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Application of Reliability to Formulation of Fixed Offshore Design Codes

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ABSTRACT

Several benefits have been proposed for the implementation of structural reliability concepts in the design of fixed offshore structures. These include:

a) Greater uniformity in platform component reliability.

b) More effective utilization of material than occurs with existing deterministic safety factor design procedures.

c) Accounts directly for randomness and uncertainties in engineering parameters.

d) Capable of consistent modifications to account for different location, platform type and life.

e) Reliability gives a logical basis for incorporating new information

f) New procedures can interface with similar reliability developments in structural engineering such as AISC or ACI.

g) Help to focus research activities to emphasize areas of greatest uncertainty and have results impact reliability factors.

In 1979, the American Petroleum Institute (API) began a series of continuing studies under the direction of the author to implement reliability design procedures for fixed offshore platforms. The effort has reached the stage where a self-contained design alternative is now available for review and feedback from the various API technical committees. A complete design document should be available in the near future.

The reliability design procedure is known in the United States design practice as load and resistance factor design or LRFD. The code contains a variety of different resistance factors for component types (connections, piles, axial compression, hydrostatic strength, etc.) and different load factors for types of loading effects (gravity; wind, wave and current; seismic, etc.).

The research in producing this LRFD document for offshore platforms included the following steps.

a) Assembling statistical data on load effects and component strengths.

b) Review present performance criteria and experience.

c) Establish a target reliability level for each component type based on performance experience.

d) Calibrate the load and resistance factors for tabulation in the code.

In addition, several comparison studies were made by consulting firms and producers between existing working stress or safety factor design (RP 2A) with the new LRFD provision. Seven full scale platforms were compared and studied. In addition to wave loadings which primarily affect shallow water platforms, the project also developed LRFD provisions for frontier area fixed platforms including fatigue, deep-water dynamically sensitive structures and seismically loaded platforms.

The paper reviews the theoretical developments of the reliability based LRFD code in addition to the impact of results on the safety, economy and design practices of offshore platforms.

INTRODUCTION

Conventional structural design practice has evolved over the last one hundred years in an interactive manner between theoretical developments in understanding behavioral phenomena and empirical observations from performance histories. Thus, during this period the actual factors of safety in conventional steel buildings have usually been reduced by a combination of improved analysis methods, better quality control of materials and an acceptably low observed probability of failure. In fact, most reported failures in conventional structures are not the consequence of an inadequately specified factor of safety. Rather, most failures are a consequence of inadequate communication between the design engineer and the construction phase or the result of limited technological understanding of the failure mechanism at the time of design. The fact that few reported failures are due to inadequancy of specified factors of safety has led increasingly to proposals to reduce these factors in the overall interest of economy. In areas of new technolgy, such as marine and other offshore structures, the selection of safety factors often followed land-based building and bridge rules. This occurred despite the differences in fabrication and material quality control and especially the impact of overall system failure modes, reserve strength and differences in types of loading phenomena.

In recent years, changes in structural design codes and safety philosophy have increasingly been based on concepts of balanced risk. These ideas fostered over fifty years ago by Fruedenthal and others (1) suggest that optimal structure design required a balance between material cost and the risk cost (probability of failure times cost of failure). Application of the concepts to building, ship, bridge and offshore platform design codes have illustrated that the historical evolutionary approach to mode development has not always provided either uniform risk or optimal structures.

In many instances, the impetus for applying reliability methods has come from the evaluation of existing structures rather than new designs. The evaluation activity highlights the fact that when the cost of increasing safety margins is inordinately high, then a more economical alternative to achieve adequate safety may be to reduce uncertainties by better site investigations, more accurate analysis or by testing. As illustrated below, alternatives for meeting the same high reliability targets can be accomplished by either high safety factors or lower uncertainties.

This paper reviews the application of reliability methodology for the development of offshore platforms. In particular, these activities have been carried out for more than twelve years sponsored by industry, government and the American Petroleum Institute (API). Some eight years ago, it was concluded that the best avenue for implementing reliability concepts in offshore structures was through the introduction of a reliability-based design specification (2). This effort parallels similar recent proposals or adoption of reliability methods for codes dealing with steel building structures, concrete buildings, bridges, transmission towers, etc (3-5). The rationalization of a strategy for implementing reliability in codes rather than in specific projects was the following:

1) Design codes cover all applications of a specific structure type, e.g. fixed offshore platforms.

2) Engineers, designers and managers usually give closer attention to code changes than research findings.

