Westinghouse
Electric Corporation

Energy Systems

Box 355
Pittsburgh Pennsylvania 15230-0355

October 21, 1996

NSD-NRC-96-4854
DCP/NRC0632

Document Control Desk
U.S. Nuclear Regulatory Commission
Washington, D.C. 20555

ATTENTION: T. R. Quay

SUBJECT: Responses to NRC Questions Related to Building Seismic Design

Dear Mr. Quay:

Attached are responses to two open items related to the seismic design and classification of the turbine building and the radwaste building.

Please contact Donald A. Lindgren on (412) 374-4856 if you have additional questions.

Brian A. McIntyre, Manager
Advanced Plant Safety and Licensing

/jwh

Attachments

cc: D. T. Jackson - NRC

T010

RD-8-2 WEST.

1/1

DSER Item 3.7.2.8-3 (OITS #660)

If the Radwaste Building was to impact on the nuclear island or collapse during an SSE, it would not impair the integrity of the concrete nuclear island. Westinghouse's justification is based on judgement and does not provide an adequate basis to the staff. Please provide adequate basis.

Response:

Failure of the Radwaste Building could result in impact with the Auxiliary Building. Impact of the Radwaste Building on the nuclear island safety related structures would be approximately at the 135 foot elevation of the Auxiliary Building. The roof of the Radwaste Building could impact against the floor of the Auxiliary Building at this elevation.

Two methods are used to provide the basis that a potential Radwaste Building impact on the Auxiliary Building will not impair the integrity of the nuclear island safety related structures and equipment. The two methods employed to demonstrate the acceptability of a potential Radwaste Building impact are:

1) Stress wave evaluation that shows that the stress wave resulting from the impact of the Radwaste Building on the Auxiliary Building has a maximum compressive stress below the concrete compressive stress limit.

2) An energy comparison showing that the energy involved in the subject impact is below the kinetic energy associated with the external missiles that the external walls and roofs of Seismic Category I nuclear island are designed to withstand.

There is a potential for a tensile stress wave to be reflected that could result in cracking of the concrete. This will not impair the integrity of the structure since the reinforcement steel would provide resistance to the stress. Further, the cracking of the concrete would provide energy dissipation and reduce if not eliminate the stress wave.

The impact of the Radwaste building may be at an angle to the Auxiliary Building due to torsional effects, or non-uniform response. This would not result in higher stresses than obtained assuming uniform impact. This is because:

- Impact with any twist of the Radwaste or Auxiliary Building will result with impact at or near the Auxiliary Building shear walls that have significant strength and are structurally rugged;

- Impact will be local with the potential of cracking (brittle behavior) of the concrete that will dissipate the energy and stop (isolate locally) any stress wave response, or the impact will attenuate out where the behavior is like a uniform impact.

The Auxiliary Building floor at the 135' elevation has large openings. These openings will tend to isolate or diffuse the stress wave effect.

Stress Wave Evaluation

An acceptable model for performing this evaluation is an elastic bar subject to an impact load. It is assumed that the Radwaste Building acting as a rigid body moving at a defined velocity, v, impacts

the Auxiliary Building. The impact occurs at the Radwaste Building roof elevation against the Auxiliary Building at a floor elevation. The compressive wave occurs in the Auxiliary Building wall and the floor. These structural elements act as an elastic rod. Formulas given in the reference cited below for maximum compressive stress are used.

Reference: Young, Warren C., Roark's Formulas for Stress and Strain, McGraw-Hill Book Company, Sixth Edition, 1989, pp. 710-719.

The velocity associated with the impact, v, considers the seismic response characteristics of both the Radwaste Building and the Auxiliary Building. The impact velocity is defined as the absolute sum of the maximum seismic velocity of the Radwaste Building and the Auxiliary Building using the AP600 design response spectra for a 0.3g seismic level.

Impact Velocity

Figure 1 prvide a plot of impact velocity versus the range of potential Radwaste building frequencies. It is not expected that the building frequency will be as high as 33 hertz; however, the frequency range is extended to this level for completeness. The impact velocity is the absolute sum of the Radwaste Building velocity and the Auxiliary Building velocity.

