

Probability-Based Safety Checking of Nuclear Power Plant Structures

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Abstract

The basis for the development of practical probability-based criteria for the design of nuclear plant structures is described. Existing criteria are reviewed critically in order to highlight those features of probability-based limit states design that are especially advantageous. Design load combinations and strength criteria are developed using code optimization techniques. These criteria are consistent with code performance objectives that are stated in probabilistic terms. The selection of a set of structures to test the validity of the design equations also is described.

1. Introduction

Structures in nuclear power plants must be designed to withstand the effects of loads that arise from normal operating conditions, from the environment, and from postulated accidents. Codes and standards used for design and regulatory purposes [e.g., 1, 2] include sets of safety checking equations that are intended to ensure that the structural performance requirements of the plant are met. These checking equations incorporate factors of safety in the form of allowable stresses or load and resistance factors to account for uncertainties in the strength of materials, fabrication, the operating environment, and structural modeling. Heretofore, the selection of these factors of safety has been largely subjective. However, recent advances in structural reliability theory, probabilistic design, and an expanded data base [3-6] make it feasible to develop practical load combinations rules and resistance criteria that are consistent with code performance objectives stated in probabilistic terms.

2. Review of Existing Bases for Design

Several codes and standards currently in use [e.g., 1] are based on working stress design principles, in which the effects of various combinations of nominal loads are checked against an allowable stress. This allowable stress is a preassigned fraction of the stress at which failure occurs by yielding, fracture or buckling. Different stress limits, which depend on the severity perceived to be associated with a particular load combination and limit state, may be provided for different load combinations. While working stress design usually has served the profession well for traditional construction, it has a number of fundamental shortcomings [4, 5]. For example, working stress (elastic)

analysis usually is not relevant as a measure of structural behavior in circumstances where nonlinear behavior may be significant. Structural research is oriented toward determining the strength of components, connections and systems. These research data must be related to linearly elastic behavior under working loads, often in an artificial manner, before they can be used in working stress design. In contrast to working stress design, strength design methods such as those specified for the design of noncontainment concrete structures [2] enable data on structural behavior to be incorporated directly into design codes with a minimum of adjustment. Rather than using multiple-level stress checks, the factored resistances are the same for all load combinations and limit states.

The selection of allowable stress factors, load and resistance factors, and design load combinations for nuclear plant structures has been largely subjective and has relied heavily on experience with ordinary building construction. We find that higher load factors have been applied to those loads for which the probability of exceeding the design load is higher; load factors of 1.0 are used with loads that have extremely low probabilities of occurrence; a load factor greater than 1.0 may be specified when the load affects a large portion of the structure rather than only a local part; the 25 percent reduction in total load effect is permitted for combinations involving thermal stresses because they tend to be self-relieving. These extensions of design criteria developed for ordinary building construction to nuclear plant structures, where experience with structural performance during extreme events is limited or nonexistent, is questionable. Moreover, from a reliability viewpoint, existing criteria (especially working stress criteria, in which the loads in the combinations frequently are unfactored) may lead to nonuniform probabilities of unacceptable performance for similar design situations.

3. Probability-Based Limit States Design

Recent advances in structural code development have been made within the context of limit states design [4, 5]. A limit state is a condition at which the structure ceases to perform its intended functions in some way. Ultimate or safety-related limit states pertain to structural behavior during extreme conditions, while serviceability limit states pertain to performance under service conditions. Limit states design is a behavior-oriented performance checking method which forces the designer (or, at least, the code writer) to consider explicitly and make provision for all likely conditions of adverse structural behavior.

Structural reliability theory provides the basis for safety evaluation methods and design checking procedures [4, 5] in probability-based limit states design. The reliability analysis requires, first of all, a set of mathematical models derived from principles of structural mechanics and/or experimental data which relate the resistance and load variables for the operating, environmental and accidental loading conditions of interest. For the k th limit state, we would have,

$$g_k(\underline{X}) = g_k(X_1, X_2, \dots, X_m) = 0 \quad (1)$$

in which X_i are (generally) random resistance and load variables. By convention, the limit state is attained and failure occurs when $g_k(\underline{X}) < 0$. If the joint probability density of \underline{X}

is known, the limit state probability is obtained by integrating over that region of \underline{x} in which $g_k(\underline{x}) < 0$:

$$P_F = \int \dots \int_{g_k(\underline{x}) < 0} f_{\underline{x}}(x_1, x_2, \dots, x_m) dx_1 dx_2 \dots dx_m \quad (2)$$

Because of complexities in $g_k(\underline{x})$ and in the multidimensional integration, approximations to P_F in eq. (2) frequently must be sought [3].