3) Specific project application of reliability is not precluded by code adoption of reliability concepts. Rather, the latter may encourage individual firms especially for complex projects to expand and broaden the scope of their reliabilty analyses. Codes by nature, use a general data base, which may be too conservative or not even applicable for a particular venture. A reliability basis in a code may foster further risk studies after designers and review agencies have become more comfortable with reliability techniques.

The main attention in this paper will focus on implementing the reliability concepts into conventional design. The paper will describe some of the developments and proposed advantages of the recently developed draft API - LRFD - RP 2A alternative (6).

That is, a load and resistance factor design (LFRD) approach to design of fixed offshore platforms utilizing reliability concepts. This approach is now being reviewed as an alternative to the present, working stress document, API (American Petroleum Institute) RP 2A (7). Since the LRFD is intended for routine design checking it is important to understand the premises of such an approach. The steps in checking any structural design including platforms, ships, bridges, etc., include:

1) Select specified nominal loads including gravity, wind, wave, earthquake, etc.

2) Combine appropriate load cases and analyze for component force effects (moment, shear, etc.).

3) Check the design by comparing load effects with specified component strength equations.

In most existing specifications, a working or allowable stress approach (WSD) is used in which the nominal load effects are compared to the nominal strength divided by a specified safety factor. In relibility-based LRFD formats a series of unique load factors are selected for each type of load effect (gravity, seismic waves, etc.) and a series of unique resistance factors are specified for each type of component failure mode (bending, column, hydrostatic, etc.). The load and resistance factors are calibrated by code writers (not by designers!) to produce uniform or consistent reliability levels for each design check. The actual reliability development is transparent to users of LRFD specifications who utilize the factors in a deterministic checking mode (i.e. computerized and organized program for checking the large number of components in typical fixed platform structures).

Only the code writers and researchers deal with the reliability issues, namely, 1) defining the statistics of each variable in a load effect or strength (resistance) equation, 2) establishing the appropriate target reliabilities, 3) fixing the nominal loading and strength equations, and 4) selecting the load and resistance factors which best achieve the target reliabilities.

RELIABILITY ANALYSIS

A reliability model for a structure should incorporate safety margins and uncertainties in evaluating the risk to a component or system. Data often comes from different sources including environmental, material and analysis so a common method of incorporating statistical information is needed. The first step in the model must be a clear definition and description of the failure mode being controlled. For example, the failure of a brace, connection or pile in a fixed offshore structure will occur when the load effects exceed their corresponding resistance. In conventional structural design practice, only the single component strength is checked. The impact of the component failure on possible system consequences is discussed below.

The probability of component failure is

illustrated as shown in Figure 1. The risk is due to the (extreme) load frequency curve overlapping the strength curve. The model should be viewed as a situation in which the probabilities correspond to the worst loading case (annual or lifetime as appropriate). Thus, for each variable, the extreme load effect and resistance is sampled once and failure occurs if load exceeds strength. The probability of failure (overlap) will decrease if either: i) the mean margin of safety is increased, i.e. higher safety factors or: ii) the uncertainty in load or resistance is reduced, i.e. more data or better analysis. The latter impact helps to explain why evaluation of existing structures for possible rehabilitation can often be satisfied with lower safety factors than for new construction since presumably more information or data is available on performance and behavior of an existing structure.

The analysis of reliability may be carried out by defining a failure function, g, such that g < 0 denotes failure, or:

$$g = R - S \qquad (1)$$

where R is the resistance or capacity and S is the load effect. An exact solution for probability of failure, P_f, may be obtained if R and S are both assumed as normal distributions with respective mean values, R and S and coeficients of variations (cov), V_R, V_S. The cov is the standard deviation divided by the mean value. With these assumptions and data, P_f can be written as:

$$P_f = \Phi\ [-\bar{g}/\sigma_g]$$

$$= \Phi\left[-\frac{\bar{R}-\bar{S}}{\sqrt{\sigma_R^2+\sigma_S^2}}\right] = \Phi\left[-\frac{\bar{R}/\bar{S}-1}{\sqrt{V_R^2(\bar{R}/\bar{S})^2+V_S^2}}\right] \qquad (2)$$

where Φ denotes the standard normal distribution and $\bar{g}$ and σ_g are the mean and sigma of the safety margin, g. Thus, for example, a mean safety factor of 2.5 (= $\bar{R}/\bar{S}$), and covs of R and S of 13% and 30% respectively would lead to a failure probability of:

$$P_f = \Phi\left[-\frac{2.5-1}{\sqrt{(.13)^2\ 2.5^2 + .3^2}}\right] = \Phi[-3.39] = .00035 \qquad (3)$$

