May 29, 2003

Mr. W. E. Cummins, Director
AP600 & AP1000 Projects
Westinghouse Electric Company
P.O. Box 355
Pittsburgh, PA 15230-0355

Dear Mr. Cummins:

As you are aware, the U. S. Nuclear Regulatory Commission (NRC) staff is preparing the draft safety evaluation report (DSER) for the AP1000 design certification application submitted by Westinghouse Electric Company (Westinghouse) on March 28, 2002. The staff expects to issue the DSER in June, 2003. As of this date, the staff has identified 28 potential open items for DSER Chapter 3, "Design of Structures, Components, Equipment, and Systems" which are enclosed for your information. Please note that the staff's review of the application will continue during preparation of the DSER, which may result in changes to the potential open items identified in the enclosure, or the addition of other open items.

Five of the potential open items in the enclosure are new issues. The other 23 potential open items in the enclosure have the original request for additional information (RAI) number included for reference. If the staff cannot resolve the potential open items before the issuance of the DSER, these items will be issued as DSER open items and will be tracked with a corresponding open item number.

Previously, Westinghouse committed to provide responses to all identified open items within 9 weeks after the issuance of the DSER. The staff will be prepared to review your responses to the open items and have conference calls and meetings with your staff, as appropriate, after the DSER is issued. If Westinghouse chooses to address some or all of these open items before the issuance of the DSER, the staff may not have sufficient time to evaluate every response to the potential open items that Westinghouse submits to the NRC and make changes to the DSER before the scheduled DSER issuance in June, 2003.

Please contact one of the following members of the AP1000 project management team if you have any questions or comments concerning this matter: Mr. John Segala (Lead Project Manager) at (301) 415-1858 or jps1@nrc.gov, Mr. Joseph Colaccino at (301) 415-2752 or jxc1@nrc.gov, or Ms. Joelle Starefos at (301) 415-8488 or jls1@nrc.gov.

Sincerely,

/RA/

James E. Lyons, Director
New Reactor Licensing Project Office
Office of Nuclear Reactor Regulation

Docket No. 52-006

Enclosure: As stated

cc: See next page

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NAME	JColaccino*	MGamberoni* w/ cmts.	JLyons
DATE	05/28 /03	05/28/03	05/29/03

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Westinghouse AP1000
Draft Safety Evaluation Report
Potential Open Items
Chapter 3
Design of Structures, Components, Equipment, and Systems

Open Item Number: 3.3.1-1

Original RAI(s): n/a

Summary of Issue: Pressure generated from the design wind velocity is further dependent on exposure and gust response factors corresponding to the exposure categories. The applicant has used exposure Category C which is consistent with open shoreline and flat open country exposure. Category C exposure is suitable for most sites in the Eastern United States; however, it is not suitable for sites near open inland waterways, the Great Lakes and coastal areas of California, Oregon, Washington and Alaska. The wind load design for AP1000 makes it unsuitable for sites that fall under the exposure Category D. Seismic Category I structures for AP1000 are robust and their lateral load resistance is generally governed by seismic and tornado loading. It may be feasible to demonstrate that the AP1000 wind design is adequate for exposure Category D. Without such a demonstration, the use of wind exposure category is an open issue. This issue is Open Item 3.3.1-1.

Open Item Number: 3.3.1-2

Original RAI(s): n/a

Summary of Issue: In order to calculate the pressure loadings on structures for the design tornado wind velocity and the associated vertical distribution of wind pressures and gust factors, the applicant has used American Society of Civil Engineers (ASCE) 7-98. But the shape coefficients for the shield building are calculated using ASCE Paper No.3269, “Wind Forces on Structures,” Vol. 126, Part II (1961). The ASCE Paper 3269 is a reference in the Standard Review Plan in Section 3.3.1. It is not clear why the applicant used the latest ASCE standard for the basic wind velocity, importance category and exposure category, but did not use the recommendations of ASCE 7-98 for the velocity pressure and the corresponding pressure and force coefficients. AP1000 structures are dynamically rigid and the use of pressure coefficients different from those recommended by ASCE 7-98 is not likely to produce an unacceptable design, since the lateral strength of the AP1000 structures is likely to be governed by seismic and tornado loads. Nevertheless, the applicant should clarify its inconsistent use of the ASCE 7-98 recommendations for wind load design. This issue is Open Item 3.3.1-2.

