Proceedings of the ASME 2011 Pressure Vessels & Piping Division Conference
PVP2011
July 17-21, 2011, Baltimore, Maryland, USA

PVP2011-57600

STRUCTURAL DESIGN CHALLENGES IN DESIGN CERTIFICATION APPLICATIONSFOR NEW REACTORS

Manuel Miranda
Joseph Braverman
Xing Wei
Charles Hofmayer
Nuclear Science and Technology Department
Brookhaven National Laboratory
Upton, New York 11973-5000
Email: mmiranda@bnl.gov

Jim Xu
Office of New Reactors
U.S. Nuclear Regulatory Commission
Washington, DC 20545-0001

ABSTRACT

The licensing framework established by the U.S. Nuclear Regulatory Commission under Title 10 of the Code of Federal Regulations (10 CFR) Part 52, "Licenses, Certifications, and Approvals for Nuclear Power Plants," provides requirements for standard design certifications (DCs) and combined license (COL) applications. The intent of this process is the early resolution of safety issues at the DC application stage. Subsequent COL applications may incorporate a DC by reference. Thus, the COL review will not reconsider safety issues resolved during the DC process. However, a COL application that incorporates a DC by reference must demonstrate that relevant site-specific design parameters are confined within the bounds postulated by the DC, and any departures from the DC need to be justified.

This paper provides an overview of structural design challenges encountered in recent DC applications under the 10 CFR Part 52 process, in which the authors have participated as part of the safety review effort.

INTRODUCTION

INTRODUCTION

For the current fleet of nuclear power plants (NPPs), the U.S. Nuclear Regulatory Commission (NRC) issued operating licenses after plant construction was completed, under the licensing framework established by Title 10 of the *Code of Federal Regulations* (10 CFR) Part 50, "Domestic Licensing of Production and Utilization Facilities" [1]. Therefore, the analysis and design of NPP structures, systems, and components (SSCs) were performed using site-specific design parameters based on actual site investigations. The SSC analysis and design typically followed the design acceptance criteria of NUREG-0800, "Standard Review Plan for the Review of Safety Analysis Reports for Nuclear Power Plants: LWR Edition" (SRP) [2], which the NRC staff uses for performing its safety reviews.

For new reactors, the licensing framework established under 10 CFR Part 52, "Licenses, Certifications, and Approvals for Nuclear Power Plants" [3], provides requirements for standard design certifications (DCs) and combined license (COL) applications. The intent of this process is the early resolution of safety issues at the DC application stage. Subsequent COL applications may incorporate a DC by reference. Thus, the COL review will not reconsider safety issues resolved during the DC process. However, a COL application that incorporates a DC by reference must demonstrate that relevant site-specific design parameters are confined within the bounds postulated by the DC, and any departures from the DC need to be justified.

In a DC application, the analysis and design of SSCs need to postulate design parameters that are bounding for future potential sites. Therefore, design parameters (e.g., seismic ground motions, environmental loadings, soil conditions) are often conservatively specified, representing envelopes of the site-specific characteristics to be expected at potential sites. This poses certain challenges to the structural design, since design demands tend to be overestimated. In addition, the lack of specific design parameters may lead to a design approach based on postulated worst-case scenarios, which may appear to be overly conservative but are necessary to establish design bounds. This paper provides some examples of these challenges related to the design of NPP structures and foundations and to stability evaluations. It is hoped that a better understanding of these design challenges will contribute to the improvement of future DC and COL applications, as well as facilitate NRC safety reviews.

STRUCTURAL DESIGN CHALLENGES

This section discusses four structural design topics that have been found to be particularly challenging in recent DC applications. In the first two topics, related to the structural design for differential settlements and construction sequence and to seismic stability evaluations, the challenges arise because of the lack of site-specific design parameters at the DC stage. The third topic, related to the methodology for selecting critical sections, is unique to the DC application process and has only recently been addressed in a systematic manner. The final topic, related to the combination of multiple interacting structural response quantities when three directions of seismic load are present, has been the source of some confusion in several recent DC applications.