In general, normal distributions are not always applicable to both load and strength variables and further both R and S may in turn depend on several other random variables (e.g. gravity and wave loads, beam-column stability, combined hydrostatic, axial and bending strength, etc.). Thus, a generalization is needed to carry out realistic reliability analyses. These results are often described in terms of a safety index, (beta), which is developed from Equations 1 and 2, namely,

$$\beta = \bar{g}/\ \sigma_g \qquad (4)$$

i.e. β is the distance in terms of number of standard deviations (σ_g) from the mean ($\bar{g}$) of the safety margin to the failure region (g $\leq$ 0). In the general case, where g is a function of several variables we let

$$g = g(X_1, X_2, \ldots X_n) \quad (5)$$

Where X_1 to X_n are n different random variables. The function g is deterministic and represents the mechanics of the failure mode, namely failure occurs if a realization of the random variables X_1 to X_n causes g to be negative. If the function in Eq. 5, is linear and the X's are all normal, then Eq. 4 provides an exact expression for P_f using normal probability tables. If g is nonlinear, then the magnitude β should be obtained by an iteration which finds the point on the failure surface, g = 0, which is closest to the mean of g. This point, X_1^*, $X_2^*, \ldots X_n^*$, is often denoted as the design point. For nonnormal or correlated variables, a mapping can be used to introduce equivalent independent normal variables with the same frequency and probability distribution at the value of X^*. This leads to an iterative procedure for finding the safety index, β . It has been shown that if the g function is reasonably convex (i.e. not having a lot of indentations) and the variables X_1 to X_n are reasonably close to normal then the value of P_f taken from the computed safety index β will be close to the exact value. The latter must be found from numerical integration or extensive Monto Carlo simulation. Several studies by Baker and others have shown that for practical structural design problems that beta correlates well to the computed risk (8). Furthermore, any differences become even smaller when code calibration procedures as described below are introduced.

At present, there are several computer programs available which make it easy to obtain the safety index provided:

i) the random variables and their distributions are given (including statistical correlations, if available)

ii) a failure function, g, is known from equations of mechanics such that realization with $g < 0$ denotes failure.

Such programs are known in the structural reliability literature as advanced first order second moment (AFOSM) methods and are commonly used by structural code writing committees throughout the world (3). Recently, the author extended this program to model strength interaction models (such as combined hydostatic, axial and bending) in which a theoretical strength failure mode equation (g) is not available but rather is statistically fit from test data. In this instance, an additional "test" random variable must be defined whose statistics may vary in different regions of the intraction space. This work is described below.

CODE CALIBRATION

The checking format for introducing reliability-based design was described above as an LRFD check. This leads to a so-called level I type code in which the specified factors achieve on the average, the target reliability. Level II codes may be simply thought of as codes in which the factors can be adjusted by the designer to achieve a precise target safety index. Level III codes are defined as codes in which the reliability calculation is exact i.e., Monte Carlo or numerical integration instead of approximate AFOSM methods. To date, it appears that most code writing committees are utilizing level I procedures although applications differ in the number of factors utilized. In typical LRFD applications in the U.S., the checking format appears as:

$$\phi R = \gamma_D D + \gamma_L L + \gamma_W W \qquad (6)$$

where R, D, L and W are the nominal resistance and nominal dead, live and environmental effects, respectively. ϕ is a partial resistance factor and depends on the type of strength mode being checked e.g., member bending, connection, pile, etc. γ is the load factor appropriate for each load type, e.g., dead, live and environmental. Each γ and ϕ is selected by the code committee in a calibration process which produces the target betas. These depend on: i) The target beta, ii) the corresponding uncertainty of the variable associated with the factor R, D, W, etc. and iii) The bias value. Bias is defined as the rate of the mean divided by the nominal value. Thus, if a code committee specifies a nominal design different from the mean value this affects the selection of the corresponding factor. For many traditional reasons, codes committees often specify nominal values such as yield stress (e.g., 36 ksi for A36 grade steel) or 100 year wind loads which are different from their corresponding mean values.

It should be noted that some code committees outside the United States prescribe additional factors than those contained in the LRFD. These "partial" factors may also account for load combinations, different materials (steel or concrete), quality control, failure consequences, etc. These applications have not yet been standardized and are to the authors knowledge not under consideration for any code in the U.S. although some Canadian codes have adopted partial safety factor concepts as a goal. The process of selecting factors is often denoted as code calibration.

The philosophy utilized in the draft LRFD RP 2A for fixed platforms differs somewhat from other codes (9). Target reliabilities are based on implied past performance history rather than on an actuarial risk based on economic factors. The reasoning in this action is as follows. Data on loads and resistances for fixed offshore structures is still rather limited. Extensive data on storm occurences is only available in a few instances. Detailed structure responses have been measured on only a single platform test structure in the Gulf of Mexico. Component strengths especially for pile foundations have been modelled from only a few tests usually on a reduced scale. To account for possible differences between full scale and model and other limits in the data, performance history must be used to guide the selection of target betas. The advantages in LRFD are still maintained, namely more uniform reliability than existing allowable stress methods. Thus, the calibration process operated as follows.

