Wind Load Analysis Uncertainty for Petrochemical Structures

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ABSTRACT

Petrochemical structures are often characterized by open framing, exposed equipment, and the presence of piping, cable trays, ladders, and open stairwells. These features (in combination) are not amenable to familiar wind load estimation techniques, and intuitively one would expect that their estimation would carry relatively greater uncertainty. This paper describes work toward quantifying this uncertainty. Wind load estimates are compared to wind tunnel test results for a variety of structures. From these comparisons, statistics describing the bias and variance of the estimates are developed, allowing computations of the reliability of a generic structure to be performed. Somewhat surprisingly, the resulting reliabilities for petrochemical structures appear to be only slightly lower than those of regularly-shaped, enclosed buildings.

INTRODUCTION

A research program was undertaken at Louisiana State University to investigate wind loads for structures common to the petrochemical industry. Although there are tools available for estimating the wind loads for these structures [1], their complex geometries often defy conventional wind load analysis techniques. Since the history of wind engineering research for these structures is not as long as that of regularly-shaped enclosed buildings, it was expected that the analysis techniques for industrial process structures would reflect greater uncertainty. In this scenario, the increased uncertainty in wind load analysis would be associated with lower structural reliabilities for process structures compared to buildings with more common geometries. This paper presents the results of work that (1) compared wind load estimates to wind tunnel measurements for a variety of model petrochemical structures, (2) used the resulting data to estimate statistical descriptions of the uncertainty in force coefficient estimation for petrochemical structures, and (3) used these uncertainty statistics to compare the structural reliabilities of petrochemical structures under the action of wind to those of regularly-shaped enclosed buildings.

SELECTED DATA SET

A reliability analysis requires that the variables involved in the process under investigation be defined either as deterministic or random variables. The deterministic variables will have a single value, but the random variables will take on values that vary according to probability distributions defined by distribution type, mean value, and standard deviation. Other parameters may be required depending on the distribution type. The primary variable of interest in the reliability analysis of this paper is the force coefficient, C_f. This variable has been measured in the wind tunnel for a variety models representing petrochemical structures for this research program and also by a previous researcher [2]. The first task in this reliability analysis is to establish the statistical description of this random variable. The models selected for this analysis include an open frame tower in 4 configurations for each of two wind directions totaling eight configurations, a vertical vessels in 14 configurations [3], and a set of the open frame structures modeled in the wind tunnel by Qiang [2]. In order to keep the data set to a manageable size (hand calculations must be conducted for each model included), a randomly generated selection of 26 cases from Qiang's study were included in the data set. These cases include a variety of plan and elevation aspect ratios, framing conditions, and equipment types and configurations. A primary assumption of the analysis in this paper is that the wind tunnel testing results are representative of the prototype conditions.

In order to form a single data set from the variety of structures required for this analysis, the measured force coefficients were normalized by the calculated force coefficients using different analysis techniques. Values greater than unity indicate that the analysis technique underestimated the measured force coefficient, and vice versa. The statistical properties of the normalized data set were used for the reliability analysis. The mean value of all of the normalized data represents the bias factor. The bias factor, which is often referred to by the variable λ, is defined as the mean value divided by the nominal value. For the normalized data in this paper, the nominal value is 1.0 by definition. Bias factors less than unity indicate overestimation of the design parameter (and vice versa). Bias factors less than unity are conservative for variables that contribute to structural resistance, whereas bias factors less than unity are unconservative for variables that contribute to the load, or demand, on the structure.

Two different data sets were used in the reliability analysis. The first was generated by calculating the force coefficients on the selected models using current analysis methods recommended by the ASCE [1]. The second data set was generated by calculating the force coefficients for the models by using analysis techniques which are similar to the ASCE methods, but modified in the following ways [3]:

- Accounting for diagonal bracing located in frame planes oriented nominally along the wind direction for open frame structures;
- Accounting for the presence of solid floors;
- Using an alternative method of accounting for equipment shielding for open frame structures; and
- Increasing the force coefficients for vertical vessels to account for aerodynamic interference due to cross-wind proximity when neighboring vessels are within three diameters of one another.