Enclosure

Open Item Number: 3.3.2-1

Original RAI(s): n/a

Summary of Issue: The procedures used to calculate pressure loads from the tornado wind velocity are the same as those used for wind, as discussed in Section 3.3.1 of this report. The tornado missile effects are determined using procedures discussed in DCD Tier 2 Section 3.5, and the acceptability of these procedures is given in Section 3.5 of this report. Tornado loading includes tornado wind pressure, internal pressure by tornado-created atmospheric pressure drop, and forces generated by the impact of tornado missiles. These loads are combined with other loads as described in DCD Tier 2 Section 3.8.4. The acceptability of these loads and load combinations is discussed in Section 3.8.4 of this report. The applicant has indicated that a maximum pressure drop of 13.8 kPa (2 psi) is used for non-vented structures, unless a lower value is justified by a detailed analysis using the provisions of ASCE 7-98 for partially vented structures. However, the applicant has not identified any structure within the scope of its standard design, AP1000, for which a lower pressure drop has been used. Design certification is a final decision by the NRC subject to provisions of changes through rule making; consequently, the applicant needs to identify all the structures for which it has used lower than 13.8 kPa (2 psi) pressure drop in the design. Therefore, the use of tornado pressure drop of less than 13.8 kPa (2 psi) for vented structures in the future is an open issue. This issue is Open Item 3.3.2-1.

Open Item Number: 3.5.1.3-1

Original RAI(s): 251.001, 251.002

Summary of Issue: The methodology and analytical results of the probability of turbine missile generation are contained in the applicant's submittals, including WCAP-15783, and WCAP-15785. The NRC staff requested information in RAI 251.001 about the modifications made to the current turbine missile methodology from methodologies previously approved by the staff. In the response to RAI 251.001, the applicant did not directly provide the information requested regarding changes from previously approved methodologies. However, the staff performed a detailed review to identify and evaluate the modifications made to the previously approved turbine missile methodologies; therefore, RAI 251.001 is considered to be closed.

WCAP-15783 evaluated four potential failure mechanisms: (1) ductile burst from destructive overspeed; (2) fracture from HCF cracking; (3) fracture from LCF cracking; and (4) fracture from SCC. WCAP-15783 concludes that ductile burst will not occur before destructive overspeed is reached (the probability of reaching destructive overspeed is discussed in WCAP-15785). Also, the applicant concluded that the effect due to HCF cracking and LCF cracking can be ignored because of their extremely low

probabilities of generating turbine missiles. Notwithstanding, the staff reviewed this information and determined that the applicant's evaluation methodology and results for the ductile failure from overspeed and HCF cracking are consistent with approved methodologies, and is therefore acceptable. The evaluation methodology for fracture from LCF cracking is similar to that previously reviewed in approved methodologies. The NRC staff evaluated the two parameters in the Paris fatigue crack growth rate equation and found them acceptable because they were derived from test and actual plant data. Although the data set is limited, this is acceptable to the staff because a very conservative fracture toughness is used for the disk material in the evaluation. The NRC staff has also examined the failure equation and determined that it is based on fracture mechanics using the Paris fatigue crack growth rate that is acceptable to the staff. Further, except for the maximum undetectable crack size, all remaining deterministic parameters such as flaw shape factor, critical crack depth, and cyclic stress range are conservative and acceptable to the NRC staff. Consequently, the NRC staff requested in RAI 251.002 that the applicant justify the use of the specified value for the maximum undetectable crack size. In the response to RAI 251.002, the applicant provided a value for the maximum undetected flaw size but did not provide a basis for the maximum undetected flaw size. This is Open Item 3.5.1.3-1.

Open Item Number: 3.5.1.3-2

Original RAI(s): 251.002

Summary of Issue: Regarding failure by SCC, the applicant responded to a staff question in RAI 251.002 on the interaction of the LCF and SCC failure mechanisms. The response indicates that crack initiation by SCC and LCF begins at different locations and the interaction between them is not considered in the analysis. This is an acceptable explanation for not considering their interaction in the analysis. However, the applicant's response to RAI 251.002 regarding the SCC growth rate is not satisfactory. The SCC growth rate reported in WCAP-15783 is based on statistical analysis of 12 data points. Since the data set is considerably smaller than the data relied upon in past analyses, the applicant needs to include other data of the same material available in industry in its current analysis. Further, the staff is concerned that specified values or the units for the coefficients in the SCC growth equation may not be correct. Using SCC growth rate of 4.5×10^{-4} mm/h (1.77×10^{-5} in./h) from Table 4-5 of WCAP-15783 and the specified values for the coefficients (a, b, and c) for SCC growth calculation would give a negative value for the yield strength. This is Open Item 3.5.1.3-2.