1) Assemble a representative sample of members from different fixed platform types, locations, water depths, steel grades, etc. In addition, develop a corresponding generic range of members to cover most possible applications.

2) From available statistical data and corresponding load and strength models, develop the best available risk model. This includes failure mode (g) equations, bias values, coefficients of variation and frequency distributions (e.g. normal, lognormal, etc.).

3) For the range of platform members, compute the corresponding safety indices (β) using existing design checking equations. Since platforms now in place utilize working stress methods, the sections should be checked with an RP 2A WSD format, namely:

$$R = (D + L + W)/\text{Safety Factor} \qquad (7)$$

For purposes of comparison, the members should have interaction ratios, I.R., equal to 1.0 i.e. utilization at maximum allowable values based on WSD format. This gives us an evaluation of safety indices for the in-place performance history.

4) From the betas obtained in step 3, a target beta is extracted. Typically, we find that with WSD the betas are not uniform over the sample. For example, betas are typically much higher for gravity load cases and lower for environmental loading espically when the latter include the one-third increase in allowable stress typically found in many codes including RP 2A. The target beta is usually selected as the average beta found in the WSD. If, however, we feel the target is either too high or too low, it is adjusted. Since the target is selected individually for each type of failure mode, the different target betas should obviously not deviate too much. Average betas from existing designs should also be considered acceptable if there is no field performance data to indicate that either we are presently too conservative or not conservative enough.

5) The final step is to select the specified load and resistance factors which reduces the scatter in beta over the applicable range. Absolutely uniform beta is not possible using a limited number of load and resistance factors, since the identical load factors must be applied to all types of component checks.

Figure 2 illustrates the calibration of the bending strength term, where β is plotted over a large range of the ratio of environmental to gravity load effect. The β's with WSD are seen to have a large difference or range in upper and lower value compared to the betas for LRFD. The average β values are similar for WSD and LRFD especially in the more commonly occurring high environmental load ratios.

In addition to calibration over the load effect ratios, the calibration must consider other parameters such as diameter to thickness (D/t) for tubulars or effective length parameter, $\lambda = 1/\pi \sqrt{F_y/E}\, kL/r$. In some instances, this requires an adjustment by trial and error of the nominal resistance equation. For example, Figure 3 shows the mean strength for local buckling strength based on test data assembled by J. Cox (10). It was concluded from review of the data that V_R was constant over the geometrical parameter D/t. Thus, a single resistance factor provides a uniform reliability for all values of D/t as long as the nominal strength curve is selected to have a uniform bias.

Figure 4 shows the mean strength for axial loaded tubular members, again with test data compiled by J. Cox (10). Because the buckling changes from inelastic to elastic for higher values of λ, V_R varies considerably. Thus, the nominal strength equation to be used by the designer should not have a uniform bias (bias = mean/nominal). Figure 4 also shows the nominal strength selected to provide uniform reliability over all λ values.

INTERACTION EQUATIONS

A large proportion of strength checking involves multidimensional interaction equations. Examples for tubular members typical of offshore practice include, a) beam-column strength, b) beam-column stability, c) connection axial and bending loads, d) hydrostatic and axial tension, e) hydrostatic plus axial compression and bending. In general, the strength equations can be modelled by weak interaction - type formula such as parabolic or cosine shapes, while stability phenomena may show greater interaction and be better described by a linear interaction. In the last few years there have been several test programs worldwide sponsored by API, Joint Industry programs (JIP), Universities and foreign research agencies especially aimed at data for combined loadings including hydrostatic. An example of this reliability modeling to produce a nominal strength equation and corresponding resistance factors to achieve uniform reliability is described herein. The data for the combined hydrostatic, axial compression and bending is described in a report by John Cox (10). Using this test data a normalized three dimensional formulation can be created in which the three axes are:

a) X = axial load/axial strength capacity

b) Y = bending load/bending strength capacity and

c) Z = hydrostatic load/hydrostatic strength capacity

The advantage of this approach is that there can be a continuous checking model across the different types of failure modes. At present, RP 2A has been shown to lead to discontinuity when moving from the tubular strength equations without hydrostatic to the inclusion of hydrostatic. For example, a check of one platform including the hydrostatic term showed that even members near the waterline with very little hydrostatic pressure were actually controlled by hydrostatic checks. This anomoly is removed by the procedure developed in reference (11) described herein. Further, this same formulation could also be used for combined hydrostatic and column instability (long columns) after current research tests on such components are completed.