Differential Soil Settlements and Construction Sequence

In accordance with SRP Section 3.8.5, Revision 3, seismic Category I structures (foundations and superstructures) should be designed to take into account the additional forces and moments that are induced by differential settlements of the soil under the foundation, as well as by the effects of the construction sequence. Past experience and current industry codes and standards (e.g., American Concrete Institute (ACI) 349-06, “Code Requirements for Nuclear Safety-Related Concrete Structures (ACI 349-06) and Commentary,” issued 2006 [4]) indicate that these are important design considerations. It should be noted that, in recent NPP designs, the most important seismic Category I structures (e.g., the containment and shield buildings, fuel buildings) are grouped together on a nuclear island (NI) that is supported on a single large concrete foundation mat.

To investigate the effect of soil settlements on the structural performance of the NI foundation mat and connecting superstructure walls, it is necessary to distinguish between the settlements that are expected to occur during construction and postconstruction settlements caused by soil consolidation and compaction—the soil stiffness for these two cases can be substantially different. In addition, from the point of view of the DC application, it is necessary to consider (1) how the effects of these settlements are accounted for in the standard design, where assumptions need to be made regarding generic soil parameters, and (2) how the specifications are established such that the COL applicant can demonstrate that, for a particular construction sequence, forces and moments induced by predicted and measured settlements at a particular site are bounded by those considered in the standard design.

On the one hand, the standard design should adequately address the effects of the construction sequence and soil settlements to ensure safety. On the other hand, these issues are often site specific *per se*. Therefore, it can be a significant challenge for a DC applicant to establish an interface that allows for the standard design to account for construction sequence and settlement loads and also permits the COL applicant to verify that these loads are not exceeded during or after construction. To adequately address the interface issue, a standard design could consider (1) a postulated set of soil stiffness parameters for the construction phase, (2) a postulated set of soil stiffness parameters for the postconstruction phase, and (3) a postulated construction sequence and corresponding set of construction loads. To account for construction sequence and settlement loads in the standard design, it may be necessary to perform a detailed, sequential, finite element (FE) analysis of the NI foundation and superstructure with realistic modeling of the supporting soil stiffness and construction sequence, including anticipated effects through the end of the operating life of the NPP. The sequential FE analysis should be based on the postulated soil conditions and validated with geotechnical soil settlement analyses. The envelope of forces and moments computed during the sequential FE analysis can then be compared with corresponding forces and moments obtained from a “reference” FE analysis that does not include construction sequence or settlement effects. From this comparison, any difference in forces and moments could be taken as a separate construction sequence/soil settlement load case, to be considered in the structural design of the foundation and superstructure in addition to all other load cases, in accordance with ACI 349-06, Section 9.2.2. The settlement profiles at all stages of the sequential FE analysis should also be computed; the COL applicant can then use these profiles for verification purposes, as described below.

The soil conditions postulated in the standard design would correspond to a relatively soft soil to bound a majority of potential sites. However, soft clayey soils behave differently from soft sandy soils, especially regarding long-term settlements. The sequential FE analysis should consider the soil conditions, clay or sand, that result in greater induced moments and forces on the foundation and superstructure. In addition, the possibility that a stiffer soil could result in greater induced moments or forces in certain areas of the foundation or superstructure should also be investigated.

In the above discussion, it is assumed that the total magnitude of soil settlements is not a design consideration during either construction or postconstruction phases. It should be noted that it is not so much the magnitude of the settlements that affects the structural performance of the NI foundation and superstructure; rather, it is the relative shape of the settlement profile in terms of slope and curvature. This last statement is only valid when considering an individual structure; total settlement is clearly of interest when considering adjacent structures connected by appurtenances (nonflexible commodities, such as piping and conduit), which would need to be addressed in the design.

Based on the settlement profiles established in the DC, the COL applicant should perform a site-specific geotechnical investigation to determine predicted settlement profiles during construction and postconstruction phases, based on the actual construction sequence to be used. If the predicted settlement profiles compare favorably to the DC settlement profiles—in terms of slope and curvature, not necessarily in absolute magnitude—then it is inferred that the forces and moments induced by the predicted settlements are bounded by the forces and moments considered in the standard design. This comparison may be made in terms of the “angular distortion” concept, as described in U.S. Army Corps of Engineers Manual No. 1110-1-1904, “Engineering and Design: Settlement Analysis,” issued 1990 [5]. In addition to the predictive calculations, a settlement monitoring program needs to be established to verify whether measured settlements are consistent with predicted settlements during the operating life of the NPP.