Open Item Number: 3.6.3.4-1

Original RAI(s): RAI 251.004

Summary of Issue: In RAI 251.004, the staff requested that the applicant address the following: (1) clarify whether Alloy 600 material, which is susceptible to PWSCC as indicated by the V.C. Summer primary loop leakage, will be used in any of the AP1000 LBB candidate piping systems, (2) provide test and plant operational data demonstrating that the proposed weld material, Alloy 52/152, is not susceptible to PWSCC, and (3) provide an inspection plan licensees would be required to perform that addresses additional inspection techniques for detecting tight flaws that might exist in LBB piping welds.

The applicant's response to RAI 251.004 states the following: (1) Alloy 600 will not be used for any of the AP1000 LBB candidate piping systems; (2) Alloy 52/152 weld material (for Alloy 690 base material) has been used in various applications such as steam generator welds and safe end-nozzle welds for 9 plants (7 years in one application) without any reported instances of environmental degradation, and although laboratory data for Alloy 52/152 in simulated primary water is limited, they indicated no environmentally-related crack propagation for periods up to 4122 hours; and (3) since Alloy 52/152 weld material has better crack resistance than for Alloy 82/182, augmented inservice inspection using eddy current testing (ET) to supplement ultrasonic testing (UT) should not be required for the AP1000 applications.

The staff considers the information provided for (1) to be complete and that no further information is required. Regarding (2), although the chrome content of Alloy 52/152 is approximately twice the chrome content of Alloy 82/182, the test and plant operational data for Alloy 52/152 are for periods less than 7 years, which is not long enough for the NRC staff to consider the question of PWSCC in the AP1000 LBB candidate piping to be resolved. The staff concurs with the applicant that using ET to supplement UT is not required for AP1000.

To address this issue for currently operating plants, the industry has undertaken an initiative to (1) develop overall inspection and evaluation guidance, (2) assess the current inspection technology, and (3) assess the current repair and mitigation technology. An interim industry report, "PWR Materials Reliability Project Interim Alloy 600 Safety Assessment for U.S. PWR Plants (MRP-44), Part 1: Alloy 82/182 Pipe Butt Welds," was published in April 2001 to justify the continued operation of PWRs while the industry completes the development of the final report. The final industry report on this issue has not yet been published. Subsequent to staff review and evaluation of the final report and receipt of additional UT inspection data from piping involving Alloy 82/182 weld material from the industry, the staff will determine if additional regulatory actions will need to be imposed to address the potential for PWSCC to

occur in lines with current approved LBB analyses in operating plants. To address this issue for AP1000 application, the applicant needs to modify its DCD Tier 2 Section 3.6.4 on COL information to indicate that COL holders will be required to implement inspection plans, evaluation criteria, and other types of measures imposed on or adopted by operating PWRs with currently approved LBB applications as part of the resolution of concerns regarding the potential for PWSCC in those units. This is Open Item 3.6.3.4-1.

Open Item Number: 3.6.3.4-2

Original RAI(s): 251.005

Summary of Issue: In RAI 251.005, the staff requested the applicant provide crack morphology parameters, e.g., surface roughness, number of 45 degree and 90 degree turns, etc., that were used in generating the BACs for LBB. The NRC staff also asked for a comparative study, using the crack morphology parameters associated with transgranular stress corrosion cracking (TGSCC). In its response to RAI 251.005, the applicant provided the crack morphology parameters used in generating the BACs. However, since chlorides will be controlled at minimum levels in the AP1000 LBB candidate piping systems water environment and the hydrogen overpressure will keep the oxygen levels to near zero, the applicant discounted the possibility of the TGSCC and considered the comparative study using the crack morphology parameters associated with TGSCC not necessary. The NRC staff considers that the applicant's argument does not address the thrust of RAI 251.005 and embarked on an independent sensitivity study to assess the impact on the BACs due to a consideration of TGSCC type of crack in the LBB analysis. The NRC staff's independent sensitivity study shows that the BACs might not be easily met by the most limiting piping. DCD Tier 2 Appendix 3B.3.3.4 does not rule out the possibility of a LBB candidate piping not meeting the BAC limit either, as evidenced by the statement: "If the point falls above the bounding analysis curve, the leak-before-break analysis criteria are not satisfied and the pipe layout or support configuration needs to be revised to meet the leak-before-break bounding analysis."

The applicant's LBB evaluation approach using BACs for various piping systems is similar to that of the AP600 application. To provide additional assurance that all AP1000 LBB candidate piping systems are likely to meet their BACs at the COL phase, the applicant needs to provide the ratios between the critical flaw size and the leakage flaw size for the five AP600 LBB systems, for which it had performed traditional LBB analyses. Further, the applicant needs to perform a traditional LBB analysis for the most limiting AP1000 candidate piping system, provide the basis for selecting the limiting piping system, and report the calculated ratio between the critical flaw size and the leakage flaw size. This is Open Item 3.6.3.4-2.

