Resistance Factor Design (LRFD) guideline, i.e. API 2A-LRFD (1993), in 1993 after about 10 years development and calibration work through the 1980s. The LRFD-format was intended to improve the conventional Working Stress Design (WSD) format in the following ways (Lloyd and Karsan 1988, Moses 1981, Moses and Russell 1980, Moses 1986, and Moses and Stahl 2000): (1) produce more uniform reliability levels by using load and resistance factors that account for the relative variability in the load and resistance; (2) provide greater flexibility to incorporate new technological findings and extend existing experience to new geographic areas and new platform types; and (3) keep pace with modern code development in other engineering applications.

Although API 2A-LRFD (1993) was widely recognized and had a significant influence on the development of the International Standard ISO 19902, it was not used commonly in the United States and was withdrawn in 2012 due to a lack of maintenance. API 2A-LRFD was re-introduced in 2014 as a draft 2nd Edition in an effort to align and merge the API and ISO standards as a single global standard (Ku and Farrel 2014). The motivation for this study is to evaluate the suitability of the load factors in API 2A-LRFD (1993) given the accumulated experience since the 1980s about hurricane hazards and platform performance.

1.2 Report Contents

This report contains six chapters and two annexes.

Chapter 1 summarizes the development of the LRFD-format for design fixed offshore platforms and describes the motivation and objectives of this study.

Chapter 2 presents basic design check equations for the WSD- and LRFD-formats and compares the calibration methodologies in API 2A-LRFD 1st Edition and ISO 19902.

Chapter 3 reviews the characterization of wave load uncertainty in the development of API 2A-LRFD 1st Edition and updates this characterization based on our current understanding of hurricane hazards.

Chapter 4 assesses the reliabilities achieved in the design of components using both the WSD- and LRFD-formats and investigates the effect of the environmental and gravity load factors on the reliability levels achieved for different components.

Chapter 5 assesses the component and system reliabilities for three case study platforms (3-leg, 4-leg, and 8-leg) and investigates the effect the environmental load factor on the system reliability.

Chapter 6 presents conclusions because of this work.

Annex A presents the detailed analyses of the reliability levels of each component.

Annex B presents the assumptions, models, and input parameters for the reliability analyses of the case study platforms.

2 Review of API and ISO Methodologies

2.1 Introduction

The objective of this chapter is to review and compare the methodologies used in calibrating the load and resistance factors for API 2A-LRFD 1st Edition (1993), and in calibrating the partial action and partial resistance factors in ISO 19902.

2.2 Design Check Formats for API

Platforms design checks using API 2A-WSD and API 2A-LRFD are based on checking the loads and capacities of individual components, such as legs, joints, and braces. The design check-format for a component in the API 2A-WSD can be expressed as follows:

$$R_n \geq FS(D_n + L_n + W_n) \quad (2.1)$$

where R_n is the nominal resistance; D_n , L_n and W_n , are the nominal dead load, nominal live load, and nominal environmental load, respectively; and FS is the design factor of safety. For some component checks, the FS is not stated explicitly but implied by the ratio between the nominal strength and the allowable stress (e.g. axial tension of cylindrical members). Also, for some component checks, the nominal strength corresponds to elastic yielding rather than the ultimate strength (e.g. bending of cylindrical members). The FS values vary for different component checks.

The design check-format for a component in the API 2A-LRFD can be expressed as follows:

$$\phi_R R_n \geq \gamma_D D_n + \gamma_L L_n + \gamma_W W_n \quad (2.2)$$

where ϕ_R is the resistance factor; $\gamma_D, \gamma_L, \gamma_W$ are load factors for D_n, L_n and W_n , respectively. D_n, L_n and W_n . The determination of the nominal loads, D_n, L_n and W_n , is identical in the WSD- and LRFD-formats. However, there are some small variations in how the nominal resistance, R_n , is determined for an individual component (e.g. the strength for cylindrical members in bending is the plastic moment capacity in the LRFD-format versus the elastic yielding stress in the WSD-format). The load factors in API 2A-LRFD (1993) are $\gamma_D = 1.3, \gamma_L = 1.5$, and $\gamma_W = 1.2$ for operational conditions and $\gamma_D = 1.1, \gamma_L = 1.1$, and $\gamma_W = 1.35$ for extreme environmental conditions. The resistance factors vary for different component checks.

2.3 Calibration Methodology for API 2A-LRFD 1st Edition

The objective of the calibration work for the API 2A-LRFD 1st Edition was to achieve, on average, the same component reliability levels for the LRFD-format (Equation 2.2) as obtained using the WSD-format (Equation 2.1). In this work, the event of failure for a component was defined as the load exceeding the ultimate capacity obtained using the design equations in the API standard. The reliability level was represented by the probability of failure of the component in a 20-year design life (the smaller the probability of failure, the greater the reliability level). The results were presented using a reliability index, β , which is inversely related to the probability of failure (Figure 2.1).

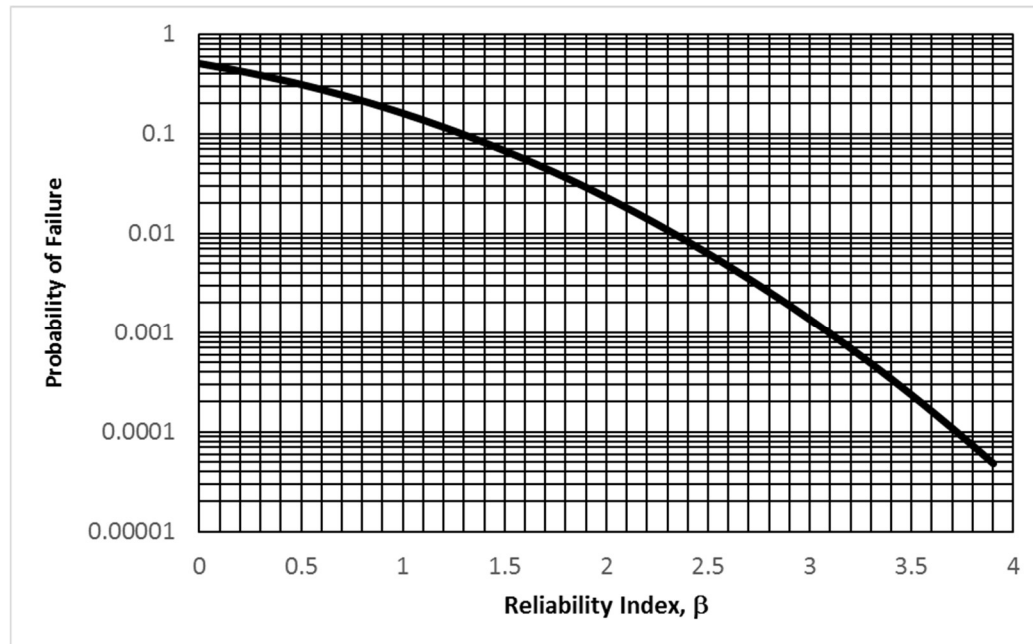


Figure 2.1—Relationship between Probability of Failure and Reliability Index

In order to achieve a comparable reliability with the LRFD-format as with the WSD format, the following steps were carried out (Moses 1981, Moses 1986, Moses and Russel 1980, Moses and Stahl 2000):

- 1) Review the environmental statistics and existing studies on the wave climate in the Gulf of Mexico to establish a random load model.
- 2) Review the available test database to assess the resistance uncertainty for each key component.
- 3) Compute the 20-year reliability index, β_{20} , for each component based on the WSD-format for a range of environmental to gravity (W/G) ratios and calculate the average β_{20} implied by the WSD-format for W/G ratios from 2 to 40. The average values for β_{20} were about 2.7 to 2.8 (i.e. a probability of failure in a 20-year design life of about 0.003).
- 4) Take the average β_{20} from the WSD-format as the target reliability level and propose candidate load and resistance factors to meet the target reliability level for the LRFD format.
- 5) Study the sensitivity with respect to the uncertainties in the load and resistance to check the robustness of the proposed load and resistance factors.

Early in the calibration work (Moses, 1982), the environmental load factor, γ_w , was fixed at 1.35. This value was established through sensitivity studies to provide reasonably uniform reliability levels over a range of possible environmental load characterizations (i.e. the parameters describing the random load model). Once the environmental load factor was fixed, the remaining calibration work focused on establishing the resistance factors for all the different component design checks.

2.4 Design Check Format for ISO 19902

The design checks in ISO 19902 for components follow a similar format to API 2A-LRFD, except that partial resistance factors, which are the inverse of API resistance factors, are used:

$$R/\gamma_R \geq \gamma_{f,G}G + \gamma_{f,Q}Q + \gamma_{f,E}E \quad (2.3)$$

where R is the nominal (or characteristic) capacity of a component; γ_R is a partial resistance factor; G , Q , and E are nominal dead, live and environmental loads, respectively; and $\gamma_{f,G}$, $\gamma_{f,Q}$, and $\gamma_{f,E}$ are partial load (or action) factors. While similar in format for component design checks, there are three differences with the environmental load between ISO 19902 and API 2A-LRFD 1st Edition:

The partial action factor for environmental loads in ISO 19902, $\gamma_{f,E}$, can vary with geographic location since this standard applies to offshore environments worldwide and not only the Gulf of Mexico.

The partial action factor for environmental loads in ISO 19902 can vary with exposure level, which reflects the consequences of a platform failure.

The partial action factor for environmental loads can be increased if the reserve strength for the platform system is less than for a typical space frame type structure.

2.5 Calibration Methodology for ISO 19902

Different mean and c.o.v. values were used for the steel yield strength bias factor in earlier phases of the calibration work. For a structural component subjected to yielding under axial tensile loading, a mean of 1.1 and a c.o.v. of 0.13 were used by Moses (1981) and Moses and Larrabee (1988). For a structural component subjected to compressive and bending loads and tubular joints, a mean value 1.05 and a c.o.v. of 0.05 were used by Moses (1981).

Moses (1999) essentially used the same methodology for the calibration of ISO partial action factors and partial resistance factors ¹ as for the calibration of API 2A-LRFD 1st Edition. The uncertainty in the environmental action was based on Efthymiou et al. (1997), and the component strength statistical parameters were recommended by the ISO committee in addition to the fabrication, material, and geometric variability. The following are the key steps in the calibration (Moses 1999, and Moses and Stahl 2000):

- 1) Fix the partial action factor for environmental loads to be 1.35 for the Gulf of Mexico location, and determine the partial resistance factor to achieve a target average value for the 20-year reliability index similar to what is achieved in the API 2A-LRFD 1st Edition calibration. The final set of partial resistance factors in ISO 19902 is essentially based on a target $\beta_{20} = 2.5$, with some modifications for columns in compression and tension and for hydrostatic loading to account for differences in the severity of consequences associated with exceeding ultimate capacity.

¹ Moses (1999) and Moses and Stahl (2000) used the terms “load factor” and “resistance factor” for the ISO factors. “Partial action factor” and “partial resistance factor” are used here in order to be consistent with the current terminologies in ISO 19902.

- 2) Keep the partial resistance factors calibrated for the Gulf of Mexico the same for other locations, and vary the environmental action factor for each geographic region to achieve the same target average value for the 20-year reliability index as is achieved in the Gulf of Mexico. For example, Moses and Stahl (2000) recommended partial action factors for environmental load of 1.18 for the central and southern North Sea, 1.25 for the northern North Sea and 1.36 for north-east shelf of Australia.

Following this calibration work, the default partial action factor for environmental load presented in the Informative Annex of ISO 19902 (2007) for the highest exposure level (L1) is the same as for the Gulf of Mexico, 1.35. However, the Informative Annex also includes values of the partial action factor for environmental load for other geographic regions (the North Sea and Australia) and for other exposure levels (e.g. a partial action factor of 1.17 is presented for an L2 exposure level in the Gulf of Mexico). The values presented are based on system reliability analyses (i.e. system failure is defined as a reserve strength ratio less than one for the platform) conducted by Van de Graaf et al. (1994). They used target probabilities of system failure of 3×10^{-5} per year for an L1 exposure level and 5×10^{-4} per year for an L2 exposure level. These partial environmental action factors are about 0.15 higher than those obtained by Moses and Stahl (2000) due to the different reliability models (component versus system based) and different target reliability levels.