CHARACTERISTICS OF THE DATA SET

The reliability analysis requires definitions of the means and standard deviations for the data sets as well as the form of the probability distribution function describing the variations in the data sets. The resulting data were ordered and plotted on a normal probability plot to facilitate this process. The normal probability plot has the value of the variable on the horizontal axis and the standard normal variable on the vertical axis. The standard normal variable corresponding to each variable value was calculated using the inverse of the standard normal probability distribution with the following input probability for each data item:

$$P_i = \frac{i}{N+1} \tag{1}$$

where *i* is the position of each data item in an ascending ordered list of the data, and *N* is the total number of data items (48 in this case). The resulting probability plot is shown in Figure 1. A line of best fit has been generated for each of the two data sets. The inverse of the slope and the *x*-intercept of these lines provide estimates of the standard deviation and mean values of each of the data sets (provided that a normal distribution describes the variation in the data). Using this technique, the bias factors for the data generated from the original analysis techniques and the modified techniques have been calculated to be 0.94 and 0.95, respectively. This shows that both methods systematically overestimate force coefficients, and the original analysis methods do this only very slightly more than the modified methods. The respective standard deviations are 0.189 and 0.129.

The data sets contained force coefficient estimates for both open frame and vertical vessel structures. The vertical vessel structures contributed approximately one-quarter of the force coefficient estimates in the data sets. Without surveying actual petrochemical facilities in the field to determine the relative distribution of open frame and vertical vessel structures, the distribution implied by the composition of these data sets is a potential source of error in the analysis of wind load analysis uncertainty and the resulting estimates of structural reliability. Therefore, the biases and standard deviations have been calculated for the original and modified analysis methods excluding the vertical vessel data (i.e. using only the open frame structure data). The bias factors for data generated using the original and modified analysis methods were 0.92 and 0.95, respectively. The respective standard deviations were 0.197 and 0.116. The changes in the statistical properties of the two data sets due to the exclusion of the vertical vessel data appear to be minor. The actual effects of these differences on estimates of structural reliability will be addressed in a later section.

The steepening of the slopes of the linear regression lines in Figure 1 between the original and modified analysis methods provides a visual or qualitative representation of the fact that the range of likely values for the normalized force coefficient has been reduced. This means that uncertainty in a designer’s ability to predict the force coefficient has decreased as a result of the proposed modifications to analysis practice. Furthermore, the degree to which a linear model represents the variation of data on a plot such as Figure 1 provides some insight into the suitability of assuming a normal probability distribution. There appears to be a good correlation between the data and the linear regressions, indicating that a normal distribution is indeed appropriate for the force coefficient data. The scatter of the data about the respective regression line is decreased for the modified analysis methods. This can be detected visually in Figure 1, but this feature is also captured by the correlation coefficient for the linear regression, which is slightly higher for this case. This indicates that the variation in the force coefficient for the data generated using modified analysis methods is more closely described by a normal distribution than the data generated using the original analysis methods.

The agreement of the data with a normal distribution can also be investigated quantitatively using a “goodness-of-fit” analysis [4]. A chi-squared test statistic is calculated for the data using the following formula:

$$\chi^2 = \sum_{i=1}^{n} \frac{(f_i - E(f_i))^2}{E(f_i)} \tag{2}$$

where

i = class interval number;

n = number of class intervals;
f_i = frequency within class interval, I; and
$E(f_i)$ = expected frequency within class interval, i for a normal distribution with a proposed mean value and standard deviation.

The test statistic is compared to the chi-squared distribution with the degrees of freedom equal to $n - 1$ for the desired level of confidence. If the chi-squared test statistic exceeds the value of the chi-squared distribution for a given level of confidence, then the null hypothesis that the proposed normal distribution describes the variation in the data is rejected.