Open Item Number: 3.7.1.5-1

Original RAI(s): 241.001

Summary of Issue: In Section 2.0, “Site Characteristics,” and DCD Tier 2 Table 2-1, the applicant specified that the COL applicant will use the following design site-parameters to confirm the adequacy of the AP1000 seismic design for a specific site:

- The site-specific ground motion response spectra, defined at the foundation level, are bounded by the proposed design response spectra (the modified RG 1.60 ground response spectra) anchored to 0.3g as shown in DCD Tier 2 Figures 3.7.1-1 and 3.7.1-2.

- No potential for fault displacement is expected at the site.

- No liquefaction is expected at the site.

- The average allowable static bearing capacity is greater or equal to 402 kPa (8,400 psf) over the foot print of the NI at its excavation depth. The allowable bearing capacity under static plus dynamic loads exceeds 4,070 kPa (85,000 psf).

- the minimum shear wave velocity of the rock foundation is equal to or greater than 8,000 ft/sec.

Based on its review experience of other advanced reactors such as ABWR, System 80^{+} and AP600, the staff concludes that the above design site-parameters are reasonable and acceptable bounding limits for the COL applicant to use in confirming the adequacy of the AP1000 seismic design, except for the definition for the average allowable static bearing capacity for the hard rock site. The staff requested the applicant to clarify whether this term refers to allowable strength or allowable displacement of the foundation. In its response to RAI 241.001, the applicant stated that the design will be acceptable for a hard rock site that has an allowable bearing capacity of 450 kips per square foot. The staff’s review experience indicates that this is an extremely high value of “allow bearing capacity,” that is difficult for the COL applicant to substantiate. Also, the response still did not clarify whether this definition refers to strength or displacement considerations. In addition, the review of the Civil/Structural Criteria document performed by the staff during the November 12 through 15, 2002, audit indicated that hard crystalline bedrock should have an allowable bearing capacity of four (4) kips per square foot. The definition of allowable bearing capacity for the hard rock site must also account for the influence of bedding direction, level of cracking and other discontinuities in the rock material which can serve to limit bearing capacity. These discrepancies need to be clarified by the applicant. The staff identified this as Open Item 3.7.1.5-1.

Open Item Number: 3.7.2.1-1

Original RAI(s): 230.002

Summary of Issue: In DCD Tier 2 Sections 2.5 and 3.7.1, the applicant proposed to found the NI structures on a hard rock site with an embedment of 39’-6". The staff’s review identified a question regarding how lateral soil pressures due to embedment were calculated for use in the design of exterior walls of the NI. In its response to RAI 230.002 dated October 4, 2002, and January 21, 2003, the applicant stated that the exterior walls of the NI were designed for two lateral soil pressure cases: lateral earth pressure equal to the sum of static earth pressure plus the dynamic earth pressure, and lateral earth pressure equal to the passive earth pressure. The applicant also agreed to perform additional calculations of total earth pressures for the various load cases to ensure that the load case will lead to the maximum wall moments and shears. This is Open Item 3.7.2.1-1.

Open Item Number: 3.7.2.3-1

Original RAI(s): 230.020, 230.021, 230.022

Summary of Issue: During the audits conducted on November 12, 2002, and April 2, 2003, the staff discussed with the applicant the development of the dynamic model of the NI structures and reviewed the applicant’s analysis reports based on both 3D lumped-mass stick model and 3D finite element model. The seismic analysis results from the 3D finite element model of the coupled auxiliary/shield building shows net tension in the shield building wall. This phenomenon suggests that during the postulated seismic event, parts of the basemat will lift up from the rock surface resulting in changes in the basemat stresses and reduction of shear wall stiffnesses due to reinforced wall cracking. As a result of the detailed review of the seismic modeling approach and analysis methods, the staff identified an issue that the assumptions of uncracked reinforced concrete walls and fixed-base foundation may become invalid. With this finding, the applicant was requested to provide justification to show that the current seismic analysis results used for the design of the NI structures, systems and components are reasonable and acceptable.

In resolving this issue, the staff, during the meeting conducted on April 2, 2003, explained its concern and expectation to the applicant regarding the significance of uplift due to seismic excitation of the NI and the effect of reduction of stiffness of shear walls. The discussion reached the following conclusions:

- The applicant will use East-West lumped-mass stick model of the NI structures supported on a rigid plate with nonlinear springs that transmit reactions in horizontal and vertical directions to simulate the foundation contact area, and perform a seismic time history

analysis (the nonlinear springs will be in action only when the rigid plate is in contact with the subgrade). The results of this seismic time history analysis will be compared against the peak accelerations and the floor response spectra at the lumped mass node points obtained from the current three dimensional model analysis without the uplift consideration. If the comparison shows differences, the applicant should evaluate the significance of these differences and their effects on the current seismic design.