In contrast with the reliability analysis of a specific structure, reliability-based design begins with a code performance requirement stated in probabilistic terms:

$$P_F(\underline{\theta}) \leq P_{F_0} \quad (3)$$

in which $\underline{\theta}$ is a vector of nominal or characteristic design variables (material strengths, dimensions, loads),

$$\underline{\theta} = (F_y, f_c, A_s, \dots, D, E_s, P_d, \dots) \quad (4)$$

and P_{F_0} is a target reliability. The design objective is to define a structure or set of structures in terms of $\underline{\theta}$ so that $P_F(\underline{\theta})$ calculated for each structure using eq. (2) is approximately equal to P_{F_0} . To do this in a practical manner for design and code purposes, the performance requirement given by eq. (3) must be replaced by a set of safety checking equations which have a conventional appearance. For strength limit states, we would have,

$$\text{Design Strength} = R_d \geq U_d = \text{Effect of Design Loads} \quad (5)$$

from which the structural details can be calculated. The format for the design strength and design loads in eq. (5) is arbitrary, and should be chosen such that any structure that falls within the scope of the code has a limit state probability calculated by eq. (2) that is approximately equal to P_{F_0} .

A review of alternate formats [5] suggests that an appropriate format for the design loads is,

$$U_d = \gamma_D D + \gamma_1 X_{ni} + \sum_{j \neq 1} \gamma_j X_{nj} \quad (6)$$

in which $\gamma_1 X_{ni}$ is denoted the principal factored nominal load in the combination and $\gamma_j X_{nj}$ are factored point-in-time (companion) nominal load values to be combined with $\gamma_1 X_{ni}$. Separate load factors in eq. (6) facilitate setting the design loads so that the probability that their combined effect is exceeded is uniform for different load combinations. In theory, each variable load must be positioned, in turn, as the principal variable load in order to determine the maximum combined load effect; in practice, only a limited number of load combinations need be specified. Within the basic context of eq. (6), dynamic effects still might have to be combined using some rule such as the square root of sum of squares.

For example, for strength limit states it might be required that the design strength for combinations of operating and environmental loads should not be less than the maximum of [5]:

$$\gamma_D D + \gamma_L L + T_o + \gamma_R R_o \quad (7)$$

$$\gamma_D D + \gamma_L L + (\gamma_E E_s \text{ or } \gamma_W W \text{ or } \gamma_{Wt} W_t) + T_o + \gamma_R R_o \quad (8)$$

$U_d =$

$$\gamma_D D + \gamma_L L + T_o + \gamma_R R_o \quad (9)$$

$$\gamma_D D + \gamma_L L + (\gamma_E E_s \text{ or } \gamma_W W) + T_o + \gamma_R R_o + \gamma_{SRV} (SRV) \quad (10)$$

For combinations involving accidental or abnormal loads, the design strength should not be less than the maximum of,

$$D + \gamma_L L + \gamma_E E_s + \gamma_A (P_a + R_a) + T_a \quad (11)$$

The nominal loads in eqs. (7) - (11) are those defined in existing standards and regulatory documents [1-2]. The SRV is the safety relief valve discharge load in BWRs. Loads and effects due to weight or lateral pressures of fluids or soils with well-defined densities should be included with the permanent load, D. Permanent equipment loads need not be factored [6]; moveable equipment loads should be included with live load, L. Only one nominal design earthquake has been specified, and that value E_s is equivalent to the current SSE. The OBE and SSE are different manifestations of the same event, and from a reliability viewpoint, one nominal load is sufficient. The effect of forces of constraint due to prestressing, temperature gradients, creep and other such effects depends on the structural stiffness and on the limit state. If the structural behavior is ductile and forces can be redistributed by inelastic action as the limit state is approached, the effect of constraint forces is benign and a load factor of 1.0 can be assigned to them. However, in some cases, this may not be appropriate. Although forces of constraint may not affect the ultimate strength, they always should be considered in computing structural deformations at service load levels.

The "design loads" portion of eq. (5) should be the same for all safety-related structures within the plant, regardless of whether they are of steel or reinforced concrete. The use of common load requirements would simplify the design and regulatory process, reduce the chances of design error, facilitate subsequent periodic revisions of the codes, and encourage uniformity in reliability for alternate plant designs.