This analysis was carried out for both data sets with mean values and standard deviations derived from the probability plots. The value of the chi-squared test statistic for the data derived from original analysis methods was 15.5. For 16 degrees of freedom, the value of the chi-squared distribution for the 90% confidence level is 23.5. It is therefore concluded that there is not sufficient reason to reject the hypothesis that the normal distribution is suitable at this confidence level. In fact, the confidence level would have to be reduced to 51% before the null hypothesis is rejected (i.e. $p = 0.49$). The value of the chi-squared test statistic for the data generated using modified analysis techniques was 8.2. This leads to a p-value of 0.94 for 16 degrees of freedom and a rejection of the null hypothesis at only the 6% confidence level. Clearly there is not sufficient justification for rejecting the hypothesis that these data are normally distributed in either case.

Just because the normal distribution appears to fit the data reasonably well does not eliminate the possibility that another distribution would describe the variation in the data more effectively. The shapes of the probability distribution functions can sometimes provide clues about possible distribution functions. Histograms of the data did not display any appreciable skew, but the bias factor has a reasonable lower bound value of zero. As such, a distribution function that is bounded from below may also fit the data well. The lognormal distribution is an example of such a function. A lognormal distribution is one in which the natural logarithms of the variable values are normally distributed. The two data sets were examined using the chi-squared "goodness-of-fit" test to determine if a lognormal distribution provided a superior description of the variation in the data. The resulting p-values from this test for the data from the original analysis methods and the modified methods were 0.79 and 0.85, respectively. The lognormal distribution performed slightly better for the original analysis methods but not as well for the modified methods. The relatively small sizes of the data sets restrict the resolution of the data on the low ends of the probability distributions. This may explain the equivocal results for the "goodness-of-fit" tests for the lognormal distribution. Since the lognormal distribution was not clearly superior to the normal distribution, the subsequent reliability analysis assumes that the variations in the force coefficient data are normally distributed.

RELIABILITY ANALYSIS

A structural reliability analysis was conducted for a generic structure. A structural frame was devised such that the reliability of an individual member could be checked for wind load in the absence of other loads. The bracing member in a braced frame was designed for tension loads that develop only under the action of wind for the assumed model. The structural model is shown in Figure 2.