In Figures 2 and 3 are given the results from the stress wave analysis. As seen from the results, the ratio of compressive wave stress to fc' does not exceed 0.6 for a concrete compressive stress of 3000 psi.

The conclusion from this analysis is that there is no damage to the Auxiliary Building resulting from Radwaste Building impact that would adversely affect the structural integrity of the Auxiliary Building.

Impact Energy Evaluation

The energy involved in the impact of the Radwaste Building on the Auxiliary Building will not be sufficient to cause damage to safety systems or impair the integrity of the safety class structures. This is demonstrated by comparison of the upper bound kinetic energy associated with the Radwaste Building at the time of impact, to the kinetic energy associated with external missiles that the external walls and roofs of Seismic Category I nuclear island structures must be designed.

The missiles that are part of the AP600 natural phenomena criteria are used to define the kinetic energy that the Seismic Category I nuclear island structures must be designed. In subsection 3.5.1.4 of the AP600 safety analysis report, the following tornado missiles are defined:

1) A massive high-kinetic-energy missile, which deforms on impact. It is assumed to be a 4000-pound automobile impacting the structure at normal incidence with a horizontal velocity of 105 mph or a vertical velocity of 74 mph. This missile is considered at all plant elevations up to 30 feet above grade.

2) A rigid missile of a size sufficient to test penetration resistance. It is assumed to be a 275-pound, eight inch armor-piercing artillery shell impacting the structure at normal incidence with a horizonal velocity of 105 mph or a vertical velocity of 74 mph.

3) A small rigid missile of a size sufficient to just pass through any openings in protective barriers. It is assumed to be a one inch diameter solid steel sphere assumed to impinge upon barrier openings in the most damaging direction at a velocity of 105 mph.

The kinetic energy of the of the Radwaste Building impact is defined at impact in the most conservative manner using the velocity (Figure 1). The velocity is defined as the absolute sum of the Radwaste and Auxiliary seismic velocities.

In Figure 4 is shown the comparison of kinetic energies over a range of resonant frequencies for the Radwaste Building impact. As seen from this figure, the kinetic energy for which the Seismic Category I nuclear island structures are designed is higher than the energy that would be imparted by the Radwaste Building from impact or collapse except for the frequency range below 3 hertz where the armor-piercing artillery shell kinetic energy is exceeded. This is not a problem with respect to the Auxiliary Building structural integrity since:

- the kinetic energy associated with the automobile tornado missile is not exceeded;
- impact of the Radwaste Building against the Auxiliary Building is in an area where it is structurally "rugged";
- the kinetic energy associated with the Radwaste Building impact is conservatively calculated;
- if the combination of seismic velocities associated with the Radwaste Building and Auxiliary Building was by square-root-sum-of-squares (SRSS) and not absolutely combined, the kinetic energy of the armor-piercing artillery shell would not be exceeded.

From the evaluation based on energy it can be concluded that the integrity of the nuclear island safety related structures or systems that are protected by these structures could not be impaired.

Conclusion

From the stress wave and impact energy evaluations, there is an adequate basis to justify that if the Radwaste Building were to impact or collapse on the nuclear island during an SSE, no loss of integrity would occur to safety related buildings or systems.