2.5 Comparison of API LRFD and ISO Calibration Methodologies

Comparing the calibration for the API load and resistance factors and the calibration for the ISO partial action and partial resistance factors, several observations are listed as follows:

- 1) The definition of the partial resistance factor γ_R (which is generally greater than 1.0) in ISO 19902 is the inverse of the resistance factor ϕ_R (which is generally less than 1.0) in the API 2A-LRFD 1st Edition.
- 2) The load and resistance factors in the API 2A-LRFD 1st Edition were calibrated based on the inferred reliability levels in the WSD-format (i.e. the average β_{20} is about 2.7 to 2.8), while the partial action and resistance factors in ISO 19902 were calibrated based on a target β_{20} of 2.5 with a fixed environmental action factor of 1.35 for the Gulf of Mexico (with slight modifications for some components).
- 3) The calibration of the API 2A-LRFD 1st Edition was based on the Gulf of Mexico; hence, the direct application of the API 2A-LRFD 1st Edition in other locations is expected to reach different reliability levels. For other locations, Moses and Stahl (2000) used a component based calibration for the ISO environmental action factor with the same target reliability level in the Gulf of Mexico, while a system reliability approach with different target reliability levels for different exposure levels is included in the informative annex in ISO 19902.
- 4) The recommended partial resistance factor for column compression in ISO 19902 is 1.18 which is different from the suggested value of 1.10 by Moses (1999). The partial resistance factor for tension is decreased from 1.13 (with a target $\beta_{20} = 2.5$) to 1.05 in ISO 19902 since the column under tension has considerable strength after first yielding.
- 5) The partial resistance factor for hydrostatic loading is increased from 1.08 (with a target $\beta_{20} = 2.5$) to 1.25 in ISO 19902 due to the severe consequence of hydrostatic failure. However, the resistance factor for hydrostatic loading is increased from 0.75 by Moses (1986) to 0.80 in API 2A-LRFD (1993) presumable due to the high reliability of hydrostatic loading comparing to other components.

2.6 Summary

The chapter presents the format of the component design check equations for the API WSD- and LRFD-formats and summarizes the methodology used to calibrate the load and resistance factors in API 2A-LRFD 1st Edition. The load and resistance factors in API 2A-LRFD 1st Edition were calibrated to achieve a component reliability level that was comparable to that achieved with the WSD format; this reliability level corresponds to a 20-year probability of failure for a component that is about 0.003 in the Gulf of Mexico. This same approach was then used to calibrate comparable resistance and load (action) factors in ISO 19902 (2007) for the Gulf of Mexico.

3 Wave Load Uncertainty

3.1 Introduction

The objectives of this chapter are: (1) to describe the method used to represent wave load uncertainty in the 1980's calibration of the API 2A-LRFD 1st Edition and (2) to update this method based on information developed since that time.

3.2 Wave Load Uncertainty in API 2A-LRFD 1st Edition

The 1980's calibration of the API 2A-LRFD 1st Edition (Moses 1981, Moses and Russell 1980, and Moses 1986) modeled the wave load uncertainty as follows:

$$F_{w,20} = B_{w,20} W_n \quad (3.1)$$

where $F_{w,20}$ is the maximum global base shear force applied to the platform at the mudline in a platform lifetime of 20 years; W_n is the nominal design load due to waves, i.e. the 100-year wave load used in design; and $B_{w,20}$ is a wave load bias factor, which is a random variable to account for uncertainty in the maximum wave force over a 20-year lifetime relative to the nominal design load. The wave load bias factor, $B_{w,20}$, was modeled as a lognormal distribution with a mean value of 0.7, $\bar{B}_{w,20}=0.7$, and a coefficient of variation (c.o.v.) value of 0.37, $V_{w,20}=0.37$.

The mean value and c.o.v. for $B_{w,20}$ were established using the following simplified model to relate the wave height to the wave force on the platform (Moses 1986):

$$F_{w,20} = A c_1 (H_{20})^\alpha \quad (3.2)$$

and

$$B_{w,20} = \left(\frac{1}{W_n} \right) F_{w,20} = \left(\frac{1}{W_n} \right) A c_1 (H_{20})^\alpha \quad (3.3)$$

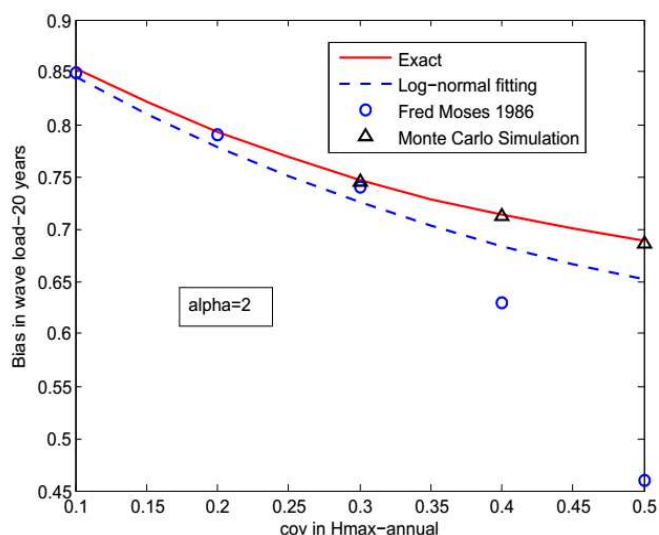
where H_{20} is a random variable representing the maximum wave height in a 20-year lifetime, c_1 and α are deterministic parameters relating the wave height to the base shear force for a particular member (α is close to 2 for typical members where the wave force is dominated by drag); and A is a wave load bias factor, which is a random variable to account for uncertainty in the force for a given wave height. Since the nominal wave force, is W_n obtained from a deterministic version of Equation 3.3, $W_n = c_1 (h_n)^\alpha$ where h_n is the nominal (100-year) wave height, Equation 3.3 can be simplified as follows:

$$B_{w,20} = \left(\frac{1}{h_n} \right)^\alpha A(H_{20})^\alpha \quad (3.4)$$

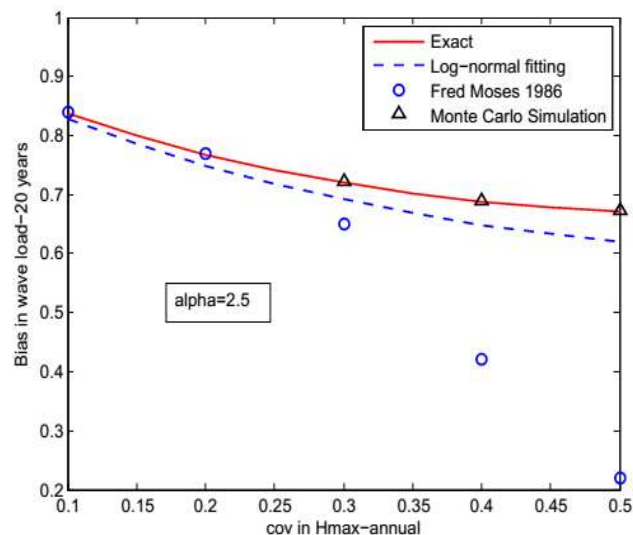
The distribution for H_{20} was approximated by: (1) assuming that the maximum annual wave height has a lognormal distribution with a c.o.v. value V_H (note that the median for this lognormal distribution is tied directly to the 100-year wave height, h_n , which is the 99th percentile value of the maximum annual wave height); (2) obtaining the cumulative distribution function for the maximum wave height in 20 years by raising the cumulative distribution function for the annual maximum wave height to the 20th power: $CDF(H_{20}) = CDF(H)^{20}$, where H is the annual maximum wave height; and (3) fitting an approximate lognormal distribution to the cumulative distribution function for the maximum wave height in 20 years by equating the average of the 10th and 90th percentile values (expressed as natural logarithms) for the exact and approximate distributions (Moses and Russell 1980). The distribution for A was assumed to be a lognormal distribution with a mean value of 0.93, $\bar{A} = 0.93$, and a c.o.v. value of 0.25, $V_A = 0.25$, based on load and wave height measurements at Offshore Test Structure (Haring et al. 1979).

Example results for the mean value and c.o.v. for $\beta_{w,20}$ using the (Moses 1986) approach illustrate that the mean ($\bar{\beta}_{w,20}$) decreases with increasing α and with increasing V_H (Figure 3.1) and that the c.o.v. ($V_{w,20}$) (increases with increasing α and with increasing V_H (Figure 3.2). There are several results shown in Figures 3.1 and 3.2 to confirm what Moses (1986) did and to check the significance of approximating the probability distribution for the 20-year maximum wave height as a lognormal distribution. When we use the same approach as that described by Moses (1986), our results (“Log-normal fitting” in Figures 3.1 and 3.2) generally match well with his results. The notable exception is an apparent discrepancy in the results by Moses (1986) for $\bar{\beta}_{w,20}$ with V_H values of 0.4 and 0.5 (Figure 3.1). We suspect that Table 2.2 in the report by Moses (1986) contains an error for these values; this error does not impact the calibration work because the results in the table were presented only for illustration. The “Exact” and “Monte Carlo Simulation” results in Figures 3.1 and 3.2 correspond to an exact analytical solution and a corresponding numerical approximation for the probability distribution of the 20-year maximum wave height. The lognormal fit used by Moses (1986) provides a reasonable approximation, particularly for V_H values less than 0.3 (Figures 3.1 and 3.2).

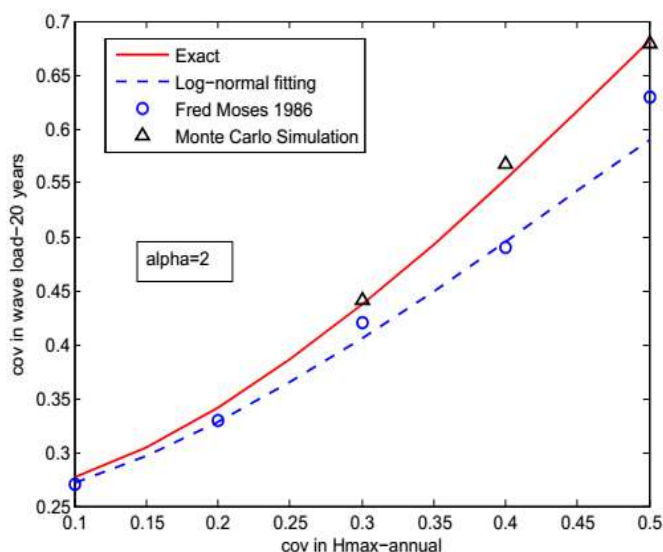
The final parameters selected to represent uncertainty in the wave load in the calibration work, $\bar{\beta}_{w,20} = 0.7$ and $V_{w,20} = 0.37$ were obtained from a site-specific reliability study performed by Anderson et al. (1982). This study was for an 8-leg platform with 16 piles in Mississippi Canyon Block 268 in a water depth of 343 ft. These values of $\bar{\beta}_{w,20}$ and $V_{w,20}$ used in the calibration work correspond approximately to typical α values between 2.0 and 2.5 and typical V_H values between 0.2 and 0.3 (Figures 3.1 and 3.2).



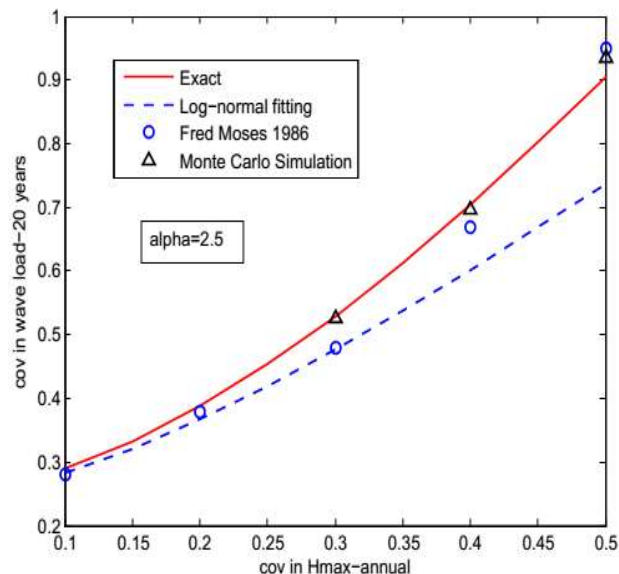
(a)



(b)

Figure 3.1—Comparison of $\bar{B}_{w,20}$: (a) $\alpha = 2.0$; (b) $\alpha = 2.5$ 

(a)



(b)

Figure 3.2—Comparison of $V_{w,20}$: (a) $\alpha = 2.0$; (b) $\alpha = 2.5$

3.3 Updated Representation of Wave Load Uncertainty

The representation of wave load uncertainty from Moses (1986) is updated based on developments and information obtained since that time. First, the model is adapted to reflect more recent approaches for modeling uncertainty in wave loads. Second, the model is calibrated with information about observed and predicted platform performance in the Gulf of Mexico obtained from recent hurricanes. Third, the model is applied using new information about the probability distribution of wave heights contained in API 2MET (2014).