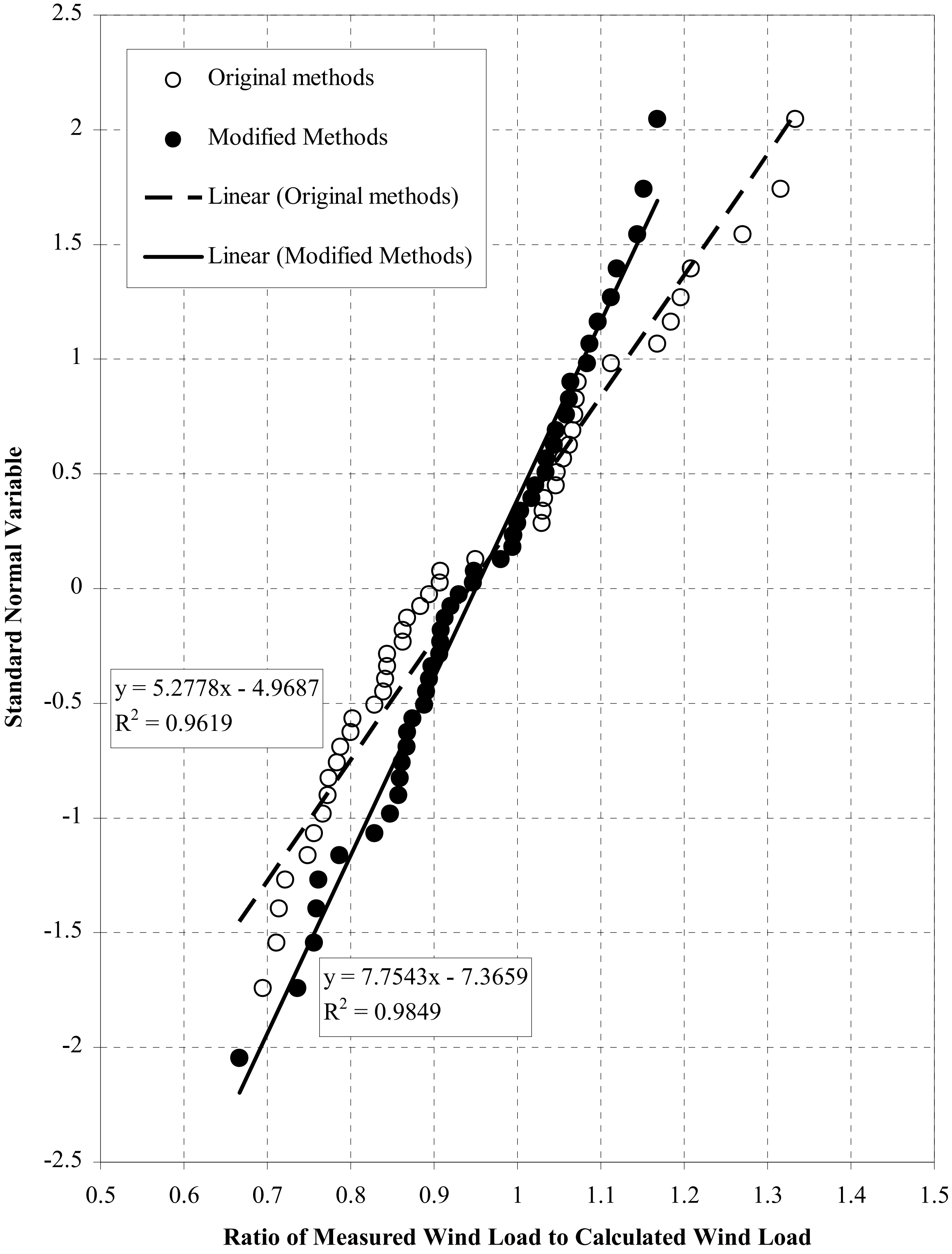

Figure 1 Normal probability plot of force coefficient data for the reliability analysis.

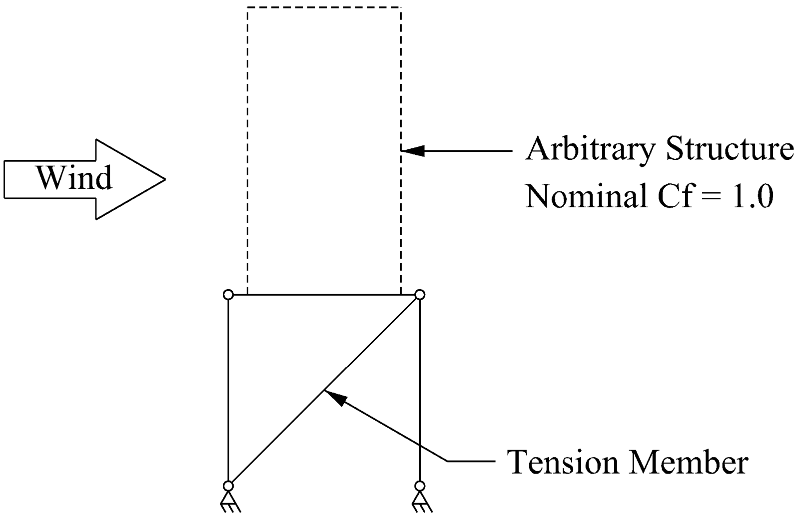

Figure 2 Generic structure used for the reliability analysis

In this configuration, the arbitrary structure is assumed to have a nominal force coefficient of 1.0. This allows for a reliability analysis encompassing the various structure types represented in the data set using normalized variables. The wind load generated on the arbitrary structure will have an associated base shear and overturning moment. However, in order to simplify matters, it has been assumed that only the base shear is effective in transmitting force to the lateral load resisting member (tension brace) in the frame. The nominal cross sectional area of the tension member was specified according to the following equations:

$$0.9 \cdot A_s \cdot F_y = 1.6 \cdot W_n \tag{3}$$

$$W_n = 1.414 \cdot (0.00256 \cdot K_z \cdot K_{zt} \cdot K_d \cdot V^2 \cdot I) \cdot C_f \cdot G \cdot A \tag{4}$$

where

$F_y = 36{,}000 psi$;
K_z = 1.26, 0.90, or 0.62 - for three different exposures;
$K_d = 0.85$;
V = 150, 115, 105, or 90 mph – for various locations in the United States;
I = 1.0;
$C_f = 1.0$;
G = 0.85 – assuming the structure is rigid; and
A = 200 ft^2.

The design space over which the reliability of this system was evaluated incorporated variation in exposure (through the variable K_z) and design wind speed, V. Having specified the structural system it is possible to estimate the reliability (or conversely the probability of failure) of this system for each location in the design space. The Rackwitz-Fiessler Modified Matrix Procedure was chosen for this analysis. This method is a first-order, second-moment reliability method. This particular method was selected for this analysis because it has the ability to accommodate non-normal random variable distribution types. The method is iterative and is performed as described in Nowak and Collins [5]. More detail has been included in Amoroso [3]. Tables 1 and 2 list the random variables that were considered in this analysis.

Table 1 Variables for the limit state function that define the design space for the reliability analysis

Variable	Description	Nominal Value	μ	λ	σ/μ	Dist. Type	Comments
K_z	Exposure Factor	1.26	1.210	0.96	0.116	Normal	Exposure C with z = 100 ft [6]
		0.9	0.84	0.93	0.143	Normal	Exposure C with z = 20 ft [6]
		0.62	0.63	1.016	0.190	Normal	Exposure B with z = 20 ft [6]
V	Design wind speed (mph)	90	60.0	0.667	0.170	E.V. Type I	St. Louis, MO [6, 7]
		115	56.7	0.493	0.332	E.V. Type I	Baytown, TX [8]
		105	52.8	0.503	0.297	E.V. Type I	Baton Rouge, LA [8]
		150	91.2	0.608	0.190	E.V. Type I	Pascagoula, MS [8]
		90	55.5	0.617	0.233	E.V. Type I	Philadelphia, PA [8]

Table 2 Variables for the limit state function that do not define the design space for the reliability analysis

Variable	Description	Nominal Value	μ	λ	σ/μ	Dist. Type	Comments
R	Tension member resistance = $A_s \cdot F_y$ (lb)	Varies	Varies	1.05	0.11	Normal	The uncertainty in A_s and F_y are handled together [5].
K_{zt}	Topographic Effect Factor	1	N/A	N/A	N/A	N/A	deterministic for nominally flat terrain
K_d	Directionality Factor	0.85	0.86	1.012	N/A	N/A	deterministic except for some bias [6]
I	Importance factor	1	N/A	N/A	N/A	N/A	Deterministic - used for adjusting MRI of design wind speed
G	Gust Effect Factor	0.85	0.82	0.965	0.098	Normal	[6]
C_f	Force Coefficient	1	0.94	0.94	0.201	Normal	Original analysis (present work)
		1	0.95	0.95	0.136	Normal	Modified analysis (present work)
A	Reference Area (ft^2)	200	N/A	N/A	N/A	N/A	uncertainty handled within estimate of C_f

RESULTS AND DISCUSSION

The above procedure was carried out for the structural configuration described in section 4 with input values for the means and standard deviations of the force coefficient which corresponded to the data sets shown in Figure 1. Since the wind speed probability distribution was based on the probability of experiencing an annual maximum wind speed, the resulting reliability indices and associated probabilities of failure also represent results on an annual basis. This is consistent with the methodology employed by Ellingwood and Tekie [6].