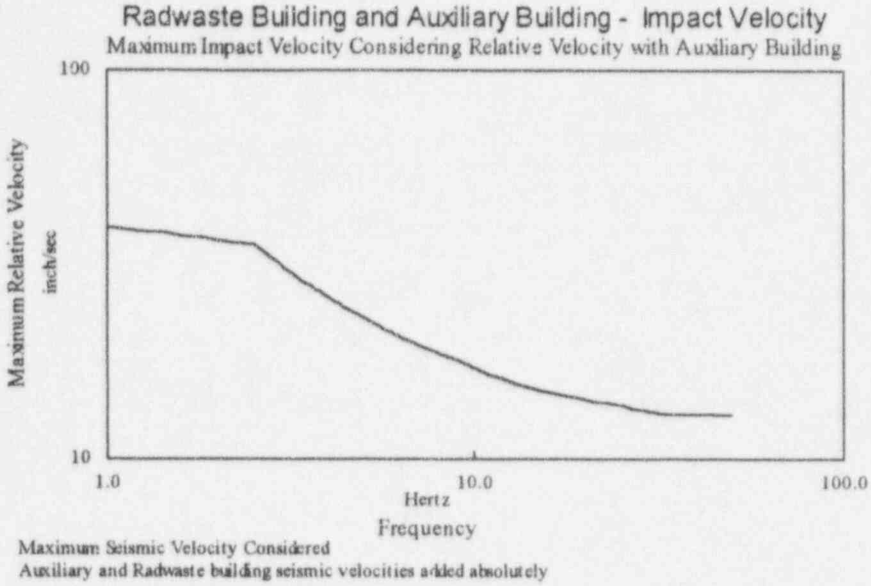

Figure 1 - Impact Velocities Associated With Radwaste and Auxiliary Buildings

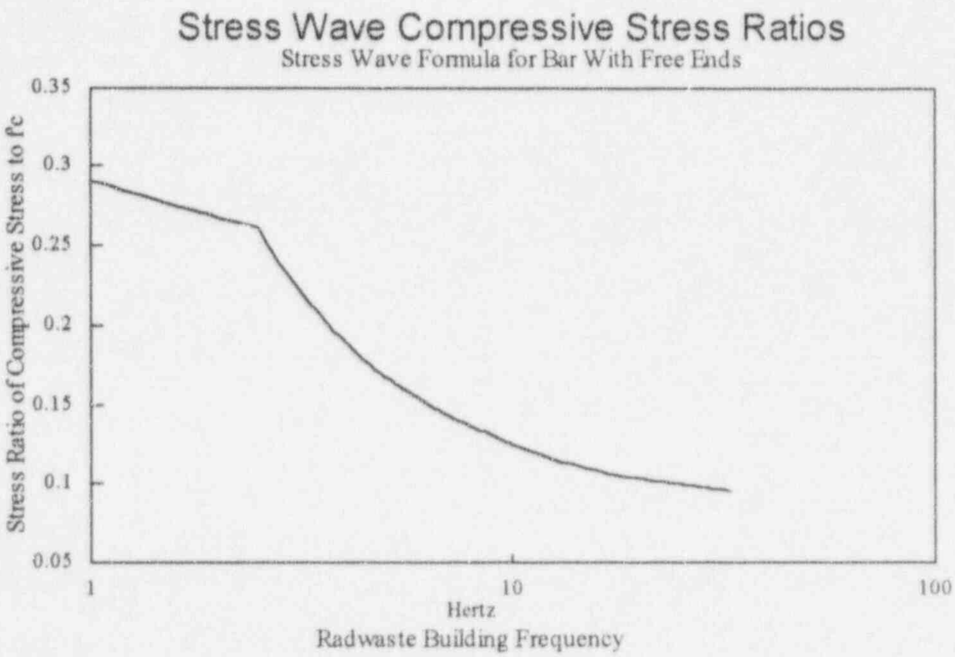

Figure 2 - Stress Wave Compressive Stress Ratio Using Model Having Bar With Fee Ends