3.3.1 Model for Wave Load Uncertainty

It is common practice in the Gulf of Mexico (Anderson et al. 1982, PMB 1993 and 1996, ABS 2004, and Energo 2005 and 2007) to divide the wave load bias factor in Equation 3.4, the A term, into the product of a random variable that accounts for a systematic modeling bias (i.e. a bias that applies to all waves), A_{model} , and a random variable that accounts for random (or aleatory) variability in the wave load bias (i.e. a bias that varies randomly from wave-to-wave), $A_{aleatory}$: $A = A_{model} A_{aleatory}$. The A_{model} term accounts for systematic variations from the wave load recipe, such as errors in estimating hydrodynamic coefficients (e.g. Anderson et al. 1982). The $A_{aleatory}$ term accounts for independent wave-to-wave variations in the wave load recipe, such as variations in directional spreading from wave-to-wave (e.g. Jonathan and Taylor 1996). Moses (1986) assumed that $A_{aleatory} = 1$ and $A = A_{model}$. However, the measured statistics for A from Haring et al. (1979), $\bar{B}_A = 0.93$ and $V_A = 0.25$, were developed by comparing the predicted and measured wave forces from a number of individual waves and included contributions both from A_{model} and $A_{aleatory}$.

Accounting for random variations in the load bias from wave-to-wave is known as the wave-by-wave approach (e.g. Efthymiou et al. 1997). The difficulty with the wave-by-wave approach is that both the wave height and the aleatory portion of the wave load bias vary from wave-to-wave during a hurricane. Therefore, the maximum load in a sea state is not simply obtained directly from the maximum wave height in the sea state. Figure 3.3 illustrates the effect of random variations in the wave load bias on the probability distribution for the maximum base shear during a 3-hour sea state in Hurricane Andrew (see Chen 2016 for details). The results labeled “Sea state approach” assume that the only variations during the sea state are due to the variations in wave height, while the results labeled “Wave-by-wave approach” also account for random variations in the wave load bias factor from wave-to-wave with $(\bar{A}_{aleatory})_{singlewave} = 1$ and $(V_{A_{aleatory}})_{singlewave} = 0.2$. The effect of the random variations in the wave

load bias over the approximately 1000 waves during this sea state is to increase the maximum base shear (i.e. to shift the probability distribution to the right in Figure 3.3); this increase in the maximum load reflects the possibility that the maximum load during the sea state could come from a smaller wave height with a larger wave load bias. However, the additional variation in the load bias from wave-to-wave has a negligible effect on the variability in the maximum load over the sea state (i.e. the shapes of the curves for the “Sea state approach” and “Wave-by-Wave approach” are nearly identical in Figure 3.3); this negligible impact of the aleatory variability in the wave load bias on the variability in the maximum load over the sea state is because the aleatory variability in the wave load bias is significantly smaller than the variability in the square of the wave heights during a sea state. An approximation to account for the effect of $A_{aleatory}$ is to multiply the probability distribution obtained from considering only random variations in the wave height during the sea state by a deterministic aleatory peak factor, ψ_{sw} (Figure 3.3).

The net result of applying a simplified aleatory peak factor to the wave-by-wave approach is an effective wave load bias factor on the maximum load during the peak sea state with $(\bar{A}_{aleatory})_{sea\ state} = \psi_{sw}$ and $(V_{A_{aleatory}})_{sea\ state} = 0$.

Furthermore, since it is unlikely to have multiple peak sea states from different hurricanes that all have large enough waves to produce the maximum force at a particular platform location in a single year, the effective wave load bias factor on the annual maximum load and on the maximum load during the lifetime is essentially the same as that for the peak sea state: $A_{aleatory}$ in Equation 3.4 has $\bar{A}_{aleatory} = \psi_{sw}$ and $V_{A_{aleatory}} = 0$, or $A = A_{model} \psi_{sw}$.

Therefore, the model used by Moses (1986) can be extended to account for the random variability in the wave load bias factor as follows:

$$B_{w,20} = \left(\frac{1}{h_n} \right)^\alpha A_{model\psi_{sw}} (H_{20})^\alpha \quad (3.5)$$

where the aleatory peak factor, ψ_{sw} , depends on the c.o.v. value for the random variations in the wave load bias factor from wave-to-wave, $\left(V_{Aleatory}\right)_{singlewave}$. The aleatory peak factor was assessed by Chen (2016) for 18 Gulf of Mexico platforms as a function of $\left(V_{Aleatory}\right)_{singlewave}$, producing the best-fit curve shown in Figure 3.4; note that the results ψ_{sw} for individual platforms were all within several percent of this best-fit curve. A typical value of $\left(V_{Aleatory}\right)_{singlewave}$ is 0.2 for waves not impacting the deck levels (e.g. Jonathan and Taylor 1996, ABS 2004, and Energo 2005), giving $\psi_{sw} = 1.18$.

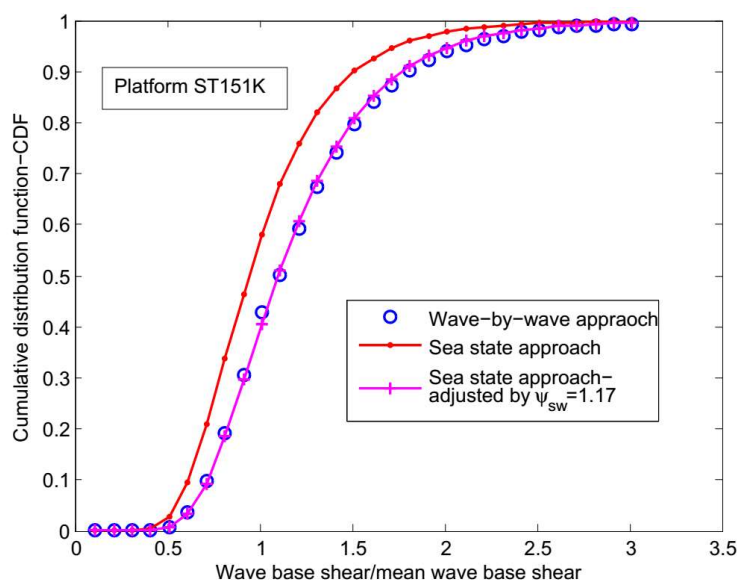


Figure 3.3—Comparison of Probability Distribution for the Maximum Wave Load for Wave-by-Wave and Sea State Approaches during a 3-Hour Sea State

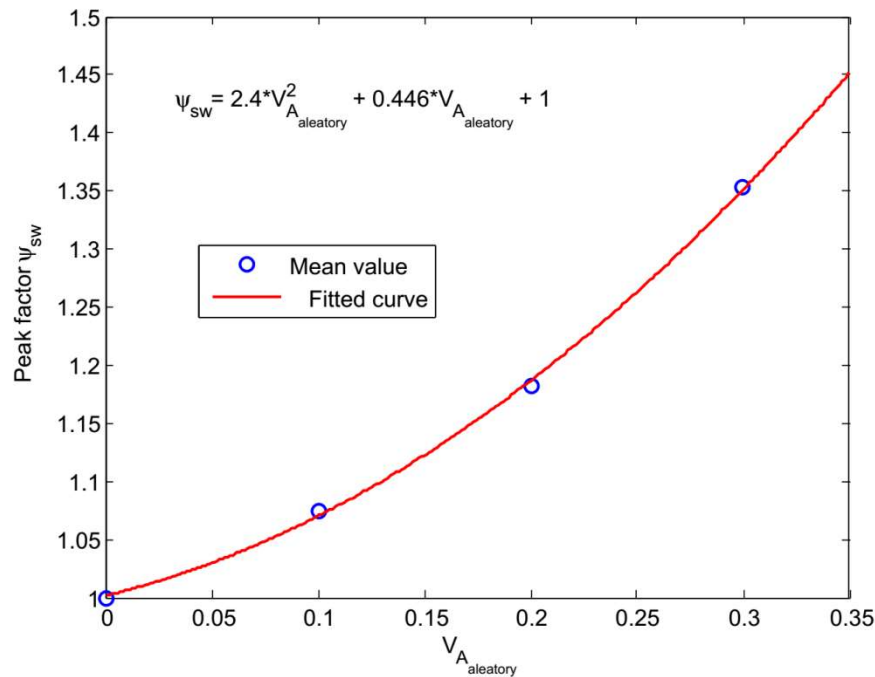


Figure 3.4—Relationship between Aleatory Peak Factor, ψ_{sw} , and C.O.V. Value for Random Variations in Wave Load Bias Factor from Wave-to-Wave during a Sea State (Chen 2016)

3.3.2 Updated Model Uncertainty Based on Hurricane Performance

There have been a handful of major hurricanes in the Gulf of Mexico since Moses (1986) that provide additional information about wave loads on platforms: Andrew (1992), Lili (2002), Ivan (2004), Katrina (2005), Rita (2005), Gustav (2008) and Ike (2008). The model uncertainty in the wave load recipe (i.e. A_{model} in 3.5) has been calibrated with comparisons of observed and predicted performance after each hurricane (PMB 1993, ABS 2004, Energo 2005 and 2007 and 2010). The most recent calibration work, which incorporates all these earlier studies, was performed by Chen (2016).

Based on a comparison of analytically predicted and actually observed performance of 18 offshore platforms (both jacket structures and pile systems) in a Bayesian framework, Chen (2016) calibrated A_{model} in Equation 3.5 to have a mean $\bar{A}_{model} = 0.92$ and a c.o.v. $V_{A_{model}} = 0.13$. For the purposes of comparing with the model uncertainty in the wave load recipe used by Moses (1986), A in Equation 3.4, the updated value for $\bar{A} = A_{model}\psi_{sw}$ is 1.08 (compared to the $\bar{A} = 0.93$ used by Moses) and the updated value for V_A is 0.13 (compared to the $V_A = 2.5$ used by Moses). Therefore, the mean value for the model uncertainty increased by about 16 % (primarily due to accounting for the possibility of random variations in the wave load bias from wave-to-wave) and the c.o.v. value for the model uncertainty decreased by about 50 % (due both to the averaging effect of the portion of the model uncertainty that varies from wave-to-wave and the new information from the 18 platforms subjected to large hurricane forces).

3.3.3 Variability in Annual Maximum Wave Heights from API 2MET (2014)

API 2MET (2014) provides the probability distribution for the annual maximum wave in each region of the Gulf of Mexico (Figure 3.5) in terms of maximum annual wave height versus return period (Figure 3.6). The probability distribution for H is taken directly from the return-period curves in API 2MET (2014): $CDF(H) = 1 - 1/\bar{T}(H)$ where $\bar{T}(H)$ is the return period for a given value of H . For example, the 100-year wave height in a water depth of 295 ft in the Central Region is 82 ft (Figure 3.6), giving $\bar{T}(H = 82 \text{ feet}) = 100 \text{ years}$ and $CDF(H = 82 \text{ feet}) = 0.99$. For a return period less than 10 years or greater than 1000 years, $CDF(H)$ is extrapolated by two-parameter Weibull

distributions fitted at the return periods of 10 to 15 years and 500 to 1000 years, respectively. The probability distribution for the maximum wave height in 20 years, H_{20} in Equation 3.5, is obtained by assuming independence in the maximum wave heights between years as in Moses (1986): $CDF(H_{20}) = CDF(H)^{20}$.