- With regard to the effect of shear wall stiffness reduction (due to shear wall cracking) on the seismic analysis results (natural frequencies, peak floor accelerations and the floor response spectra), the applicant will consider using a three dimensional (3D) lumped mass stick model with reduced member stiffnesses to conduct a time history seismic analysis. Results from this analysis will be compared against those currently used by the applicant for the design of the NI structures, systems and components. If the comparison shows differences, the applicant should evaluate the significance of these differences and their effects on the current seismic design.

When the final seismic analyses are performed for the NI structures, the applicant should incorporate the two above discussed effects in the final seismic model for calculating seismic responses. These seismic responses should also be compared against those currently used for the seismic design. If the comparison shows differences on the order of 10 percent or less, the combined effect of uplifting and shear wall cracking will be considered as insignificant. Otherwise, the seismic loads used for the design will have to be revised accordingly.

Depending on the outcome of the comparisons from the two separate analyses discussed above, one for the uplift effect and the other for stiffness reduction, the design calculations for the certified design may have to be revised. This is Open Item 3.7.2.3-1.

Open Item Number: 3.7.2.3-2

Original RAI(s): 230.018

Summary of Issue: In its response to RAI 230.18, the applicant provided the following justifications for these results:

(a) For the shield building roof structures, the maximum vertical absolute acceleration is 0.9 g for the AP600 and 0.89 g for the AP1000 (in the initial analyses). In the most recent AP1000 analyses, the dominating frequency is 5.81 Hz and the maximum absolute acceleration is 0.96 g in the vertical direction. These differences in seismic response are partly due to changes in modal properties but are also affected by the ground

motion time history which envelops the design ground response spectrum.

(b) For the steel containment vessel, the maximum vertical absolute acceleration is 1.49 g for the AP600 and 1.40 g for the AP1000 (in the initial analyses). In the most recent AP1000 analyses, the dominating frequency is 16.97 Hz and the maximum absolute acceleration is 1.13 g in the vertical direction. The reduction in the vertical response is associated with better definition of the AP1000 polar crane and the use of a multi-mass model of the polar crane instead of the single mass model used in the AP600 analyses and the initial AP1000 analyses. The first frequency (representing the polar crane mode) of the combined model in the vertical direction is 6.415 Hz compared to that of 5.843 Hz in the previous analyses.

The applicant's justification provided in (a) above for the results of the shield building roof structures appears reasonable and sufficient. As for the steel containment vessel (see (b) above), the staff's review of the DCD and the seismic analysis report of the steel containment vessel (Calculation APP-1000-S2C-037) during the November 12 through 15, 2002, audit, revealed that the first frequency (polar mode) of the combined model (combined vessel lumped-mass stick model with multi-mass polar crane model) is 6.415 Hz which is in the same range of that from the initial analyses. The frequencies (16.97 Hz and 28.201 Hz) and modal masses corresponding to the two dominating vertical modes of the revised steel containment vessel model (combined vessel model with multi-mass crane model) also remain essentially unchanged in comparison with those of the initial analyses. Because the frequency corresponding to the crane mode is more than 10 hertz apart from the dominating frequencies of the vessel, the staff does not expect the vertical absolute acceleration at the top of the AP1000 steel containment vessel would be significantly reduced due to the use of the multi-mass model of the polar crane. Based on the above discussion, the applicant needs to justify why the vertical acceleration at the containment vessel dome is reduced from 1.40g to 1.13g as a result of using different polar crane model. This is Open Item 3.7.2.3-2.

Open Item Number: 3.7.2.3-3

Original RAI(s): 230.020

Summary of Issue: The seismic model for the NI structures was developed on the basis of uncracked concrete section properties of shear walls. During the teleconference call on January 23, 2003, the staff questioned the applicant's assumption that the calculation of shear wall stiffness does not consider a reduction in stiffness due to cracking. The stiffness reduction would affect the seismic loads for the design of critical sections of the NI structures and the frequency locations of the floor response spectrum peaks. The applicant agreed to review the references provided

by the staff on the stiffness reduction in shear walls, and provide justification or correction as needed. This is Open Item 3.7.2.3-3.