Similarly, the design strength in eq. (5) can be specified as the product ϕR_n , in which the overall resistance factor ϕ is applied to the nominal structural resistance R_n in a particular limit state. Resistance factor ϕ accounts for uncertainty in strength and for the mode and consequence of failure. Alternatively, partial factors of safety can be applied to the most significant nominal strength and dimension variables in the limit state equations to reflect the variability in these individual parameters. For example, in reinforced concrete design, the design strength is computed using the values $f_y^* = \phi_y f_y$,

$f_c^* = \phi_c f_c$, $d^* = d - \Delta$, etc., in which ϕ_y and ϕ_c are nondimensional partial factors on strength and partial factor Δ has dimensional units. The same partial factors are used for all safety-related limit states. Each approach to specifying design strength has advantages and disadvantages [5]. Regardless of which format is selected, these strength criteria should depend on the type of construction and on quality control, but should not depend on the load combinations. Deformations within the structural system that are problems at operating or abnormal load levels should be identified and dealt with explicitly as separate limit states.

With the design format determined, the vectors of partial factors (ϕ , γ) can be selected so that structural components designed using the proposed format have, on the average, acceptable limit state probabilities. This selection is performed by mathematical optimization to determine (ϕ , γ) that minimize the function,

$$I(\phi, \gamma) = \sum_i w_i (\ln P_{Fi} - \ln P_{Fo})^2 \quad (12)$$

in which P_{Fi} is the limit state probability computed from eq. (2) for the *i*th structure or component designed using (ϕ , γ), P_{Fo} is the target probability, w_i is a weight that defines the relative importance of the *i*th component among all components likely to be designed using the code, and N is the number of test structures selected for the optimization.

The selection of an appropriate target, P_{Fo} , must be done with care. Reliability based criteria for ordinary construction have been calibrated to design procedures that have proved satisfactory in the past [4]. The process of code monitoring and adjustment that has been used to adjust reliabilities inherent in conventional construction is more difficult to apply to nuclear plant structures because there is limited experience with their performance during extreme events. Nevertheless, it appears that reliability analyses of existing plant structures are necessary, even though their significance in actuarial terms may be difficult to assess, to provide a starting point for developing design rules to achieve uniform reliability.

4. Selection of Test Structures

A representative set of structures must be designed using the proposed requirements and their limit state probabilities must be calculated and compared to the target using eq. (12) to ensure that the design equations meet the reliability objectives of the code.

As an illustration, a set of concrete containments used for this validation can be defined generically by a set of descriptors, summarized according to [5, 6]:

Type: Pressurized water reactor, boiling water reactor

Construction: Deformed bar reinforcement, prestressing

Geometry: Diameter of containment, thickness of containment wall

Material strengths: Concrete strength, reinforcement strength

Nominal loads: D , P_a , E_g , etc.

Detailing: Amount and arrangement of reinforcement.

A review of the criteria used in the structural design of existing and docketed containments in the United States establishes the relative frequencies in practice for each of these descriptors [5, 6]. The necessary N generic containments needed for validation purposes then can be selected by sampling randomly from the distributions of the basic

descriptors or by using a Latin Hypercube sampling plan. Each containment so selected must be designed using a trial load and resistance factor set, limit state probabilities are evaluated using eq. (12), and (ϕ, γ) are adjusted until $I(\phi, \gamma)$ is minimum [6]. This minimum value is a measure of the ability of the design equations to achieve the performance objectives of the code. If the minimum is too large, the scatter in P_F about P_{FO} can be reduced by either selecting another format with additional partial factors or reducing the scope of application of the design criteria.

5. Concluding Remarks

The purpose of code safety checking equations (probability-based or otherwise) is to guard against the consequence of unfavorable combinations of loads and resistance that may occur even when due care is taken in analysis, design and construction. Failures induced by human error or by negligence may also be significant for the overall reliability of nuclear plants. However, experience has shown that adjustments to the partial factors of safety used in design have very little effect in reducing failure rates due to these causes. It generally is more efficient to account for negligence or gross error by improving procedures for quality assurance and control.

6. References

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7. Acknowledgment

This material is based, in part, on work supported by the U.S. Nuclear Regulatory Commission. Any opinions, findings and conclusions expressed in this article are those of the author and do not necessarily reflect the views of the USNRC.