Using the available test data, the best fit (mean) curve to the combined test data was (11):

$$1.02\ X^2 + .57\ Z^2 - .44\ X\ Z + Y = 1 \quad (8)$$

Reviewing this data, the author estimated the bias and scatter (cov) of the data in different regions of the 3-D (X,Y,Z) space. The next step is to develop a nominal design check. The simplest approach is to use the same resistance factors on bending, axial and hydrostatic resistance that are proposed in the LRFD code for the cases when these terms are considered separately. This leads to a checking equation of the form:

$$1.02\left[\frac{X_n}{\phi_c}\right]^2 + .57\left[\frac{Z_n}{\phi_h}\right]^2 - .44\ \frac{X_n}{\phi_c}\ \frac{Z_n}{\phi_h} + \frac{Y_n}{\phi_b} = \text{I.R.} \quad (9)$$

where the subscript "n" denotes the nominal value. The resistance factors, ϕ_c, ϕ_h and ϕ_b are for tubular member compression, hydrostatic and bending respectively.

The betas that would be obtained with this formulation were checked by selecting a range of designs which satisfy the design check with a 1.0 interaction ratio. The load is taken as the combined dead, live and wave load with parameters consistent with the remaining LRFD development, namely,

dead load, Bias = 1.0, V_D = .08, load factor = 1.1

live load, Bias = 1.0, V_L = .14, load factor = 1.1

Wave load, bias = 0.7, V_W = 0.37, load factor = 1.35

and hydrostatic, Bias = 1.0, V_H = 0.05, load factor = 1.3.

The parameters for wave load were based on detailed calibrations including oceanographic storms and using data from the OTS study. Similarly, the resistance data was taken as:

axial (load buckling) Bias=1.185, V_R=.12, ϕ_c = 0.9

bending, Bias=1.26 , V_R=.11, ϕ_b = 0.92

and hydrostatic Bias=1.05 , V_R=.11-.15, ϕ_h = 0.88

With this data, the betas were found using an AFOSM program. The failure function was:

$$g = T\ (X,Y,Z) - [1.02\ X^2 + .57Z^2 - .44\ XZ + Y] \quad (10)$$

The variable T accounts for the test or data scatter mentioned above. Since the parameters of T are discontinuous, the beta program had to be altered to modify the parameters of T as the search proceeded for the design point (X^*). The steps are explained in more detail elsewhere (12). Examples of beta calculations are given in Table 1 . While not as uniform as may be desired, the values are much more uniform than existing RP 2A design (12). Further improvements in the nominal design formula are underway by J. Cox (10) and the author to smooth any of the variations in betas in the LRFD format. This will be accomplished when more recent test data is evaluated.

SENSITIVITY STUDIES

In general, after code calibration, it is a good idea to reevaluate the derived load and resistance factors whenever we lack sufficient statistical data. In those instances we make subjectives estimates regarding some of the parameters in the statistical distribution. The lack of sufficient data makes it difficult to approach code development from a strictly actuarial approach to risk in which some optimum or idealized risk target must be satisfied. Furthermore, most reported structural failures are not due to extreme loads exceeding design resistances but rather occur due to poor judgement, blunders or inadequate theories or technology. For example, in the offshore area, component failures have occurred due to collisions, fires, human errors or inadequate investigation on site information. Thus, structural research often refer to the probability, P [R < S], as notional probabilities of failures. Although not exact in a true actuarial sense, they are important in allocating resources to different components of a structural system including connections, steel members, foundations, or even inspection, peer review or quality assurance.

To account for limitations in available statistical data, we may use a calibration philosophy. This implies that if we are satisfied with present notional rates of failure we should use these average "computed" or notional risks expressed in terms of betas as the target for LRFD calibration. Even though the average risk may be the same as WSD, the advantages of LRFD are to limit the scatter in component risks that occur in WSD which may bring some members into an unacceptable high probability of failure. At the same time, LRFD should avoid members with such high betas that they are uneconomical. Other advantages of LRFD are summarized below.