Seismic Stability

To demonstrate stability for seismic loads in accordance with SRP Section 3.8.5, Revision 3, seismic Category I structures should have margins of safety—expressed as factors of safety (FS)—against sliding and overturning that are greater than 1.1. The implicit assumption is that FS are calculated based on the ratio of minimum seismic resistance divided by maximum seismic demand, where both resistance and demand are unique numbers derived from site-specific design parameters and simple, equivalent-static, calculations.

In DC applications, however, it has been difficult to demonstrate seismic stability using simple calculations. The reason for this difficulty seems to be that, for the reasons mentioned in the introductory section, the seismic demands tend to be overestimated while the seismic resistance tends to be conservatively underestimated. To overcome this difficulty, DC applicants have had to perform more realistic time-history seismic analyses that, in some instances, explicitly incorporated the nonlinearities caused by sliding, uplift, and lateral soil pressures at the perimeter of embedded structures. The results of these analyses often indicate that small amounts of sliding or uplift may occur during a seismic ground motion. To meet the intent of the SRP, the safety review needs to determine, on a case-by-case basis, whether the small amounts of sliding or uplift are acceptable.

Shear keys are often added under the foundations of structures to increase seismic resistance against sliding and to improve overall seismic performance. It is noted that, for typical NPP structures with relatively low aspect ratios, sliding stability is more difficult to demonstrate than overturning stability.

To evaluate seismic stability using a time-history analysis, time-dependent FS against sliding and overturning should be computed and compared against a value of 1.1. In some situations, the computed FS of less than 1.1 may still be acceptable if these FS correspond to durations that are judged too short to cause physical movement of the structure. The time-history analysis should explicitly model the friction at the foundation-soil interface when evaluating the duration, magnitude, and extent of potential sliding; particularly whether sliding corresponds to localized relative displacements at the interface or whether there is a tendency for uniform, rigid-body sliding. The model should explicitly include uplift at the foundation-soil interface to establish whether there is a tendency for rocking. The analysis should also account for lateral soil pressures, including both active and passive pressures; however, it is cautioned that passive pressures can only be activated if soil displacements are greater than a geotechnical threshold that is a function of the soil type. Since it is particularly difficult to model lateral soil pressures in a realistic manner, it may be preferable to use conservative simplifications in the analysis.

Other modeling issues also require careful consideration. For example, if a soil-structure interaction (SSI) analysis is used to determine seismic demands, then sliding, uplift, and other nonlinearities cannot be modeled using standard frequency-domain SSI analysis tools. On the other hand, these nonlinearities are much easier to model using a time-domain analysis; however, SSI effects may be difficult to incorporate in the latter case.

Additional issues include the coefficient of friction assumed in the stability evaluation, which should be sufficiently low that it bounds a majority of potential sites. The selection of the minimum coefficient of friction should consider all potential foundation-soil interfaces (e.g., soil, concrete/mudmat to soil, concrete/mudmat to waterproofing membrane, waterproofing membrane to soil, and concrete to mudmat). It is also important for the DC application to specify the requirements to be met by the COL applicant; in particular, to demonstrate that coefficients of friction at potential slip planes of the foundation-soil interface, at a future potential site, are greater than the coefficient of friction assumed in the stability evaluations.

The preceding discussion briefly reviews some of the challenges involved in the seismic stability evaluations undertaken as part of recent DC applications. It is noted that the most challenging aspects arise from the need to perform a realistic time-history seismic analysis to demonstrate stability. Recognizing that some of the modeling issues involved are rather complex, it may be necessary to use conservative simplifications in the analysis.