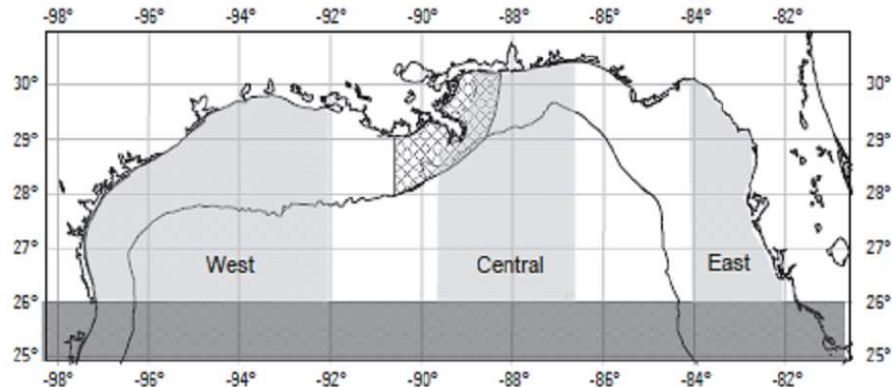


Figure 3.5—Gulf of Mexico Regions in API 2MET (2014)

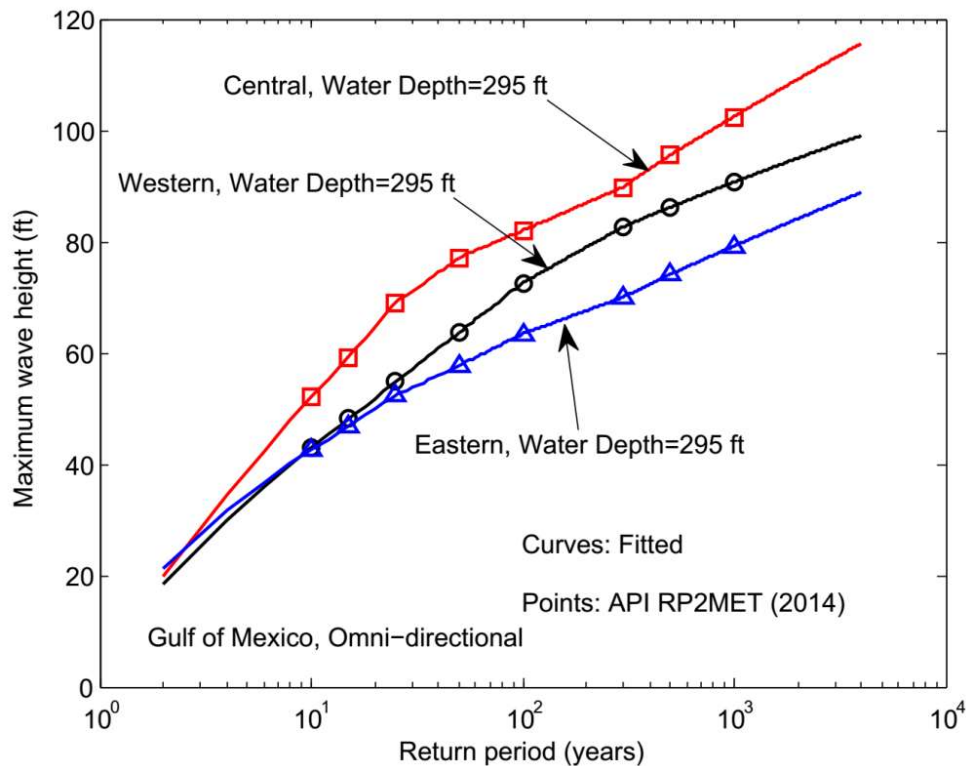


Figure 3.6—Maximum Annual Wave Height Versus Return Period from API 2MET (2014)

3.4 Comparison of Previous and Updated Representations of Wave Load Uncertainty

In order to compare the updated and previous representations of wave load uncertainty, an approximate lognormal distribution is fitted to the exact distribution in the updated representation for the maximum wave height in 20 years, H_{20} , using the same approach as Moses (1986): the median and the average of the 10th and 90th percentile values (expressed as natural logarithms) are equated for the approximate and exact distributions. Example results for the exact and approximate distributions for the updated representation of wave load uncertainty are shown in Figure 3.7. This approximate lognormal fit for H_{20} is then input into Equation 3.5 to obtain equivalent mean and c.o.v. values for $B_{w,20}$, $\bar{B}_{w,20}$, and $V_{w,20}$, that are comparable to Moses (1986).

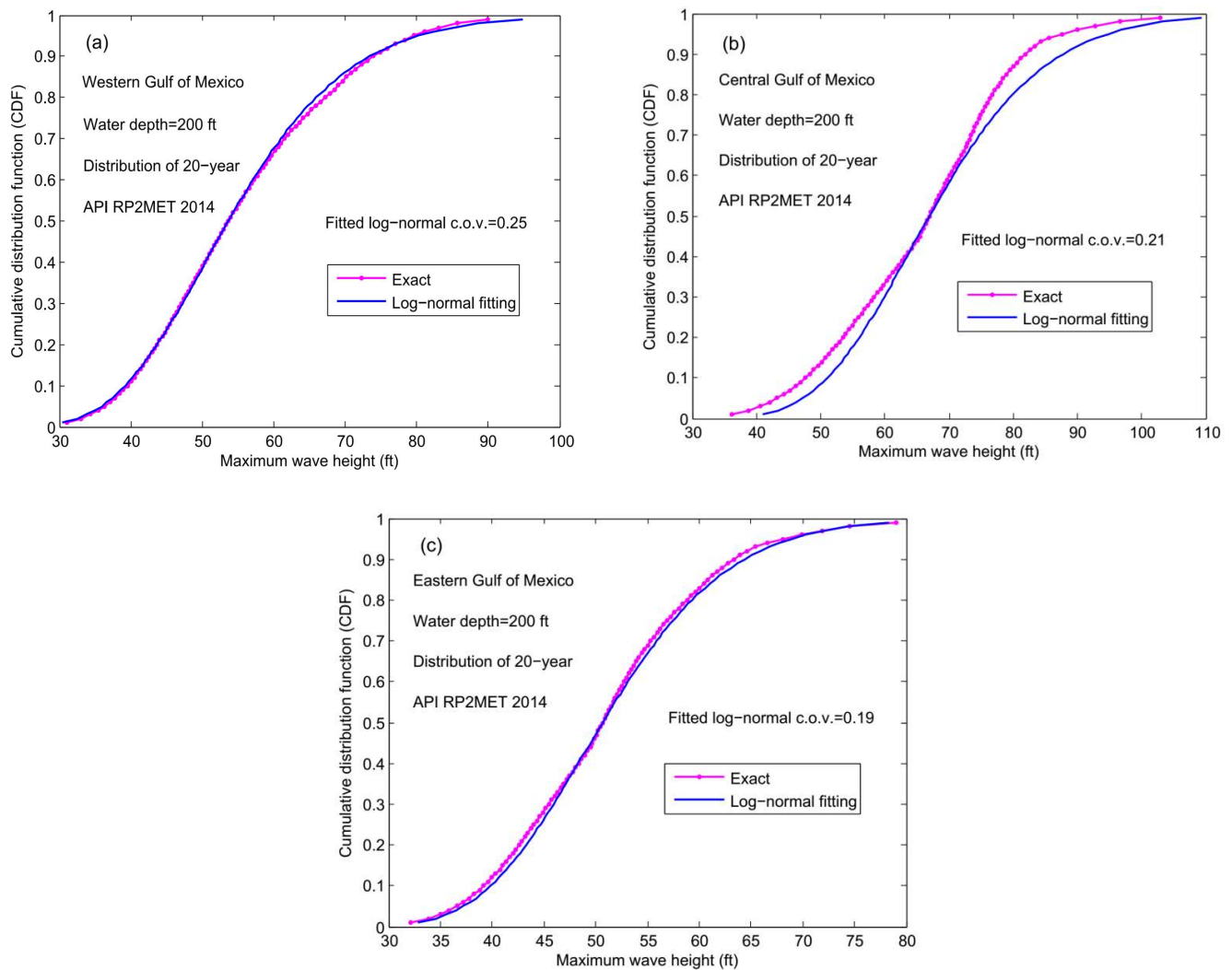


Figure 3.7—Comparison of “Exact” and Fitted Distribution for the Maximum Wave Height in 20 Years, H_{20} using Updated Representation of Uncertainty in Wave Loads

A comparison of $\bar{B}_{w,20}$ and $V_{w,20}$ is shown in Figures 3.8 and 3.9 for different water depths and different values of α . Both $\bar{B}_{w,20}$ and $V_{w,20}$ are relatively insensitive to water depth in the updated representation of wave load uncertainty (Figures 3.8 and 3.9). $\bar{B}_{w,20}$ decreases by about 0.04 with an increase of the wave height exponent α by 0.5, while $V_{w,20}$ increases by about 0.12 with an increase of α by 0.5.

For $\alpha=20$, $B_{w,20}$, $\bar{B}_{w,20}$, and $V_{w,20}$, are greater than those used by Moses (1986) (Figures 3.8 and 3.9, and Table 3.1). Therefore, a lower reliability level is expected with the updated representation of wave load uncertainty using the most recent version of metocean design criteria than the reliability obtained in the calibration of API 2A-LRFD 1st Edition in the 1980s.

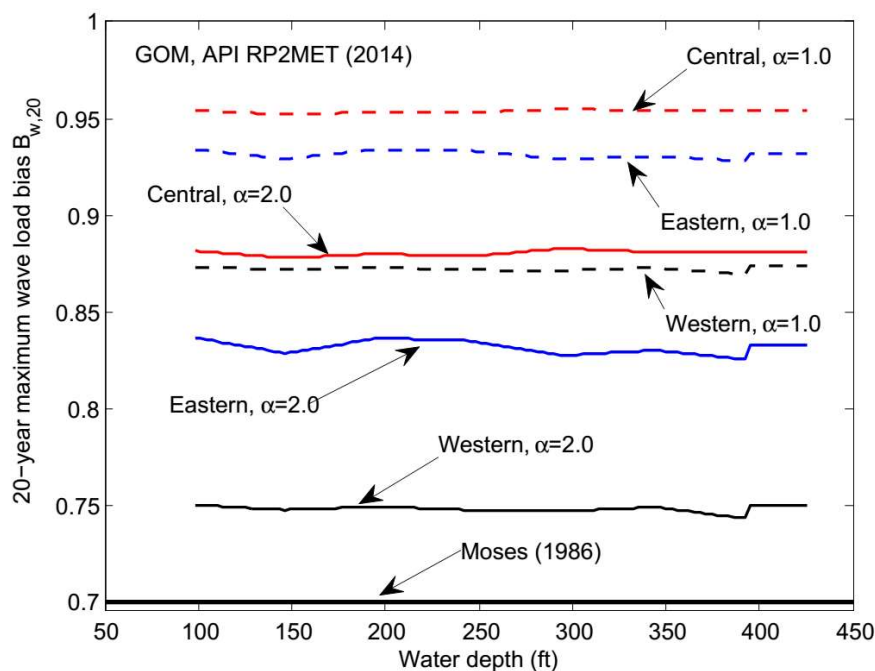


Figure 3.8—Mean Value for Maximum Wave Height in 20-year Lifetime Relative to the Nominal (100-year) Design Load for Updated Representation of Wave Load Uncertainty