Open Item Number: 3.7.2.9-1

Original RAI(s): 230.020

Summary of Issue: As described in DCD Tier 2 Section 3.7.2.9, the effects of parameter uncertainty had not been explicitly considered. To account for such effects, the applicant, following the guidelines of SRP Section 3.7.2 and RG 1.122, broadened the peaks of the floor spectra by ±15 percent based on the corresponding spectral peak frequency. The staff found this acceptable, except that Open Item 3.7.2.3-3 (see Subsection 3.7.2.3 of this report) concerning the issue of stiffness reduction due to shear wall concrete cracking remains to be resolved. This issue is especially significant when one considers the additional uncertainties associated with structural modeling. This is Open Item 3.7.2.9-1. This open item will be closed, pending the resolution of Open Item 3.7.2.3-3.

Open Item Number: 3.7.2.16-1

Original RAI(s): n/a

Summary of Issue: The seismic design basis earthquake for the AP1000 structures, systems, and components are essentially defined at the plant grade level in the free field by an SSE with the peak acceleration of 0.3g and the ground response spectra shown in DCD Tier 2 Figures 3.7.1-1 and 3.7.1-2. The seismic design of the NI features (structures including basemat, systems, and components) is predicated on the limitation of constructing the AP1000 at hard rock sites with shear wave velocity equal to 2438 m/sec (8,000 fps) or higher. If these design bases are not satisfied (i.e., the site condition is not within the range of site conditions specified in the DCD) or if the seismic analysis responses used for the design do not envelop the results obtained from a potential plant's site conditions other than the hard rock sites, the basis established for the design certification will no longer apply. The applicant should commit in the DCD (similar to the AP600 DCD) that the COL applicants should perform an analysis and an evaluation using the design basis earthquake ground motion and plant-specific site conditions to confirm the design adequacy of the AP1000 design. This is COL Action Item 3.7.2.16-1 and Open Item 3.7.2.16-1.

Open Item Number: 3.8.2.1-1

Original RAI(s): n/a (April 3, 2003, meeting summary)

Summary of Issue: The containment vessel is an ASME metal containment. The information contained in this subsection is based on the design specification and preliminary design and analyses of the vessel. During the April 2-5, 2003

audit at Westinghouse, the applicant informed the staff that the final detailed analyses, to be documented in the ASME Design Report, are not available and will be the responsibility of the COL applicant. The staff expected that the final detailed analyses for the AP1000 steel containment would be submitted for staff review as part of the design certification process for AP1000. To complete the staff evaluation of the AP1000 steel containment design, the staff will need to audit the final detailed analyses. This is Open Item 3.8.2.1-1.

Open Item Number: 3.8.2.2-2

Original RAI(s): 220.004

Summary of Issue: Stability of the AP1000 containment vessel and appurtenances is evaluated using ASME Code, Case N-284-1, Metal Containment Shell Buckling Design Methods, Class MC, Section III, Division 1, as published in the 2001 Code Cases, 2001 Edition, July 1, 2001. Since the latest version of Code Case N-284-1 has not been endorsed by the staff, the applicant is requested to provide its technical justification for the acceptability of this code case by demonstrating an equivalent level of safety when compared to Code Case N-284, Revision 0 plus the supplemental requirements of AP600 DCD Appendix 3G. In its response to RAI 220.004 (Revision 0), the applicant confirmed that the AP1000 criteria are the same as the AP600 criteria previously accepted by the staff. The staff finds this acceptable, except that Code Case N-284-1 has not been designated as Tier 2* material. The staff notes that in the AP600 DCD, Appendix 3G is designated Tier 2*. This is Open Item 3.8.2.2-2.

Open Item Number: 3.8.3.5-1

Original RAI(s): 220.010

Summary of Issue: In its response to RAI 220.010 (Revision 1), the applicant indicated that (1) responses to RAIs 230.006 and 230.007 related to DCD Tier 2 Section 3.7 provide the information to address the concern relating to seismic analysis methods and the techniques for combining spatial effects of three earthquake components used for the internal structures; (2) DCD Tier 2 Tables 3.7.2-1 to 3.7.2-7 provide numerical values for frequency and accelerations; and (3) adequate safety is maintained because code criteria stress limits are used. The applicant has revised DCD Tier 2 Section 3.8.3.5 to replace “response spectrum” with “equivalent static”; and revised DCD Tier 2 Table 3.8.3-2 to clarify the models and methodology utilized for the various analyses of the structural modules. During the April 2 through 5, 2003, design audit, the staff reviewed Westinghouse Calculation Nos. APP-1000-S2C-034, Revision 1 (finite element model of the containment internal structures), APP-1100-S2C-002, Revision 1 (seismic equivalent static analysis for containment internal structures), and APP-1200-S2C-001, Revision 0 (finite element -