The resulting reliability indices for the structure designed according to the original methods are in Table 3. The reliability index, β, can be transformed into a corresponding probability of failure, P_f, by changing its sign and using it as a standard normal variable.

$$P_f = \Phi(-\beta) \tag{5}$$

where Φ is the standard normal cumulative probability distribution function. For example, the probability of failure associated with a reliability index equal to 3.10 is the probability of sampling a value of -3.10 from a normally distributed population with a mean value of zero and a standard deviation of one. This probability is equal to 9.68 x 10^{-4}.

Table 3 Reliability indices for tension member designed according to original analysis methods for various locations and exposures

	St. Louis	Baytown	Baton Rouge	Pascagoula	Philadelphia
$K_z = 1.26$	3.54	3.37	3.54	3.74	3.26
$K_z = 0.90$	3.61	3.41	3.68	3.76	3.33
$K_z = 0.62$	3.33	3.20	3.39	3.51	3.10

The results of the reliability analysis using the modified analysis methods are shown in Table 4. It is apparent that the reliability indices increased marginally compared to those calculated for a member designed using the original analysis methods. The minimum reliability index from the results for the original analysis methods was 3.10. The minimum reliability index from the results for the modified analysis methods was 3.17. The average reliability indices from the original and modified analysis methods were 3.45 and 3.52, respectively. These average reliability indices represent respective probabilities of failure of 2.85 x 10^{-4} and 2.15 x 10^{-4}. On the basis of the average reliability indices, applying the modifications to the analysis methods resulted in a 25% reduction in the annual probability of failure for the tension member. Ellingwood and Tekie [6] mention that target reliability indices for probability-based structural design methods such as Load and Resistance Factor Design (LRFD) are in the neighborhood of 3.2. Only the lowest of the reliability indices for either of the analysis methods are below this target level.

When describing the data sets in a previous section, it was noted that eliminating the vertical vessels from the data had a minor impact on the resulting bias factors and standard deviations for the estimated force coefficients. The effects of these small differences on the resulting structural reliabilities were also small. For example, the reliability index for the structure located in Philadelphia with nominal $K_z = 0.62$ was 3.13 when the vertical vessels were excluded and the original analysis methods were used. Likewise, the reliability index for the same location in the design space, but using the modified analysis methods, was 3.18.

Table 4 Reliability indices for tension member designed according to modified analysis methods for various locations and exposures

	St. Louis	Baytown	Baton Rouge	Pascagoula	Philadelphia
$K_z = 1.26$	3.65	3.42	3.60	3.85	3.33
$K_z = 0.90$	3.72	3.46	3.64	3.86	3.40
$K_z = 0.62$	3.43	3.25	3.45	3.60	3.17

The analysis was repeated in order to generate values that would be typical of an enclosed building. Enclosed buildings have received much more research attention from a wind loading perspective, and current structural design codes have been calibrated to achieve a target reliability using enclosed buildings as reference structures. A comparison of the reliability analysis results for the petrochemical structures to results for an enclosed building would identify relative shortcomings in analysis practice for petrochemical structures. The analysis was repeated using equivalent force coefficient random variable parameters which were synthesized from the nominal values, mean values, and standard deviations for windward and leeward wall pressure coefficients documented by Ellingwood and Tekie [6]. The input mean value of the equivalent force coefficient was 0.889 and the input standard deviation of the equivalent enclosed building force coefficient was 0.147. The resulting reliability indices for the enclosed building are in Table 5. These results show that the reliability of the tension wind brace for an equivalent enclosed structure is slightly higher than for the design resulting from either of the analysis methods for the petrochemical structures. The difference is mainly attributed to the lower bias factor for the force coefficient, since the standard deviation was between the standard deviations for force coefficients derived from the original and modified analysis methods for petrochemical structures.