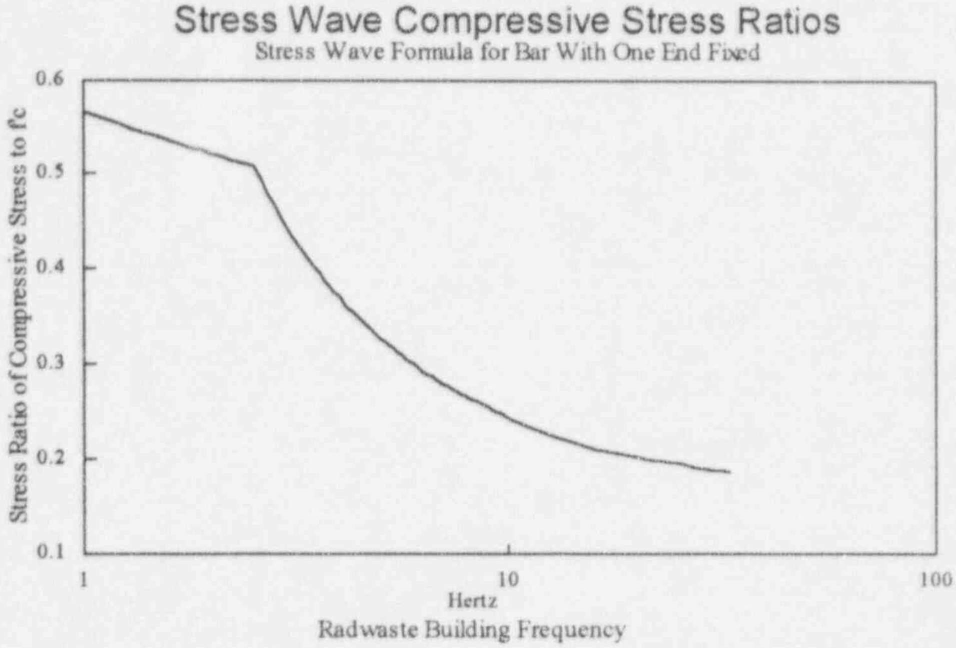

Figure 3 - Stress Wave Compressive Stress Ratios Using Model of Bar With One End Fixed

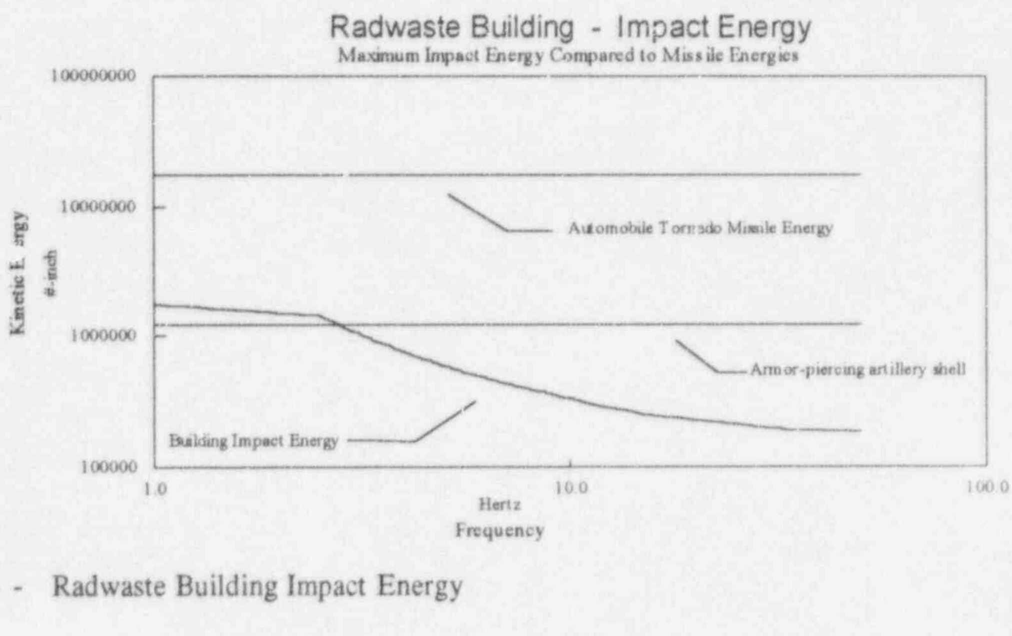

Figure 4 - Radwaste Building Impact Energy

DSER Open Item 3.7.2.8-5 (OITS #662) Turbine Building Seismic Design

NRC Position from summary of meeting to discuss Westinghouse AP600 seismic analysis, NRC letter dated July 18, 1996.

For the turbine building to be classified as nonseismic (to be based on UBC Zone 3 requirements), Westinghouse needs to:

a. demonstrate that the presently-designed turbine building frames with concentric braces can withstand, without collapse, a seismic ground acceleration of 0.3g; (this was further discussed in a telecon - see reference 2)

b. establish post-construction (i.e. as-built) verification of structural members, connections, dimensions, etc. to ensure that these are consistent with the design; and,

c. demonstrate, by quantitative analysis, that the turbine building will not pound on the NI building walls at the turbine building foundation level where the gap is only 2 inches.