The process of checking whether changes in input statistical data affects the LRFD factors is called code sensitivity. An example of a member calibration sensitivity is illustrated in Table 2. The resistance factor was calibrated at 0.92 to give LRFD similar average betas as the WSD, namely 2.8-2.9 range. The best estimate for the resistance of this element (tubular bending) was a bias of 1.26 to account for strain hardening and mean yield to nominal ratio and a cov of .11. Table 2 also shows two senstivity illustrations. Columns 3 and 4 show what happen if the bias is actualy 1.5. The average betas for both WSD and LRFD increase but notice how similar they still are. The same ϕ of 0.92 is used for LRFD based on the original data. Similarly, columns 5 and 6 show what happens if V_R is 15% instead of 11%. Again, the average betas are the same, showing that we are consistent with calibration philosophy. Note that in all cases, the LRFD average beta remains about 0.1 below the average WSD beta and most importantly the range or scatter in betas is much smaller for LRFD compared to WSD. This is a major advantage of LRFD compared to present codes which use a WSD philosophy. The advantage is realized in LRFD because of the additional "tuned" factors (load and resistance) compared to only a single safety factor in WSD.

The same sensitivity process was also considered in the LRFD development for platform environmental load factor for wind, wave and current (11). The best estimates of environmental parameters as cited in the example is a cov of 37% and a bias of 0.7 with respect to the nominal which is taken as a 100 year return period effect. The bias is based on typical Gulf of Mexico conditions including drag dominated structures, 20 year life and assumed wave kinematics and wave height uncertainties. A sensitivity study was conducted which is reported in reference (11). It used a general wave force model of the form:

$$F = AH^{\alpha} \qquad (11)$$

where F = global response

A = analysis variable

H = maximum lifetime wave height

α = wave height exponent

The statistical parameters for H depend on site location and platform life while the parameters for A depend on platform type. α reflects the location of the component in the structure and whether it is drag dominated ($\alpha = 2$) or inertial dominated ($\alpha = 1$) or if located near the waterline (α may exceed 3).

The betas were calculated for a broad range of components and failure modes. The sensitivity study reported that the 1.35 load factor recommended in the LRFD draft did provide consistent reliabilities over a wide range of applications. The reason for this is that the nominal load is selected as a 100 year recurrence interval and hence its selection is also effected by any changes in input statistical data. The study showed the possible situations in which the environmental load factor may have to be modified in terms of expected platform life, site location or wave force coefficient. For example, the results show that a common rule of using recurrence times of 5 times the expected life will not lead to uniform reliability for the case of very short exposures - under ten years and that for such cases the entire calibration procedure must be repeated to derive an appropriate γ_w.

SYSTEM RELIABILITIES

Many structures can tolerate relatively high component risks, say above 10^{-3}, provided the structure possesses system reserve strength. That is, the system reliability against large damage or collapse will greatly exceed the component reliability. Much has been written about this subject and the importance of such activities as identification of system modes of failure, selection of significant modes and computing their probabilities of faliure and combining modes into an overall risk (13). Some of the known conclusions relate to idealized models of behavior including parallel components, series members, ductile behavior and completely brittle components, General programs to model complex systems were developed by Moses and Stahl with support from industrywide participation (14). Recently, there is a JIP study by C.A. Cornell which seeks to develop programs for assessing system reliability (15). To illustrate the system importance, two examples are presented herein, including series and parallel examples. Fig. 5 shows a ten member series model in which any single failure causes overall collapse (chain-model). The beta for the element is set at 3.0. The system reliability is calculated to be only about 2.3. This is based on an assumption that strengths are uncorrelated. If there is strong strength correlation, then the member and system reliabilities approach each other. For the parallel model, the system influence is also apparent although the trends and influences of correlation may be opposite to the series model. The most important feature of a parallel system, however, is not the case of equal load sharing. In this latter case the system beta increases to 3.9 when the member beta is 3.0 for the case shown in Figure 6. If the member strengths are perfectly correlated then the system reliability <u>decreases</u> (opposite to the effect in a series). The reason in that member strength independence in a parallel model reduces overall capacity uncertainty.

In the case of unequal load sharing, the system impact is much more significant. For example, if the sharing in Figure 6 is .15,.35,.35 and .15 for a four member system, then the system beta is 6.2 compared to 3 for the worst element. Such unequal load sharing is typical of redundant structures such as offshore platforms.

The reasons for system reserve may include: 1) a need to build structures with symmetry, 2) structures are usually checked for loads in all directions (thus each brace may actually be checked for a compressive load, even if it is in tension under the critical load case), 3) fabrication and erection requirements control many members rather than environmental loading. Thus, system reserve if properly modelled and accounted may play a major role in the reliability. It requires redundancy <u>and</u> a condition where the multiple parallel load paths are <u>not</u> simultaneously fully loaded by the design case. Further, two considerations must be present to achieve full system benefits, 1) component failure must be ductile (any brittle behavior or loss of capacity will reduce system reserve) and 2) secondary members which come into play only when load path distributions are changed must have sufficient capacity to carry any required loads. These load effect changes are usually not apparent in checking nominal loading with the intact structural model.