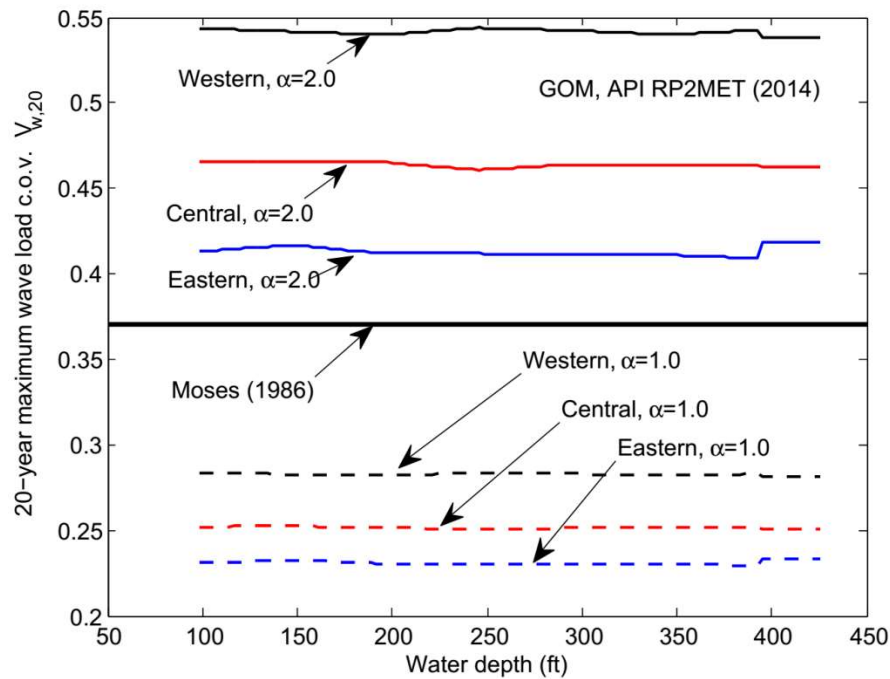


Figure 3.9—C.O.V. for Maximum Wave Height in 20-year Lifetime Relative to the Nominal (100-year) Design Load for Updated Representation of Wave Load Uncertainty

Table 3.1—Summary on $\bar{B}_{w,20}$ and $V_{w,20}$, Averaged over Water Depths from 100 ft to 450 ft, for Updated and Previous Representations of Wave Load Uncertainty

Gulf of Mexico Location	Moses (1986)		API 2MET (2014)							
			$\alpha = 1.0$		$\alpha = 1.5$		$\alpha = 2.0$		$\alpha = 2.5$	
	$B_{w,20}$	$V_{w,20}$	$B_{w,20}$	$V_{w,20}$	$B_{w,20}$	$V_{w,20}$	$B_{w,20}$	$V_{w,20}$	$B_{w,20}$	$V_{w,20}$
West	0.70	0.37	0.87	0.28	0.80	0.41	0.75	0.54	0.71	0.69
Central			0.95	0.25	0.91	0.35	0.88	0.46	0.86	0.59
East			0.93	0.23	0.88	0.32	0.83	0.41	0.80	0.52
Average			0.92	0.25	0.86	0.36	0.82	0.47	0.79	0.60

3.5 Summary

The representation of wave load uncertainty used in the calibration of API 2A-LRFD 1st Edition in the 1980s is updated based on developments and information obtained since that time: (1) the load model is adapted to reflect more recent approaches for modeling uncertainty in wave loads; (2) the model is calibrated with information about observed and predicted platform performance in the Gulf of Mexico obtained from recent hurricanes; and (3) the model is applied using new information about the probability distribution of wave heights contained in [API 2MET \(2014\)](#). In the updated representation, both the mean value and c.o.v. value in the maximum wave load bias (relative to the nominal 100-year load) in a 20-year lifetime are relatively insensitive to the water depth in all regions of the Gulf of Mexico. The updated mean and c.o.v. values for the maximum wave load bias are both greater than those used in the original calibration for API 2A-LRFD 1st Edition, meaning that a lower reliability level is expected with the updated representation of wave load uncertainty than the reliability obtained in the original calibration.

4 Evaluation of Design Component Reliability

4.1 Introduction

The design guidance for fixed offshore platforms is based on a component check-format. Therefore, the calibration work for API 2A-LRFD 1st Edition was based on achieving the same average component reliability levels for the LRFD-format design check as for the WSD-format design check. The objectives of this chapter are to: (1) assess the component reliability for both the WSD- and LRFD-formats based on updated information about wave loads and component capacities (denoted the “updated analysis” to distinguish it from the original calibration work); and (2) assess the sensitivity of the component reliability to the load and resistance factors.

4.2 Component Reliability Model

The event of failure for a component is defined as the load exceeding the capacity as defined by the design equations in the API standard. The reliability level is represented by the probability of failure of the component in a 20-year design life (the smaller the probability of failure, the greater the reliability level).

4.2.1 Performance Function

The event of failure for a component is defined using the following performance function:

$$g = B_R R_n - [B_W W_n + (B_D D_n + B_L L_n)] \quad (4.1)$$

where $g \leq 0$ indicates the safe region and $g > 0$ indicates the failure region; B_W is the bias factor for the maximum wave load in 20 years; W_n is the nominal environmental load (i.e. the load obtained using the 100-year wave height with associated wind and current); B_D and B_L are bias factors for the dead load and live load, respectively; D_n and L_n are the nominal dead load and live load, respectively; B_R is a bias factor for the ultimate capacity of the component; and R_n is the nominal capacity calculated using the design check equation.

4.2.2 Component Loads

The nominal dead load and live loads are related to each other by assuming that the ratio of the nominal live load to the nominal dead load, L_n/D_n , is equal to three following Moses (1986). Therefore, the performance function (Equation 4.1) is simplified as follows:

$$g = B_R R_n - [B_W W_n + (B_D D_n + B_L 3D_n)] \quad (4.2)$$

Also following Moses (1986), the bias factors for dead and live loads are both assumed to be lognormally distributed, the dead load bias factor has a mean value of 1.00 and a c.o.v. value of 0.08, and the live load bias factor has a mean value of 1.00 and a c.o.v. value of 0.14 (Tables 4.1 and 4.2).

Table 4.1—Random Variables Describing Component Loads—Moses (1986)

Random Variable	Description	Distribution Type	Mean	c.o.v.
B_D	Dead load bias – structural weight	Lognormal	1.00	0.08
	Dead load bias – hydrostatic pressure	Lognormal	1.00	0.05
B_L	Live load bias	Lognormal	1.00	0.14
$B_W = B_{W,20}$	20-year maximum wave load bias	Log-normal	0.70	0.37

Table 4.2—Random Variables Describing Component Loads—Updated Analysis

Random variable	Description	Distribution type	Mean	c.o.v.
B_D	Dead load bias – structural weight	Lognormal	1.00	0.08
	Dead load bias – hydrostatic pressure	Lognormal	1.00	0.05
B_L	Live load bias	Lognormal	1.00	0.14
$B_W = B_{W,20}$	20-year maximum wave load bias	$B_{W,20} = \left(\frac{1}{h_n}\right)^\alpha A_{model} \psi_{sw} (H_{20})^\alpha$ with $\psi_{sw} = 1.18$		
A_{model}	Wave load systematic model bias factor	Lognormal	0.92	0.13
H_{20}	Maximum wave height in 20 years	$CDF(H_{20}) = CDF(H)^{20}$		
H	Maximum annual wave height	Taken from annual maximum wave height versus return period in API 2MET (2014) ; h_n , is annual maximum wave height with 100-year return period.		

The environmental load on every component is assumed to be linearly proportional to the global base shear due to wave loading on the platform following Moses (1986). In this case, $B_W = B_{w,20}$ where $B_{w,20}$ is a lognormal variable with a mean of 0.70 and a c.o.v. of 0.37 for the original calibration work (Table 4.1) and is obtained from Equation 3.5 for the updated analysis (Table 4.2). The nominal environmental load, W_n , is taken to be proportional to the nominal gravity load:

$$W_n = \frac{W}{G}(D_n + L_n) = \frac{W}{G}(D_n + 3D_n) \quad (4.3)$$

where $\frac{W}{G}$ is the ratio of environmental-to-gravity load. Values of $\frac{W}{G}$ ranging from 0.01 to 40 are considered.

4.2.3 Nominal Component Resistance

The nominal component resistance, R_n , is established using the design check equations in the WSD-format (Equation 2.1) and LRFD-format (Equation 2.2). The nominal resistance is taken as the maximum resistance required from checks for both operational and extreme environmental conditions.

Two different versions of WSD are used: (1) a version that reflects the design check equations in use when API 2A-LRFD 1st Edition was calibrated ([API 2A-WSD, 2000](#)) and (2) a version that reflects the most recent design check equations ([API 2A-WSD, 2014](#)). The difference in the two WSD versions is for tubular joints; both the design equations and the FS values have been changed in API 2A-WSD between 1993 and 2014.

Similarly, two versions of LRFD are used: (1) a version using the design equations in [API 2A-LRFD](#) (1993) and (2) a contrived version that reflects the updated design equations for tubular joints in API 2A-WSD (2014). In both versions, the load factors and resistance factors in API 2A-LRFD (1993) are used.

4.2.4 Component Resistance Uncertainty—Resistance Bias Factors

The resistance bias factors, B_R in Equation 4.1, account for bias and uncertainty associated with the nominal strengths used in design. Following Moses (1986), these factors are assumed to be lognormally distributed with a mean value $\bar{B}_R$ and a c.o.v. value V_R ; the mean and c.o.v. values depend on the component. Annex A summarizes the approach used by Moses (1986) to establish values of $\bar{B}_R$ and V_R for different component checks. In addition, updates to his approach based on new information are described in Annex A.

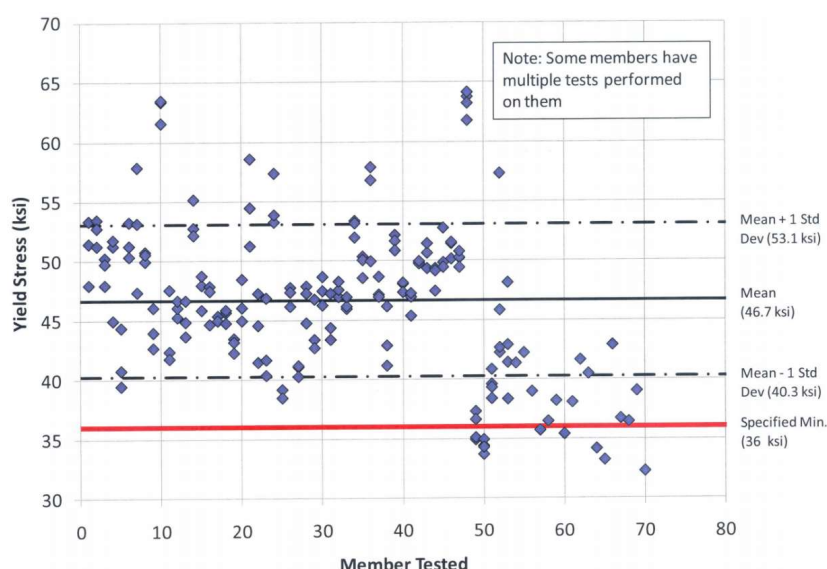
One significant difference between the resistance bias factors used in the updated analysis and those used previously by Moses (1986) is the bias and uncertainty between the actual yield strength and the nominal yield strength. Moses (1986) defined a bias factor, $B_{F_y} = \frac{\text{actual } F_y}{\text{nominal } F_y}$ with a mean value of $\bar{B}_{F_y} = 1.1$ and a c.o.v. of

$V_{F_y} = 0.08$ ². This same set of $\bar{B}_{F_y} = 1.1$ and $V_{F_y} = 0.08$ was also used in calibrating the load and resistance factors for ISO standards ([Moses and Stahl 2000](#)). A review of this bias factor was recently conducted ([Energo 2010](#)). The review covers the mean yield strength of steel for both onshore and offshore structures. Table 4.3 summarizes the mean yield strength and the c.o.v. in the yield strength for A36 and A572 Grade 50 for onshore and offshore structures, and Figure 4.1 shows the data for offshore structures. Based on this review, steel yield strength bias factors are modeled in the updated reliability analyses with $\bar{B}_{F_y} = 1.3$ and $V_{F_y} = 0.14$ for A36 steel and $\bar{B}_{F_y} = 1.15$ and $V_{F_y} = 0.09$ for A572 Grade 50 steel (Table 4.4).

² Different mean and c.o.v. values were used for the steel yield strength bias factor in earlier phases of the calibration work. For a structural component subjected to yielding under axial tensile loading, a mean of 1.1 and a c.o.v. of 0.13 were used by Moses (1981) and Moses and Larrabee (1988). For a structural component subjected to compressive and bending loads and tubular joints, a mean value 1.05 and a c.o.v. of 0.05 were used by Moses (1981).