Table 5 Reliability indices for tension member in an equivalent enclosed building for various locations and exposures

	St. Louis	Baytown	Baton Rouge	Pascagoula	Philadelphia
$K_z = 1.26$	3.74	3.51	3.68	3.94	3.42
$K_z = 0.90$	3.81	3.55	3.73	3.95	3.49
$K_z = 0.62$	3.52	3.33	3.53	3.69	3.25

Finally, the effects on the reliability index and the probability of failure with respect to the standard deviation of the force coefficient and the mean value of the force coefficient relative to its nominal value were investigated. This analysis was performed for a single location in the design space (Baton Rouge with $K_z = 0.9$). The results of this analysis are summarized in Figures 3 and 4. There is a nonlinear variation of the reliability index with the standard deviation of the force coefficient, and there is a linear relationship of the reliability index with the bias factor. These effects are not surprising given the formulation of the reliability index.

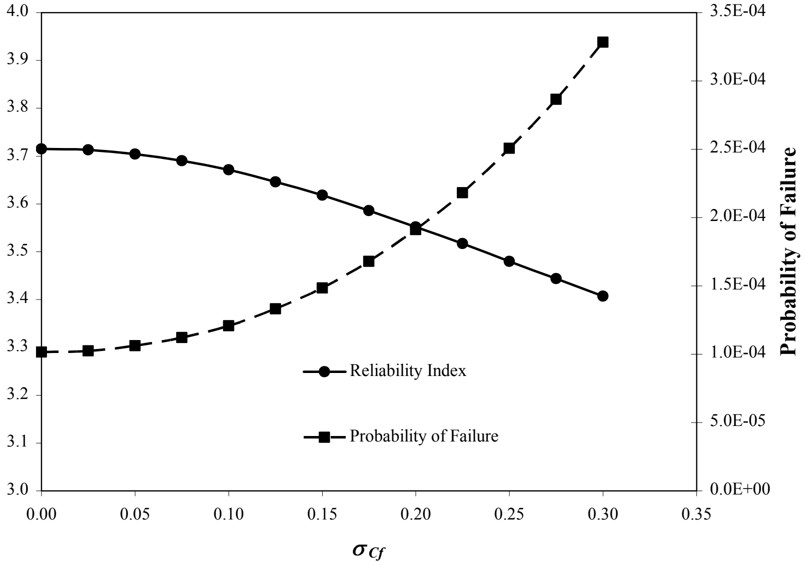

Figure 3 Variation of the reliability index and the probability of failure with the standard deviation of the force coefficient for $\lambda = 0.95$.

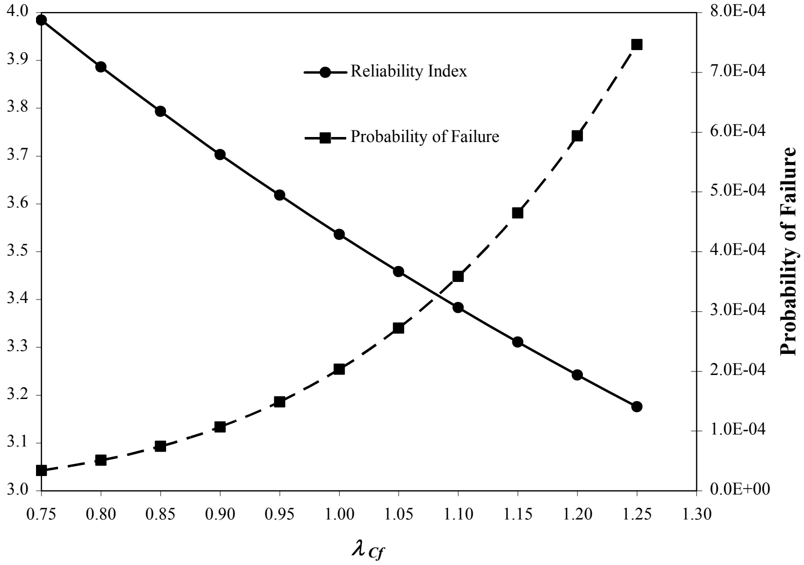

Figure 4 Variation of the reliability index and the probability of failure with the bias factor of the force coefficient for $\sigma = 0.15$.