CONCLUSIONS

The adoption of LRFD is a first step towards rationalization of design practice using reliability methods. The advantages of a reliability basis for selection of design code criteria include the following:

1) Greater uniformity for LRFD in platform

component reliability than existing WSD.

2) More effective distribution of material than WSD

3) Explicitly allows an accounting for randomness and uncertainties in engineering parameters.

4) An LRFD format concentrates the reliability modelling in terms of a safety index and avoids explicit risk and probabilistic assessments by designers.

5) The format is capable of being logically and consistently modified to account for different conditions (geographic locations, platform type, exposure period, etc.).

6) Encourages and eases the incorporation of new experimental information.

7) Interfaces with other related design LRFD documents for steel (AISC) and concrete (ACI) which may also use LRFD formats.

8) Helps focus research activity on those areas having the greatest impact on both risk and economy i.e., where subjective uncertainties are large and can be reduced by further research which is implemented in terms of less conservative factors.

9) Capable of bridging the gap between design procedures and evaluation of remedial measures for existing structures.

10) Capable of incorporating overall system capacity margins and highlighting the needs and benefits of redundant load paths.

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TABLE 1
BETA VALUES FOR COMBINED
HYDROSTATIC, AXIAL AND BENDING

HYDROSTATIC	BETA				
Z - Nominal	Nominal Axial - X*				
	0	.2	.4	.6	.8
0	2.77	2.65	2.55	2.48	2.52
.2	2.96	2.77	2.67	2.50	2.58
.4	3.26	2.97	2.74	2.61	2.63
.6	3.17	5.30	3.03	2.89	2.76

*The nominal value of Y is selected to make the I.R. value in Eq. 9 equal to 1.0. Similar results fixing Y and solving for nominal X are also reported in reference 12.

TABLE 2
ILLUSTRATION OF CALIBRATION
SENSITIVITY - BENDING COMPONENT

	BETAS					
	(1)	(2)	(3)	(4)	(5)	(6)
ENV/GRAV LOAD RATIO LRFD	BASE CASE*-WSD	BASE CASE*-LRFD	NOTE a	WSD	LRFD	NOTE b WSD
2	3.21	3.01	3.77	3.58	3.03	2.83
5	2.94	2.86	3.45	3.37	2.81	2.73
10	2.85	2.81	3.33	3.29	2.72	2.69
20	2.79	2.78	3.27	3.25	2.68	2.66
40	2.77	2.76	3.24	3.23	2.66	2.65
AVERAGE β :	2.91	2.84	3.41	3.34	2.78	2.71

*DATA: BASE CASE: $\gamma_D = 1.1$, $\gamma_L = 1.1$, $\gamma_W = 1.35$, $\phi = .92$
$V_D = .08$, $V_L = .14$, $V_W = .37$, $V_R = .11$
$B_D = 1.0$, $B_L = .10$, $B_W = 0.7$, $B_R = 1.26$
S.F. (WSD) = 1.5 x 3/4 x 1.3 to account for one-third increase and plastic to elastic section capacity (1.3),
$L_n/D_n = 3$;
Gravity = $L_n + D_n$