Table 4.3—Mean and c.o.v. for Steel Yield Strength (Energo, 2010)

Source	Onshore		Offshore	
Material	A36	A572 Grade 50	A36	A572 Grade 50
Nominal yield strength (ksi)	36	50	36	50
Mean yield strength (ksi)	49.2	57.6	46.7	—
c.o.v. in yield strength	0.1	0.09	0.14	—
Number of samples	36570	13536	183	—

**Figure 4.1—Test Database for Yield Strength from Offshore Structures in the Gulf of Mexico (Energo, 2010)****Table 4.4—Mean and c.o.v for Bias in Nominal Yield Strength for Steel**

Material	Updated Analysis		Moses (1986)	
	A36	A572 Grade 50	A36	A572 Grade 50
Nominal yield strength (ksi)	36	50	36	50
Mean strength bias, $\bar{B}_{F_y}$	1.3	1.15	1.1	1.1
c.o.v. in strength bias, V_{F_y}	0.14	0.09	0.08	0.08

A second difference between the resistance bias factors used in the updated analysis and those used previously by Moses (1986) are the biases in joint strengths. These bias factors have been updated to reflect the new design equations for tubular joints in API 2A-WSD (2014).

A final difference between the resistance bias factors used in the updated analysis and those used previously by Moses (1986) is the bias in axial pile capacity. These bias factors have been separated into a factor for piles in predominantly clay soils and one for piles in predominantly sand soils. In addition, the bias factors have been updated to reflect observed versus predicted performance of pile foundations from Hurricanes Andrew (1992) through Ike (2008).

The values of $\bar{B}_R$ and V_R use in the updated reliability analysis are presented in Table 4.5 (yielding of cylindrical members in tension), Table 4.6 (local buckling for cylindrical members), Table 4.7 (pure bending for cylindrical members), Table 4.8 (compression for cylindrical columns), Table 4.9 (hydrostatic pressure for cylindrical members), Table 4.10 (axial capacity of steel pipe pile foundations) and Table 4.11 (strength of tubular joints).

Table 4.5—Mean and c.o.v. for Tension Resistance Bias

	A36	A572 Grade 50	Moses (1986)
mean, $\bar{B}_R$	1.3	1.15	1.1
c.o.v., V_R	0.14	0.09	0.08

Table 4.6—Mean and c.o.v. for Local Buckling Resistance Bias

	A36	A572 Grade 50	Moses (1986)
mean, $\bar{B}_R$	1.35	1.20	1.14
c.o.v., V_R	0.16	0.12	0.12

Table 4.7—Mean and c.o.v. for Pure Bending Resistance Bias

	A36	A572 Grade 50	Moses (1986)
mean, $\bar{B}_R$	1.37	1.21	1.25
c.o.v., V_R	0.16	0.12	0.11

Table 4.8—Mean and c.o.v. for Column Compression Strength Bias

Variable		Nominal $\lambda = \frac{kL}{\pi r} \sqrt{\frac{F_y}{E}}$									
		0.2	0.4	0.6	0.8	1	1.2	1.4	1.6	1.8	2
A36	mean, $\bar{B}_R$	1.29	1.28	1.27	1.26	1.17	1.10	1.00	1.00	1.00	1.00
	c.o.v., V_R	0.151	0.147	0.147	0.149	0.153	0.149	0.194	0.188	0.188	0.185
A572 Grade 50	mean, $\bar{B}_R$	1.15	1.14	1.14	1.13	1.10	1.07	1.03	1.00	1.00	1.00
	c.o.v., V_R	0.109	0.108	0.112	0.119	0.135	0.142	0.192	0.188	0.188	0.185
Moses (1986)	mean, $\bar{B}_R$	1.10	1.10	1.10	1.10	1.10	1.00	1.00	1.00	1.00	1.00
	c.o.v., V_R	0.103	0.104	0.110	0.119	0.147	0.149	0.195	0.188	0.188	0.185
NOTE L = Unbraced length, k = Effective length factor, E = Young's modulus of elasticity, and r = Radius of gyration.											

Table 4.9—Mean and c.o.v. for Hydrostatic Strength Bias

	Mean, $\bar{B}_R$						c.o.v., V_R		
	WSD (2014)			LRFD (1993)			WSD (2014) and LRFD (1993)		
F_{he}/F_y	A36	A572 Grade 50	Moses (1986)	A36	A572 Grade 50	Moses (1986)	A36	A572 Grade 50	Moses (1986)
0.5	1.04	1.04	1.04	1.04	1.04	1.04	0.11	0.11	0.11
1	1.34	1.18	1.13	1.25	1.11	1.06	0.20	0.15	0.14
1.5	1.47	1.30	1.25	1.31	1.16	1.11	0.20	0.15	0.14
2	1.51	1.33	1.28	1.32	1.17	1.12	0.20	0.16	0.15
2.5	1.55	1.37	1.31	1.34	1.19	1.14	0.19	0.15	0.15
3	1.56	1.38	1.32	1.33	1.17	1.12	0.18	0.15	0.14
3.5	1.57	1.39	1.33	1.30	1.15	1.10	0.19	0.15	0.14
4	1.58	1.40	1.34	1.29	1.14	1.09	0.19	0.15	0.14
NOTE F_{he} = Elastic hoop buckling stress.									

Table 4.10—Mean and c.o.v. for Pile Axial Capacity Bias

	Clay	Sand	Moses (1986)
mean, $\bar{B}_R$	1.05	1.46	1.0
c.o.v., V_R	0.21	0.42	0.2

Table 4.11—Mean and c.o.v. for Joint Strength Biases

Joint Type	Brace Loading	Bias	A36		A572 Grade 50		Moses (1986)
			WSD (2014)	WSD (2000)	WSD (2014)	WSD (2000)	
K-joints	Balanced axial	mean, $\bar{B}_R$	1.74	1.79	1.54	1.59	1.28
		c.o.v., V_R	0.22	0.23	0.19	0.20	0.19
	In-plane bending	mean, $\bar{B}_R$	1.91	1.68	1.69	1.48	1.35
		c.o.v., V_R	0.21	0.17	0.17	0.13	0.16
	Out-of-plane bending	mean, $\bar{B}_R$	2.00	1.50	1.77	1.32	1.29
		c.o.v., V_R	0.24	0.20	0.21	0.17	0.17
Y-joints	Axial compression	mean, $\bar{B}_R$	1.57	1.89	0.39	1.67	1.17
		c.o.v., V_R	0.18	0.24	0.14	0.22	0.11
	Axial tension	mean, $\bar{B}_R$	3.33	4.49	2.94	3.97	1.55
		c.o.v., V_R	0.32	0.32	0.30	0.30	0.43
	In-plane bending	mean, $\bar{B}_R$	1.83	1.30	1.62	1.15	1.35
		c.o.v., V_R	0.21	0.35	0.18	0.33	0.16
	Out-of-plane bending	mean, $\bar{B}_R$	1.89	1.39	1.67	1.23	1.29
		c.o.v., V_R	0.30	0.30	0.28	0.28	0.17
X-joints	Axial compression	mean, $\bar{B}_R$	1.52	1.51	1.35	1.33	1.17
		c.o.v., V_R	0.17	0.18	0.13	0.14	0.11
	Axial tension	mean, $\bar{B}_R$	3.12	3.45	2.76	3.05	1.55
		c.o.v., V_R	0.31	0.56	0.29	0.55	0.43
	In-plane bending	mean, $\bar{B}_R$	2.02	1.65	1.78	1.46	1.35
		c.o.v., V_R	0.24	0.25	0.21	0.23	0.16
	Out-of-plane bending	mean, $\bar{B}_R$	1.81	1.47	1.60	1.30	1.29
		c.o.v., V_R	0.15	0.17	0.11	0.13	0.17

4.3 Component Reliability

The reliability for different components is calculated from the performance function in Equation 4.1 using the First-Order Reliability Method or FORM (e.g. Ang and Tang 1984). Two different reliability analyses are conducted for the WSD- and LRFD-formats. First, the joint strength equations in the API 2A-WSD 21st Edition are used both for the WSD- and LRFD-formats with the metocean loading characterization used by Moses (1986) (Table 4.1) and described in API 2A-LRFD (1993) and API 2A-WSD (2000). These results are denoted “Previous Metocean – WSD” and “Previous Metocean – LRFD”. Second, the new joint strength equations in the API 2A 22nd Edition are used both for the WSD and LRFD formats with the updated metocean loading characterization described in API 2MET (2014) and Table 4.2. These results will be denoted “New Metocean – WSD” and “New Metocean – LRFD”.

Results for each component are presented in terms of the average 20-year reliability index $\beta_{20,avg}$ and associated failure probability $PF_{20,avg}$ (see Figure 2.1). The average reliability indices, $\beta_{20,avg}$, are average values the reliability index for a 20-year lifetime, β_{20} over a range of “typical” design conditions similar to those used Moses (1986). These typical design conditions constitute: a range of environmental to gravity load ratios of 2 to 40 for steel members, 0.5 to 3.0 for piles in compression and 1.5 to 5.0 for piles in tension; diameter-to-wall-thickness

ratios for steel tubular members such that the nominal bending capacity is equal to the full plastic bending moment (i.e. $D/t = 40$); non-dimensional column slenderness parameter values for buckling, λ , equal to 0.8 and 1.5; and ratios of the elastic hoop buckling stress to the yield strength of 0.5 to 4.0 for hydrostatic loading of steel tubular members.

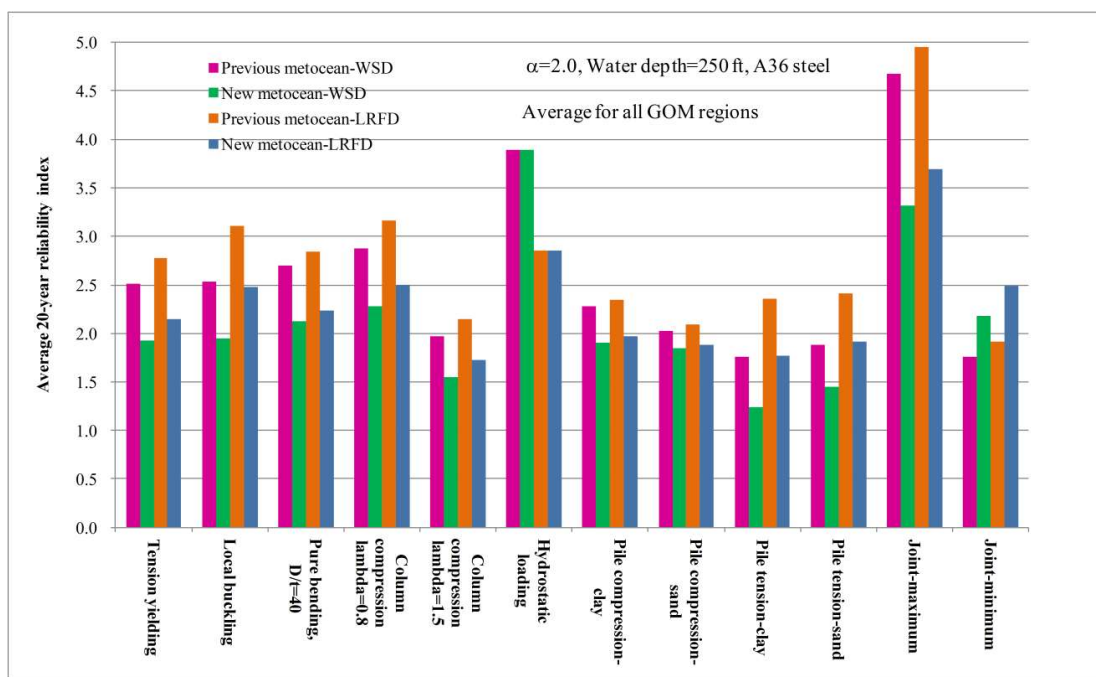
4.3.1 WSD versus LRFD

As concluded by Moses (1986), the LRFD-format produces reliability levels that are similar to or slightly greater than the WSD-format for both the 1993 and 2014 metocean characteristics (Figure 4.2). For the 2014 metocean characteristics (and new joint checking equations), the average value of $\beta_{20,avg}$ is 2.31 for the LRFD-format and 2.14 for the WSD-format. For the 1993 metocean characteristics (and older joint checking equations), $\beta_{20,avg}$ values with the LRFD-format are generally between 2.5 and 3.0 for components (Figure 4.2) and similar to the values obtained by Moses (1986) for similar assumptions. This result indicates that the updated representation of the nominal bias in the steel yield strength (Table 4.4) produces similar reliability because the updated mean bias is higher (increasing reliability) while the updated c.o.v. value is larger (decreasing reliability).