Selection of Critical Sections

DC applications typically use so-called critical sections to demonstrate the safety of the structural design. Critical sections are those portions of seismic Category I structures that (1) perform a safety-critical function, (2) are subjected to the largest stress demands, (3) are considered to be representative of the structural design, and (4) provide reasonable assurance that the structural design is being performed in a manner consistent with the guidance in the SRP, regulatory guides (RGs), and other regulatory requirements. The DC applicant needs to carry out the design of these critical sections in full detail, and the DC application should document the corresponding design information.

A significant challenge in the past has been the selection of an appropriate set of critical sections that satisfy, in a consistent manner, the considerations discussed above. Certain DC applicants relied on judgment and past experiences. A recent DC application, however, developed a systematic methodology to select the critical sections [6].

The selection methodology described in reference [6] consists of three-tiered criteria—qualitative, quantitative, and supplemental—applied in a sequential manner. The qualitative criterion is used to select critical sections of the NI structures that perform a safety-critical function. The quantitative criterion is then used, together with a numerical algorithm and the FE analysis of the NI structures, to identify critical sections that are highly stressed but are not chosen under the qualitative criterion. The supplemental criterion is based on engineering judgment and is intended to capture critical sections of seismic Category I structures that are not screened by the other two criteria but are necessary to obtain an adequate representation of all types of structural elements. Preliminary estimates discussed in reference [6] indicate that the critical sections selected using this methodology are representative of 77 percent of NI structures and 84 percent of all seismic Category I structures.

Following the selection methodology described in reference [6], or similar systematic methodologies that combine numerical evaluations with engineering judgment, would greatly simplify the selection process and ensure a consistent set of critical sections, as well as facilitate the safety review.

Combination of Structural Responses Caused by Three Directions of Seismic Load

As part of the structural design process, it is necessary to combine the structural responses resulting from each of the three directions of seismic load and use this combined response in the appropriate design load combinations. In the context of time-history seismic analysis using three statistically independent inputs, this combination is done by algebraic summation of the three time-dependent responses. However, in the context of an equivalent-static or response-spectrum seismic analysis, which computes only maximum responses, algebraic summation is not applicable.

In the past, the most common method for combining maximum responses induced by three directional seismic inputs has been the square-root-of-the-sum-of-the-squares (SRSS) method. Another combination method, commonly known as the 100-40-40 rule, has been referenced in recent DC applications and appears to be gaining in popularity. Both the SRSS method and the 100-40-40 rule are described in American Society of Civil Engineers (ASCE) 4-98, "Seismic Analysis of Safety-Related Nuclear Structures and Commentary," issued 1999 [7], and are acceptable to the NRC if implemented in accordance with RG 1.92, Revision 2, "Combining Modal Responses and Spatial Components in Seismic Response Analysis," issued July 2006 [8]. The application of the SRSS method is straightforward; however, recent reviews of DC applications have raised the question of whether applicants have correctly interpreted the 100-40-40 rule. In a recent paper, Nie et al. [9] clarified the proper implementation of the 100-40-40 rule and compared it to the SRSS method. The authors demonstrated that the 100-40-40 rule, when applied correctly, is almost always conservative compared to the SRSS method and is only slightly unconservative in very rare cases. It is also important to indicate that both the SRSS method and the 100-40-40 rule are based on the assumption of linear elastic structural response.

The preceding discussion corresponds to design situations involving three directions of seismic load and a single structural response quantity. In typical concrete design, however, multiple interacting response quantities are often considered. The combination methods for multiple interacting response quantities are distinctly different from those for a single response quantity [10-16].

To illustrate, consider a concrete wall-type structure. In this case, the interacting response quantities are identified with force/moment resultants, including membrane forces ((Tx, Ty, and Txy), out-of-plane bending moments (Mx*and My*), and out-of-plane shear (Nx and Ny). Out-of-plane bending moments are conservatively defined as Mx* = abs(Mx)+abs(Mxy) and My* = abs(My)+abs(Mxy), where Mx and My are the out-of-plane bending moments taken directly from the FE analysis and Mxy is the twisting moment. Either SRSS or 100-40-40 combination methods can be applied to each of these force/moment resultants to obtain their maximum values due to the three directions of seismic load. These maxima are denoted as Txe, Mxe*, Tye, Mye*, Txye, Nxe, and Nye.