Response

On September 12, 1996 a telephone conference call was held between Westinghouse and NRC staff members (see reference 1). The NRC expressed reservations that the concentric braced framed turbine building structure, designed to 1991 Uniform Building Code (UBC) seismic provisions, can withstand the safe shutdown earthquake (SSE). This concern was based, in part, on a 1986 National Science Foundation report (reference 2) on earthquake simulation tests of concentrically braced steel structures that suggests that concentric braced steel structures will not behave adequately in large earthquakes.

A proposed revision to the SSAR clarifying the seismic design basis for the turbine building is provided below. The basis for the AP600 position is described as follows:

The turbine building is a concentrically X-braced steel frame structure. The major structure is separated from the nuclear island by approximately 18 feet. A minimum horizontal clearance of 2 inches is provided between the turbine building's concrete base mat and the adjacent nuclear island as described in subsection 3.8.5.1. This exceeds the relative deflections calculated in the soil structure interaction analyses described in Appendix 2C.

The AP600 turbine building serves no safety related function and is classified as nonseismic. While the turbine building is not specifically designed to withstand an SSE, the seismic design is upgraded to UBC Zone 3 (seismic ground acceleration of 0.3g) to provide a high confidence level of protection against collapse under large earthquake inputs. The design makes collapse under an SSE unlikely and the separation provided between the turbine building and the nuclear island assures the turbine building structure will not pound on the nuclear island in a large earthquake. The nonseismic classification is consistent with the position accepted for the ABWR turbine building (which is designed to protect a safety function of the mainsteam line). The design approach proposed for the AP600 is similar to that proposed for the ABWR. The AP600 turbine building has concentric X-bracing. The bracing system for the ABWR is not clearly defined but is stated by NRC to be eccentric bracing.

The seismic requirements in the 1992-AISC, Seismic Provisions for Structural Steel Buildings are modeled on the 1991 National Earthquake Hazards Reduction Program (NEHRP) earthquake provisions. The bases for the seismic provisions in the 1991 Uniform Building Code are formed by the recommendations of the 1988 Seismology Committee of the Structural Engineers Association of California (SEAOC). The seismic provisions in these specifications and codes have incorporated lessons learned from actual earthquake events and from earthquake simulation tests. The 1986 National Science Foundation report referenced by the NRC is one of these tests and is included in the references listed in the 1992-AISC, Seismic Provisions for Structural Steel Buildings.

The 1986 National Science Foundation report was based on seismic testing conducted on a building with a dual lateral load resisting system consisting of a ductile moment resisting space frame and a concentric K-braced frame, and not an X-braced frame system as used in the AP600 turbine building design. The two types of bracing systems behave differently.

- The K-braced system relies on the post buckling compression capacity of the bracing. This behavior is shown in the attached Figure C-9.6 taken from the commentary of the AISC seismic provisions which shows the tension and compression behavior to be completely different. Lateral resistance requires that both the tension and compression members participate or that there be an alternate load path such as the moment frame of a dual system.

- The X-bracing carries most of the load in the brace in tension after buckling of the compression brace. The positive and negative portions of the load deflection diagram are similar as is shown in the attached Figure 1.2 of the 1986 National Science Foundation report. Lateral resistance is provided by the tension brace even if the compression brace carries no load due to buckling.

The International Conference of Building Officials (ICBO), which promulgates the UBC, and AISC have recognized the significant differences in behavior of K-braced and X-braced structures under high seismic inputs. The UBC and the 1992 AISC Seismic Provisions no longer permit concentric K-braced frames for structures in seismic zones 3 and 4. Concentric X-braced frames, on the other hand, continue to be accepted by both the ICBO and AISC as an acceptable lateral bracing system.

The UBC 1991 code specifies a response modification factor of 8 for concentrically X-braced steel frames and 10 for eccentrically braced steel frames. These are based on allowable stress design for steel structures in which the allowable stress in tension is sixty percent of yield. ASCE 7-93 defines the response modification factors relative to the strength of the structure, specifying 5 for concentrically X-braced steel frames and 7 or 8 for eccentrically braced steel frames, dependent on the type of connections. These factors provide comparable performance for the two framing systems. When AISC permits use of K-braced frames for low seismic regions, it requires a design strength of the bracing to be at least 1.5 times that required for other bracing systems. This effectively reduces the response modification factor from 5 to 3.3 relative to the strength of the structure.