Note a: Same data as base case except $B_R = 1.50$

Note b: Same data as base case except $V_R = .15$

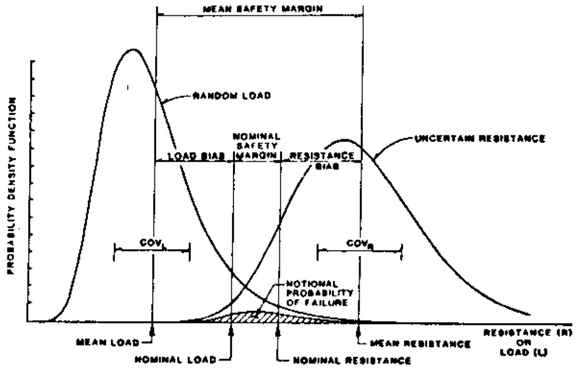


ILLUSTRATION OF MEANS, BIASES AND SAFETY MARGINS OF LOAD AND RESISTANCE
FIGURE 1

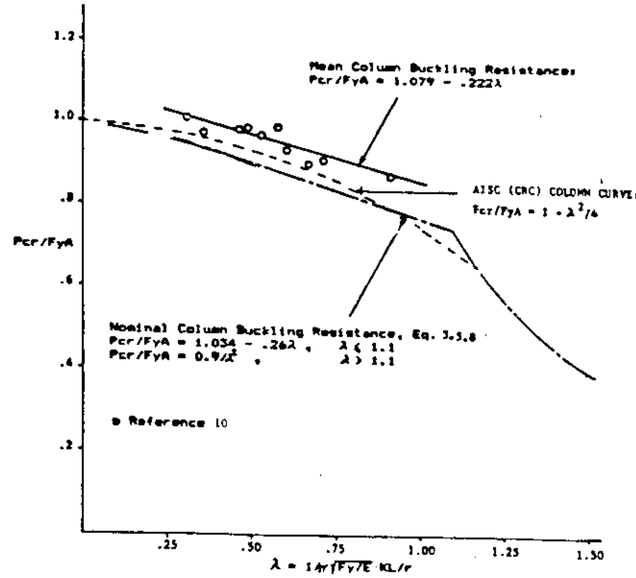


Fig. 4
TEST DATA, MEAN RESISTANCE, AND NOMINAL RESISTANCE FOR COLUMN BUCKLING OF FABRICATED STEEL CYLINDERS

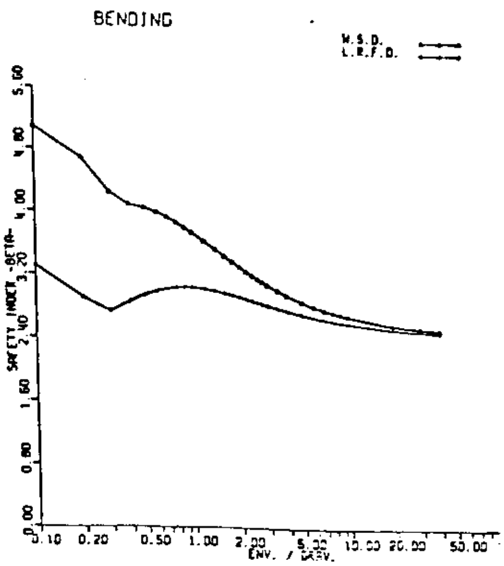


Figure 2

Example of safety index for LRFD and present design (WSD) plotted vs. component venironment to gravity load ratio (Ref. 11)

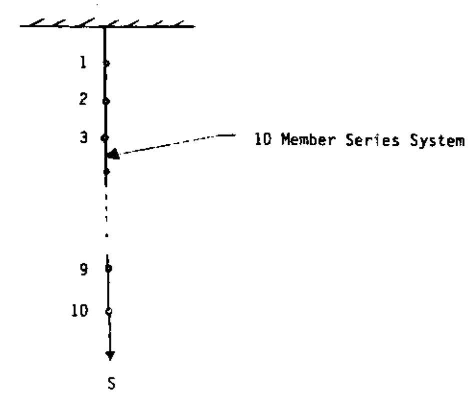
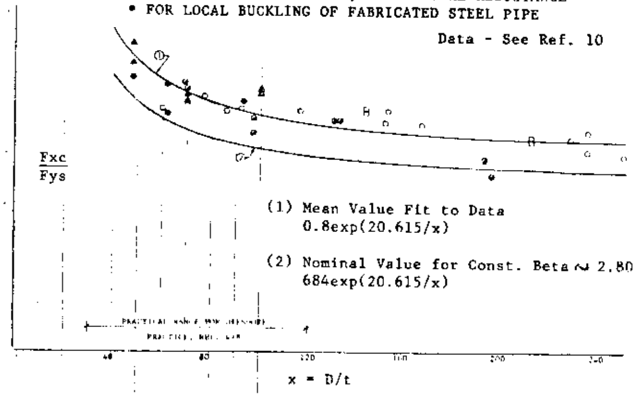


Figure 5: Series Model - 10 Elements
Resistances, R_1 to R_{10}, V_R = .13
Load, S, V_S = 25%

FIG. 3

TEST DATA, MEAN RESISTANCE, AND NOMINAL RESISTANCE
• FOR LOCAL BUCKLING OF FABRICATED STEEL PIPE



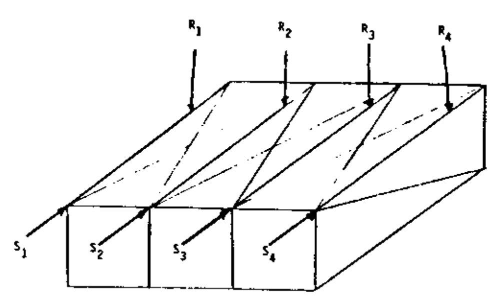


Figure 6: Parallel Strength Model
Resistances: R_1 to R_4, V_R = 13%
Load: S (Shared by Elements as S_1, S_2, S_3, S_4) where $S_1 + S_2 + S_3 + S_4 = S$
V_S = 25%

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