For structural design, it is necessary to determine the critical combinations of interacting force/moment resultants relative to the capacity surface of the wall section, as prescribed in the applicable design code formulas. It is important to note that these critical combinations may not involve any of the maxima, and they cannot be determined without knowledge of the capacity surface. To determine the critical combinations, one possibility is the approach described in ASCE 4-98, Section 3.2.7.1.3(b), based on the work by Gupta and Singh [10]. More recently, Menun and Der Kiureghian [12,13,16] have proposed an approach based on random vibration theory that has several advantages over the latter. However, none of these approaches were used in recent DC applications, possibly because of their relative complexity.

A conservative alternative to determining the critical combinations is to assume that these coincide with all possible positive and negative permutations of the maxima (see ASCE 4-98, Section 3.2.7.1.3(a)). This is a conservative assumption, because it is not likely that all maxima will occur at the same instant in time. Recent DC applications have followed this conservative assumption. Additional research is needed to quantify the degree of conservatism of this assumption relative to the more realistic approach described in references [12,13,16].

In the case of a concrete wall-type structure, the number of interacting force/moment resultants is equal to two, not seven. This is because the design checks for (1) membrane forces plus out-of-plane bending (Txe plus Mxe*, Tye plus Mye*), (2) in-plane shear (Txe plus Tye plus Txye), and (3) out-of-plane shear (Txe plus Nxe, Tye plus Nye) can all be performed independently, following the ASME Code for Concrete Containments [17] or ACI 349-06. Therefore, the total number of positive and negative permutations of maxima is $2^4 = 4$.

The permutations for membrane forces plus out-of-plane bending are:

$$+ Txe + Mxe^*, + Tye + Mye^* \quad (1a)$$
$$– Txe – Mxe^*, – Tye – Mye^* \quad (1b)$$
$$+ Txe – Mxe^*, –Tye + Mye^* \quad (1c)$$
$$– Txe + Mxe^*, + Tye – Mye^*. \quad (1d)$$

The permutations for in-plane shear are:

$$+ Txe + Tye + Txye \quad (2a)$$
$$– Txe – Tye + Txye \quad (2b)$$
$$+ Txe – Tye + Txye \quad (2c)$$
$$– Txe + Tye + Txye. \quad (2d)$$

Note that permutations with positive and negative Txye are not needed because in-plane shear capacity is independent of sign. The permutations for out-of-plane shear are:

$$+ Txe + Nxe, + Tye + Nye \quad (3a)$$
$$– Txe + Nxe, – Tye + Nye. \quad (3b)$$

Again, permutations with positive and negative Nxe and Nye are not needed because out-of-plane shear capacity is independent of sign. Finally, four basic permutations of maxima are determined by grouping the preceding partial permutations as follows:

$$+ Txe, + Mxe^*, + Tye, + Mye^*, + Txye, + Nxe, \text{ and } + Nye \quad (4a)$$
$$– Txe, – Mxe^*, – Tye, – Mye^*, + Txye, + Nxe, \text{ and } + Nye \quad (4b)$$
$$+ Txe, – Mxe^*, – Tye, + Mye^*, + Txye, + Nxe, \text{ and } + Nye \quad (4c)$$
$$– Txe, + Mxe^*, + Tye, – Mye^*, + Txye, + Nxe, \text{ and } + Nye. \quad (4d)$$

These basic permutations can then be used in the design checks for code compliance. This is the approach followed in a recent DC application.

SUMMARY

This paper provided a brief overview of technical challenges identified in the review of DC and COL applications that are related to the structural analysis and design for new reactors. It provided detailed discussions of these challenges with respect to (1) structural design for the effect of differential settlements and construction sequence, (2) seismic stability evaluations, (3) methodology for selecting critical sections, and (4) combination of multiple interacting structural response quantities when three directions of seismic load are present. It is hoped that the insights and discussions provided in this paper will contribute to a better understanding of these issues, thereby improving the analysis and design performed in support of DC and COL applications, and will also facilitate NRC safety reviews.