CONCLUSIONS

Measured force coefficients for model petrochemical structures were collected and compared to analytical results to create a data set useful for conducting a reliability analysis for petrochemical structures under the action of wind. These comparisons were carried out using existing analysis techniques as well as with modified methods that are pending approval by an ASCE task committee. Both the original and modified methods overestimated the measured force coefficients slightly. The mean values of the normalized data sets (measured value divided by the nominal value) for the original and modified methods were 0.94 and 0.95, respectively. The standard deviations for these two data sets were 0.189 and 0.129 for the original and modified analysis methods, respectively. This difference in standard deviations indicates significantly less uncertainty in the force coefficient estimates when using the modified methods. Neither data set was found to deviate significantly from a normal distribution, but the data generated from the modified analysis techniques performed more favorably than the data for the original analysis methods when subjected to a chi-squared “goodness-of-fit” test.

The reliability of a tension member in the lateral load resisting system of a generic petrochemical structure was analyzed using force coefficient probability distributions derived from the experimental data. The reliability indices calculated for both data sets were near or above the target reliability of 3.2 that has been established in the literature, with few exceptions. The modified analysis methods performed marginally better. This comparison is for structures designed for wind loads as determined from the 1997 ASCE guide. Many existing structures were designed and constructed prior to the publication of the ASCE guide [1]. The guide surveyed existing analysis practice at the time and showed that there was a great degree of variation among practices. This would seem to lead to a much greater coefficient of variation (and probably lower reliability) for the force coefficient estimate for older structures.

The analysis was repeated for a force coefficient probability distribution representative of enclosed buildings. The reliability indices of the enclosed building were calculated to be slightly higher than that of a petrochemical structure analyzed using the modified methods. This occurred primarily as a result of the difference in the bias factor (0.889 for the enclosed building and 0.95 for the petrochemical structures). Force coefficients for enclosed buildings are systematically overestimated to a greater degree than for petrochemical structures, resulting in relatively lower probabilities of failure.

Based on the analysis in this paper it is expected that the standard deviation for the force coefficient estimate will generally be between 0.1 and 0.2. An analysis of the effect on the reliability index to changes in the standard deviation of the force coefficient suggests that the structural reliability will not deviate significantly from established target levels within this range of variability in the force coefficient estimate.

REFERENCES

[1] ASCE (1997) Wind Loads and Anchor Bolt Design for Petrochemical Facilities, Task Committee on Wind Induced Forces and Task Committee on Anchor Bolt Design, American Society of Civil Engineers, New York.

[2] Qiang, L. (1998) Wind Tunnel Tests for Wind Loads on Open Frame Petrochemical Structures, Masters Thesis, Louisiana State University, Baton Rouge, LA, 204p.

[3] Amoroso, S. (2007) Wind Loads for Petrochemical Structures, Ph.D. Dissertation, Louisiana State University, Baton Rouge, LA, 264 p.

[4] Freund, R. J. and Wilson, W. J. (2003) Statistical Methods, Second Edition, Academic Press, Elsevier Science.

[5] Nowak, A. S. and Collins, K. R. (2000) Reliability of Structures, McGraw Hill.

[6] Ellingwood, B. and Tekie, P. B. (1999). "Wind load statistics for probability-based structural design." J. Struct. Eng., ASCE, 125(4), 453-463.

[7] Peterka, J. and Shahid, S. (1998) "Design gust wind speeds in the U.S." J. Struct. Div., ASCE, 124(2), 207-214.

[8] Vickery, P. J., Skerlj, P. F., and Twisdale, L. A. (2000) Simulation of hurricane risk in the U.S. using empirical track model, J. of structural eng., v. 126, n. 10, pp. 1222 – 1237.