seismic equivalent static analysis of the auxiliary shield building). These analyses show how the equivalent static analysis method was implemented for the containment internal structures and structures outside the containment. The calculations demonstrated that the maximum equivalent static accelerations obtained from the stick model time history analysis were used as input to the finite element models of the plant structures. To combine the structural responses due to the three components of earthquake motion, the calculations used either the SRSS method or 1.0, 0.4, and 0.4 method. Accidental torsion was also included in the two horizontal directions. Modal frequencies for the structures were determined and presented in the calculations. Seismic amplification for out-of-plane flexibility of walls and floors was accounted for in most cases by either including the flexibility in the seismic time history model or developing an amplification factor (such as the containment internal structure wall modules). One item that arose during the April 2 through 5, 2003, design audit, is that there is no technical guidance document that demonstrates how the flexibility of walls and floors other than critical sections will be considered in the seismic analyses. Based on the above discussion, the concern regarding the lack of a documented method for considering out-of-plane wall and floor flexibility remains unresolved. This is Open Item 3.8.3.5-1.

Open Item Number: 3.8.3.5-2

Original RAI(s): n/a (April 3, 2003, meeting summary)

Summary of Issue: As described in DCD Tier 2 Section 3.8.3.5.7, a design summary report is prepared for containment internal structures documenting that the structures meet the acceptance criteria specified in DCD Tier 2 Section 3.8.3.5. During the April 2 through 5, 2003, audit, the applicant provided the preliminary Containment Internal Structures Summary Report for review. Since the Design Summary Report has not been completed, the staff could not perform its review of the report in accordance with SRP Section 3.8.3. As indicated in SRP Sections 3.8.3.I.4 and 3.8.3.II.4, the Design Report is reviewed and considered acceptable if it satisfies the guidelines of Appendix C to SRP Section 3.8.4. based on the above discussion, review of the Design Summary Report is Open Item 3.8.3.5-2.

Open Item Number: 3.8.3.5-1

Original RAI(s): n/a (April 3, 2003, meeting summary)

Summary of Issue: Westinghouse calculation No. APP-1100-S2C-007, Revision 0 contains the design of the IRWST concrete-filled steel module walls. The staff reviewed the approach used to calculate the required steel area of the structural walls. The calculation determined the required steel reinforcement area at various locations in each of the critical walls. This was done using the methodology contained in Westinghouse guidance document APP-GW-S1-008, Revision 0 (Design Guide for Reinforcement

in Walls and Floor Slabs). During the audit, the applicant indicated that boundary elements are not required for walls that frame into other walls since the other walls act as boundary elements. The staff found that the applicant's approach for the analysis and design does not meet the requirements of Chapter 21.6 of ACI-349-01. A similar issue is presented in Subsection 3.8.4.2 of this report under Open Item 3.8.4.2-1. This is Open Item 3.8.3.5-1.

Open Item Number: 3.8.4.2-1

Original RAI(s): n/a (April 3, 2003, meeting summary)

Summary of Issue: During the April 2 through 5, 2003, design audit, the staff reviewed the applicant's approach to the design of boundary elements potentially needed to reinforce boundaries and edges around openings of structural walls. In accordance with Chapter 21.6 of ACI 349-01, if the vertical compressive stress at the opening does not exceed 0.2 f'_c, then a boundary element is not required. The applicant contended that this compressive stress limit is not applicable when seismic member forces are based on elastic analysis, no ductility reduction factor is applied, and a stress limit of 1.0 f'_c is used as the stress threshold for boundary elements. The staff disagreed with the approach proposed by the applicant, and also pointed out that the stress prediction at an opening is highly dependent on the finite element mesh refinement. In addition, the staff review of Westinghouse calculation APP-1200-CCC-102 "Auxiliary Building Wall 7.3 Reinforcement Calculation" indicated that boundary element evaluations were not considered at the intersection of reinforced concrete walls. The staff position is that the need for boundary elements around openings and at intersections of reinforced concrete walls should be evaluated in accordance with Chapter 21.6 of ACI-349-01. The applicant agreed to consider the staff's positions and to develop criteria to implement the provisions of Chapter 21.6 of ACI-349-01. The applicant's action will be reviewed by the staff when submitted. This is Open Item 3.8.4.2-1.

Open Item Number: 3.8.4.3-1

Original RAI(s): 220.015

Summary of Issue: The applicant referenced DCD Tier 2 Figure 3D.5-9 in its response to RAI 220.015; however, it does not appear to have considered the effect of rapid increase in compartment temperature. Based on its review of the selected calculations, the staff could not reach a conclusion that the applicant has adequately addressed the effects of thermal transients on concrete filled steel modules. The analysis approach for the thermal transient inside the IRWST is discussed in Subsection 3.8.3.3 of this report, and has been found to be acceptable. However, for subcompartment locations inside containment (other than the IRWST) and locations outside containment, rapid heat-up of the steel plate of the

structural wall modules must be considered in the analysis and design of the structural wall module. The concern is that for a rapid temperature transient, the mismatch in thermal conductivity between the steel faceplate and the concrete could impose significant thermal stresses on the faceplate, studs, and concrete core. This could potentially result in degradation of the faceplate/concrete bond and invalidate the assumption of composite behavior. The applicant needs to evaluate the thermal transients that can occur in the various subcompartments, and demonstrate that no unacceptable degradation would result from differential thermal expansion of the steel and concrete throughout the entire transient. This is Open Item 3.8.4.3-1.