DISCLAIMER NOTICE

The findings and opinions expressed in this paper are those of the authors and do not necessarily reflect the views of Brookhaven National Laboratory or the NRC. The paper may present information that does not currently represent an agreed-upon NRC position.

REFERENCES

[1] 10 CFR Part 50, "Domestic Licensing of Production and Utilization Facilities," U.S. Nuclear Regulatory Commission, Washington, DC.

[2] NUREG-0800, "Standard Review Plan for Review of Safety Analysis Reports for Nuclear Power Plants: LWR Edition," U.S. Nuclear Regulatory Commission, Washington, DC.

[3] 10 CFR Part 52, "Licenses, Certifications, and Approvals for Nuclear Power Plants," U.S. Nuclear Regulatory Commission, Washington, DC.

[4] ACI 349-06, 2006, "Code Requirements for Nuclear Safety-Related Concrete Structures (ACI 349-06) and Commentary," American Concrete Institute, Farmington Hills, MI.

[5] Engineer Manual No. 1110-1-1904, 1990, "Engineering and Design: Settlement Analysis," U.S. Army Corps of Engineers, Washington, DC.

[6] Jung, S.-K., Harrold, J., Goodman, A., and Alchaar N., 2010, "Critical Section Selection Methodology for the U.S. EPR Standard Nuclear Power Plant," Paper PVP2010-25348, Proceedings of the ASME 2010 Pressure Vessels & Piping Division Conference (PVP2010), July 18–22, 2010, Bellevue, WA.

[7] ASCE 4-98, 1999, "Seismic Analysis of Safety-Related Nuclear Structures and Commentary," American Society of Civil Engineers, Reston, VA.

[8] Regulatory Guide 1.92, 2006, "Combining Modal Responses and Spatial Components in Seismic Response Analysis," Revision 2, U.S. Nuclear Regulatory Commission, Washington, DC.

[9] Nie, J., Morante, R., Miranda., M., and Braverman, J., 2010, "On the Correct Application of the 100-40-40 Rule for Combining Responses due to Three Directions of Earthquake Loading," Paper PVP2010-25466, Proceedings of the ASME 2010 Pressure Vessels & Piping Division Conference (PVP2010), July 18–22, 2010, Bellevue, WA.

[10] Gupta, A.K. and Singh, M. P., 1977, "Design of column sections subjected to three components of earthquake," Nuclear Engineering and Design, 41, pp. 129–133.

[11] Menun, C. and Der Kiureghian, A., 1998, "A replacement for the 30%, 40%, and SRSS rules for multi-component seismic analysis," Earthquake Spectra, 14(1), pp. 153–156.

[12] Menun, C. and Der Kiureghian, A., 2000a, "Envelopes for seismic response vectors. I: theory," Journal of Structural Engineering, 126(4), pp. 467–473.

[13] Menun, C. and Der Kiureghian, A., 2000b, "Envelopes for seismic response vectors. II: application," Journal of Structural Engineering, 126(4), pp. 474–481.

[14] López, O.A., Chopra, A.K., and Hernández, J.J., 2001, "Evaluation of combination rules for maximum response calculation in multi-component seismic analysis," Earthquake Engineering and Structural Dynamics, 30, pp. 1379–1398.

[15] Hernández, J.J. and López, O.A., 2003, "Evaluation of combination rules for peak response calculation in three-component seismic analysis," Earthquake Engineering and Structural Dynamics, 32, pp. 1585–1602.

[16] Menun, C., 2004, "Strategies for identifying critical seismic response combinations," Earthquake Spectra, 20(4), pp. 1139–1165.

[17] ASME Code, 2004, "ASME Boiler and Pressure Vessel Code, Section 3, Division II, Code for Concrete Containments," American Society of Mechanical Engineers, New York.

DISTRIBUTION:
TBergman
MShuaibi
RidsNroDeResource

ADAMS Accession Package No.: ML111360047

OFFICE	NRO/DE/SEB2	NRO/DE/SEB2	NRO/DE
NAME	JXu	KHawkins	TBergman: MShuaibi for
DATE	4/20/2011	05/09/2011	05/05/2011

OFFICIAL RECORD COPY