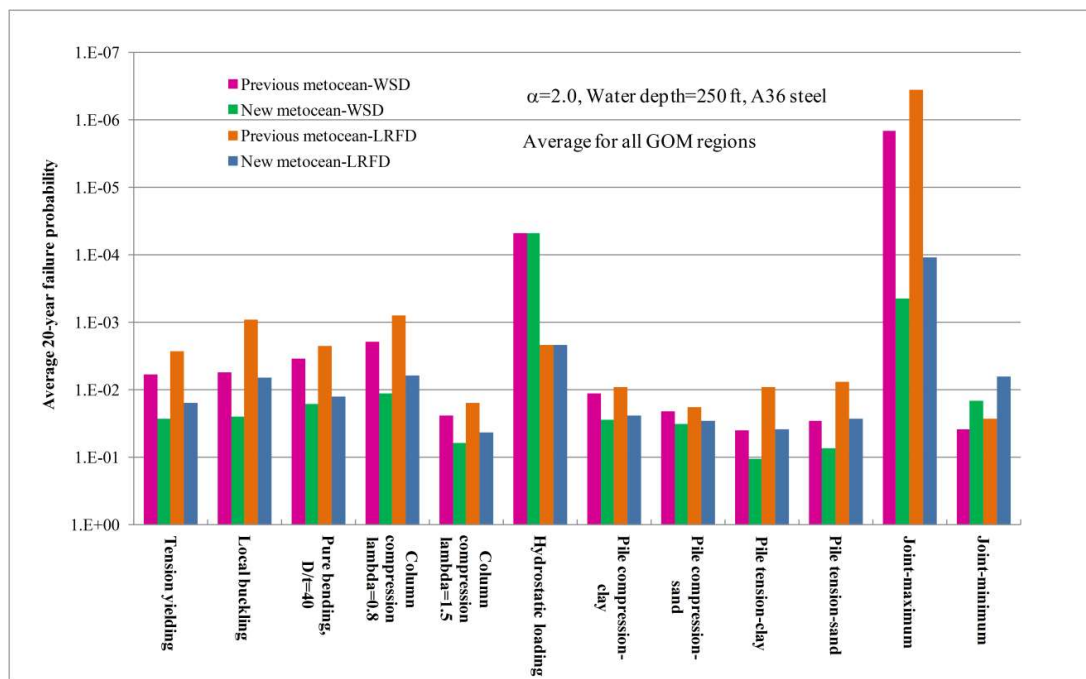
Also as concluded by Moses (1986), the LRFD-format produces reliability levels that are more uniform than the WSD-format among all components over a range of typical design conditions (Figure 4.2). An example of the variation of the reliability level over the environmental to gravity load ratio W_n/G_n is shown in Figure 4.3 for a column compression with $\lambda = 1.5$; the LRFD-format produces a more uniform β_{20} over W_n/G_n from 2 to 40 for both the 1993 and the 2014 metocean characteristics (Figure 4.3). The difference of the maximum and minimum β_{20} and the ratio of the maximum to minimum 20-year failure probability PF_{20} for all components over a range of typical design conditions are shown in Figure 4.4.

4.3.2 2014 Versus 1993 Metocean Characteristics

The average reliability index, $\beta_{20,avg}$, is generally smaller for the 2014 metocean characteristics than the 1993 metocean characteristics (Figure 4.2). This result reflects the additional variability in the maximum wave heights with the 2014 metocean characteristics; while the added variability increases the 100-year nominal load, it also increases the uncertainty in the possible maximum loads over a 20-year lifetime. One exception to this conclusion is for several joint types under particular loading conditions: Y-joints with in-plane and out-of-plane loading and X-joints in tension. While there is greater uncertainty in the maximum load with the 2014 metocean characteristics for these cases, the new joint strength equations more than compensate for this greater uncertainty. The reliability level for hydrostatic loading is independent of the metocean criteria since the wave load is negligible.



(a)

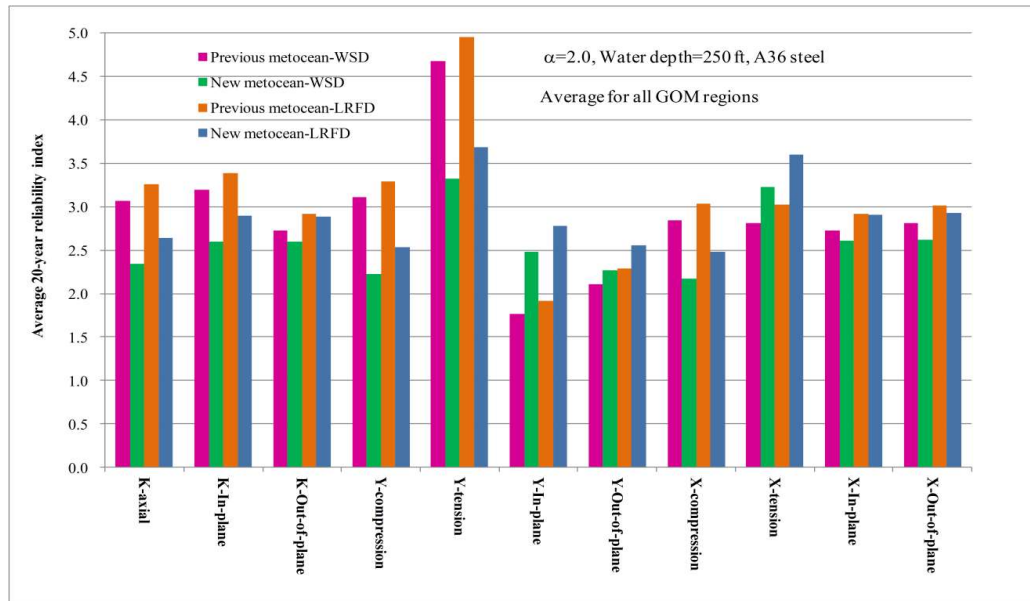


(b)

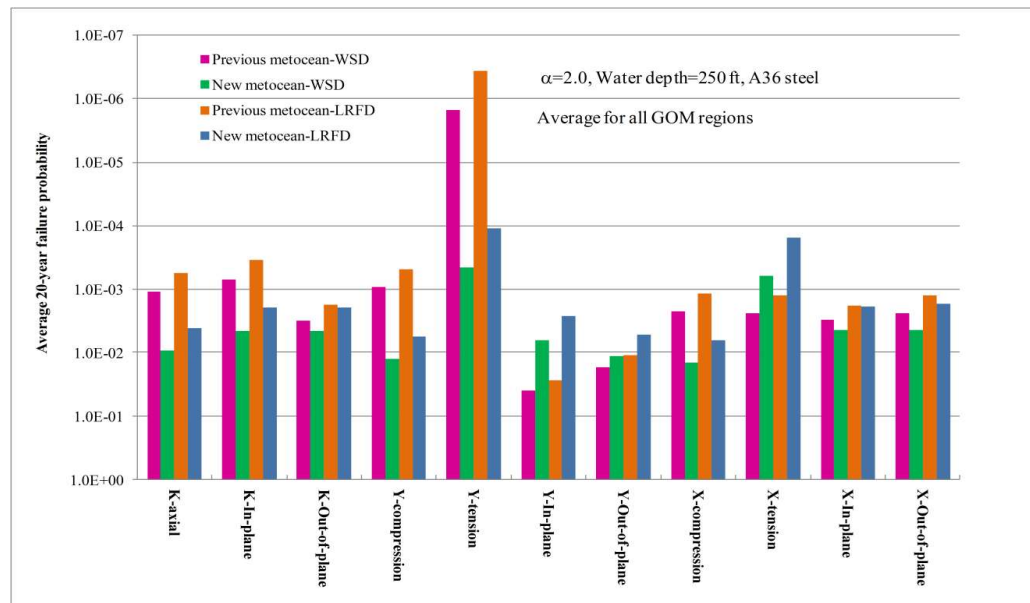
Figure 4.2—Reliability for Different Components Averaged over Range of Typical Design Conditions:

(a) All Components- $\beta_{20,avg}$; (b) All Components- $PF_{20,avg}$; (c) Tubular Joints- $\beta_{20,avg}$;

(d) Tubular Joints- $PF_{20,avg}$

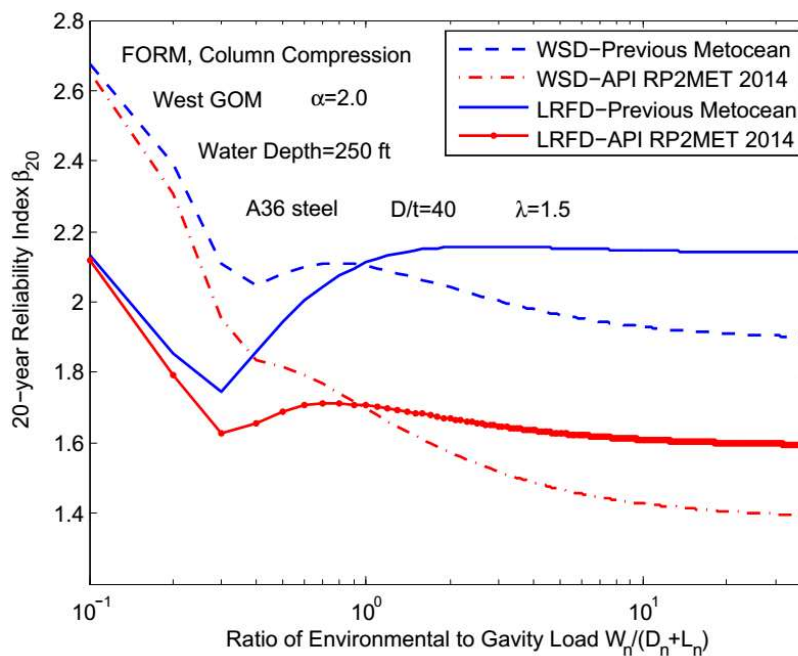


(c)

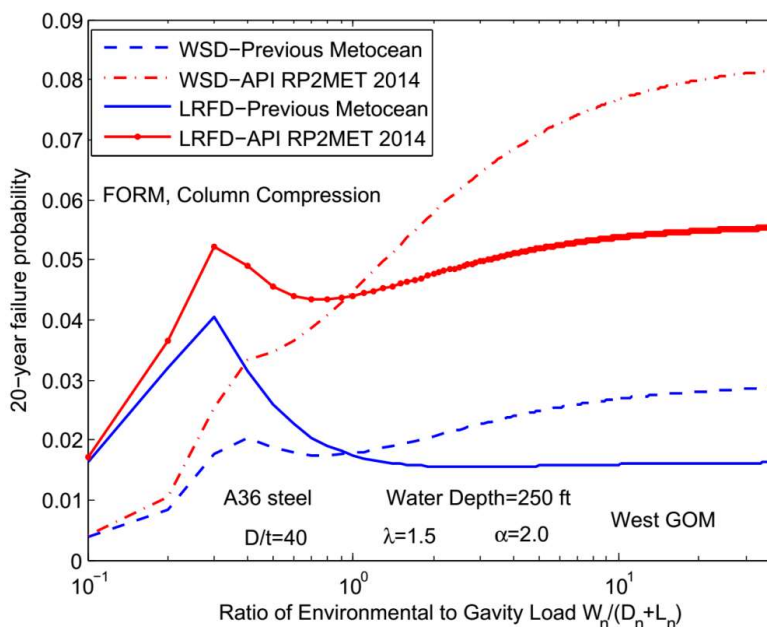


(d)