Open Item Number: 3.8.4.5-1

Original RAI(s): n/a (April 3, 2003, meeting summary)

Summary of Issue: During the April 2 through 5, 2003, design audit, the design summary report for other Category I structures was not available for staff's review. The applicant stated that the design summary reports are to be completed for staff's review at a later date. The staff considers this as Open Item 3.8.4.5-1.

Open Item Number: 3.8.4.5-2

Original RAI(s): n/a (April 3, 2003, meeting summary)

Summary of Issue: During the course of its review of the Wall 7.3 design calculation, the staff noted that the applicant had previously identified and corrected an error in the equation used by INITEC to calculate the required positive reinforcement for a section subjected to both bending moment and axial load. The staff could not conclude during the audit that the corrected equation accurately calculates required positive reinforcement. Therefore, the applicant was requested to submit the derivation of the equation currently used to calculate the required reinforcement. The applicant was also requested to submit a sample verification calculation for the computer algorithm, and verify that the corrected equation has been utilized in all calculations. This is Open Item 3.8.4.5-2.

Open Item Number: 3.8.5.1-1

Original RAI(s): 230.23

Summary of Issue: In DCD Tier 2 Section 3.8.5.1, the applicant states that the foundation is built on a mud mat, for ease of construction. The mud mat is lean, nonstructural concrete and rests upon the load-bearing rock. Waterproofing requirements are described in DCD Tier 2 Section 3.4.1.1.1. In RAI 230.23, the staff raised a question that the non-structural concrete mud mat cannot withstand the very high toe pressure predicted in the applicant's seismic analysis. This potentially affects the

safety of the NI foundation mat under design basis combination of loads. Since the applicant did not provide its response to the RAI, this issue is designated as Open Item 3.8.5-1.

Open Item Number: 3.8.5.4-1

Original RAI(s): 230.020, 230.021, 230.022

Summary of Issue: The staff's review of DCD Tier 2 Section 3.8.5.4 identified an issue that the potential uplift and slapping back of the containment internal structures foundation on the basemat through the steel containment vessel during a seismic event would affect both the seismic design loads and in-structure response spectra for all structures, systems and components associated with the containment internal structure, and would also affect the seismic response of the steel containment shell. The applicant was requested to perform additional analyses to demonstrate how the uplifting effect will be addressed and how the uplifting affect on the seismic analysis results will be used for the design of the containment and containment internal structures. This is Open Item 3.8.5.4-1.

Open Item Number: 3.8.5-3

Original RAI(s): n/a

Summary of Issue: The staff notes that the specified shear wave velocity for a hard rock site condition should be 2438m/sec (8000 fps), instead of 1067m/sec (3500 fps). This has been previously discussed with and agreed to by the applicant. The applicant needs to verify that the subgrade modulus being used in the analyses represents a rock foundation with shear wave velocity equal to 8000 fps. This is Open Item 3.8.5-3.

Open Item Number: 3.8.5.4-2

Original RAI(s): n/a (April 3, 2003, meeting summary)

Summary of Issue: SRP Section 3.8.5 prescribes the preparation of a design report containing the information listed in Appendix C to SRP 3.8.4. During the April 2 through 5, 2003 audit, the design summary report for the basemat foundation was not available for review by the staff. Completion of the design summary report and review by the staff is Open Item 3.8.5.4-2.

Open Item Number: 3.8.5.5-1

Original RAI(s): 230.018

Summary of Issue: The NS overturning moment (moment about the EW or long axis of the basemat) increases by 42.8 percent while the EW overturning moment (moment about the NS or short axis of the basemat) increases by 10.8 percent. The EW base shear increases by only 1.8 percent while the NS base shear increases by 8.1percent. The reported increases are not consistent with the 9.9 percent increase in the NI mass or 17 percent increase in simple equivalent static overturning moment. In addition, the applicant indicates in its response to RAI 220.018 that the safety factor against overturning for the NS earthquake increases for AP1000, compared to AP600, even though the overturning moment increases by 42.8 percent. Resolution of these apparent discrepancies is Open Item 3.8.5.5-1.

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