Figure 4.2 (continued)—Reliability for Different Components Averaged over Range of Typical Design Conditions: (a) All Components- $\beta_{20,avg}$; (b) All Components- $PF_{20,avg}$; (c) Tubular Joints- $\beta_{20,avg}$; (d) Tubular Joints- $PF_{20,avg}$

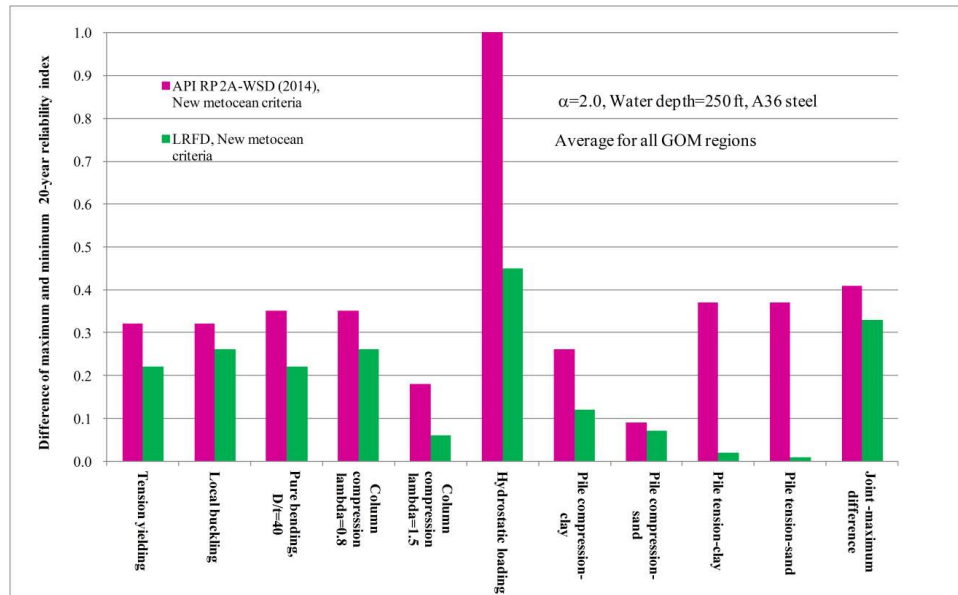


(a)

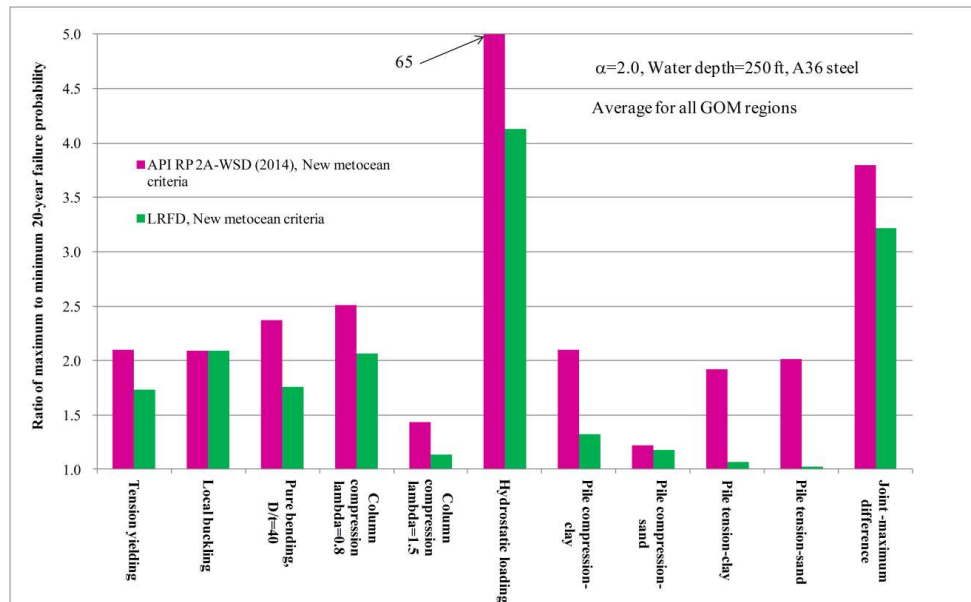


(b)

Figure 4.3—Variation of Reliability Level for Column Compression: (a) Reliability Index; (b) Failure Probability



(a)



(b)

Figure 4.4—Variation of Reliability Level for All Components over Range of Typical Design Conditions:
(a) Difference of Maximum and Minimum Reliability Indices and (b) Ratio of Maximum and Minimum Failure Probability

4.3.3 Sensitivity of Reliability to LRFD Environmental Load Factor

The reliability increases with the environmental load factor for the extreme environmental condition check in the LRFD-format, γ_w . To illustrate, the average reliability index $\beta_{20,avg}$ is shown versus γ_w in Figure 4.5 for a tubular member under pure bending. An environmental load factor of $\gamma_w = 1.28$ provides the same reliability for the LRFD-format as for the WSD-format for this component check with the 2014 metocean characteristics (Figure 4.5).

Figure 4.6 and Tables 4.12 and 4.13 summarize the effect of the environmental load factor on the reliability for all component checks with the 2014 metocean characteristics (and new joint checking equations). The average 20-year reliability index increases about 0.07 for $\alpha = 2.0$ and 0.1 for $\alpha = 1.0$ with an increase of γ_w by 0.05. Correspondingly, the average 20-year failure probability decreases about 20% for $\alpha = 2.0$ and 25 % for $\alpha = 1.0$ with an increase of γ_w by 0.05. With the value of $\gamma_w = 1.35$ (extreme case) in the API 2A-LRFD 1st Edition, $\beta_{20,avg}$ for the LRFD-format is on average larger than that for the WSD-format by about 0.3 and 0.4 for $\alpha = 2.0$ and $\alpha = 1.0$, respectively (Figure 4.6 and Table 4.3). Also, a value of $\gamma_w = 1.3$ (extreme case) for the LRFD-format gives higher reliability levels than the WSD-format for all components.

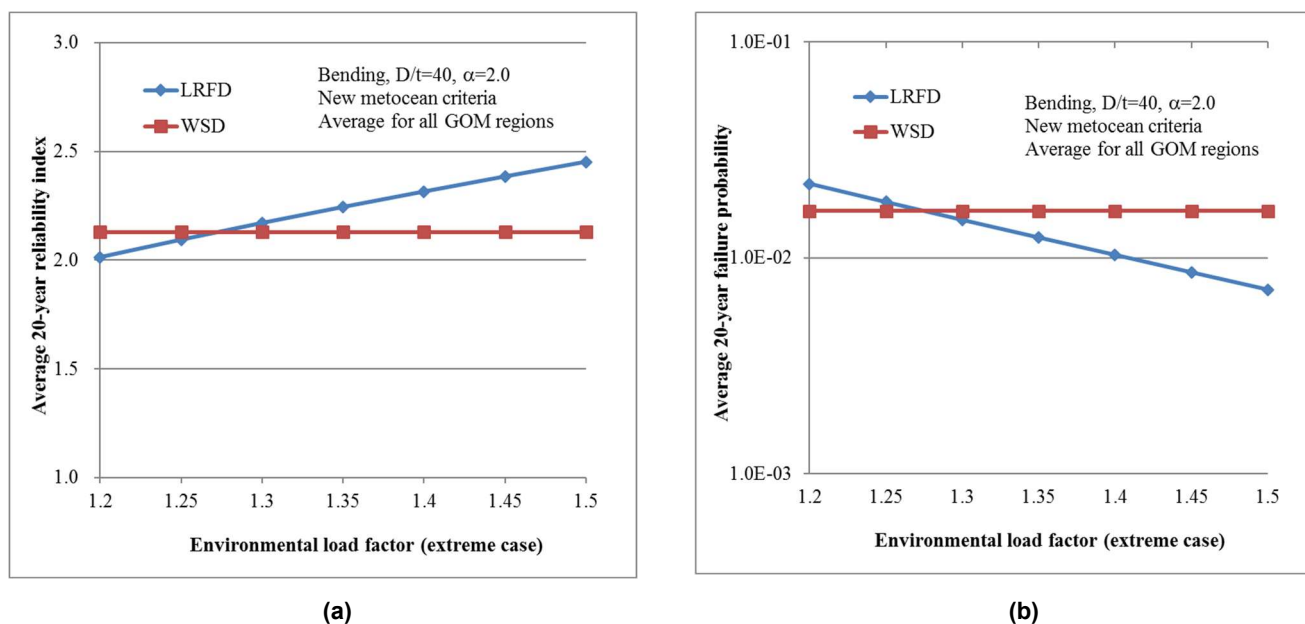
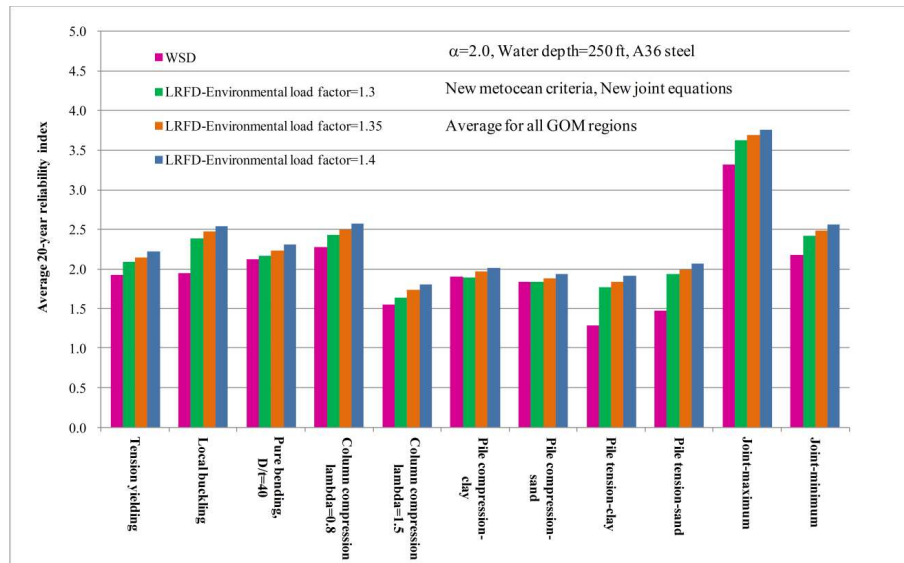
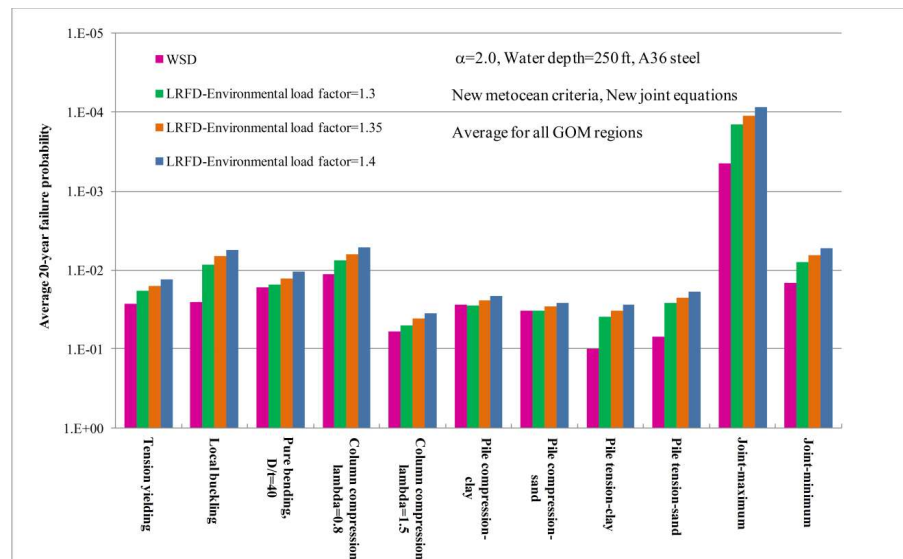


Figure 4.5—Effect of Environmental Load Factor on Reliability for Bending of Tubular Members:
(a) Reliability Index and (b) Failure Probability



(a)



(b)

Figure 4.6—Effect of LRFD Environmental Load Factor on Average Reliability over Range of Typical Design Conditions: (a) Reliability Index and (a) Failure Probability