In summary, the concern related to concentric braced systems is appropriate for K-braced frames and has been addressed in the more recent codes and standards. The AP600 turbine building uses X-braced frames. The design methods given in the proposed revision to the SSAR provide performance comparable to the eccentric braced system which NRC staff have previously found to be acceptable.

REFERENCES:

1. Summary of telephone conference to discuss the use of a concentric bracing system for the AP600 turbine building, NRC letter dated September 19, 1996.

2. National Science Foundation Report No. UCB/EERC-86/10, Berkeley, Ca., December 1986, "Earthquake Simulation Tests and Associated Studies of 0.3-Scale Model of a Six Story Concentrically Braced Steel Structure"

Revise subsection 3.7.2.8 as shown below

3.7.2.8 Interaction of Seismic Category II and Nonseismic Structures with Seismic Category I Structures, Systems or Components

Nonseismic structures are evaluated to determine that their seismic response does not preclude the safety functions of seismic Category I structures, systems or components. This is accomplished by satisfying one of the following:

- The collapse of the nonseismic structure will not cause the nonseismic structure to strike a seismic Category I structure, system or component.

- The collapse of the nonseismic structure will not impair the integrity of seismic Category I structures, systems or components.

- The structure is classified as seismic Category II and is analyzed and designed to prevent its collapse under the safe shutdown earthquake.

The structures adjacent to the nuclear island are the annex building, the radwaste building, and the turbine building. The annex building is classified as seismic Category II and is designed to prevent its collapse under the safe shutdown earthquake. The minimum space required between the annex building and the nuclear island to avoid contact is obtained by absolute summation of the deflections of each structure obtained from either a time history or a response spectrum analysis for each structure. The minimum clearance between the structural elements of the annex building above grade and the nuclear island is 4 inches.

The radwaste building is classified as nonseismic and is designed to the seismic requirements of the Uniform Building Code, Zone 2A with an Importance Factor of 1.25. As shown in the radwaste building general arrangement in Figure 1.2-22, it is a small steel framed building. If it were to impact the nuclear island or collapse in the safe shutdown earthquake, it would not impair the integrity of the reinforced concrete nuclear island. The minimum clearance between the structural elements of the radwaste building above grade and the nuclear island is 4 inches.

The turbine building is classified as nonseismic. As shown on the turbine building general arrangement in Figures 1.2-23 through 1.2-30, the major structure of the turbine building is separated from the nuclear island by approximately 18 feet. Floors between the turbine building main structure and the nuclear island provide access to the nuclear island. The floor beams are supported on the outside face of the nuclear island with a nominal horizontal clearance of 12 inches between the structural elements of the turbine building and the nuclear island. These beams are of light construction such that they will collapse if the differential deflection of the two buildings exceeds the clearance and will not jeopardize the two foot thick walls of the nuclear island. The roof in this area rests on the roof of the nuclear island and could slide relative to the roof of the nuclear island in a large earthquake. The seismic design is upgraded from Zone 2A, Importance Factor of 1.25, to Zone 3 with an Importance Factor of 1.0 in order to provide margin against collapse during the safe shutdown earthquake. The turbine building is a concentrically X - braced steel frame structure designed to meet the following criteria:

- The turbine building is designed in accordance with ACI-318 for concrete structures and with AISC for steel structures. Seismic loads are defined in accordance with the 1991 Uniform Building Code provisions for Zone 3 with an Importance Factor of 1.0. For a concentrically X - braced structure the resistance modification factor is 8 (UBC-91, reference 1) using allowable stress design. When using allowable stress design, the allowable stresses are not increased by one third for seismic loads. The resistance modification factor is reduced to 5 for load and resistance factor design (ASCE 7-93, reference 35).

- The nominal horizontal clearance between the structural elements of the turbine building above grade and the nuclear island and annex building is 12 inches.

- ~~Steel structural bracing connections are designed with sufficient strength to develop tensile yield in the bracing before the connection fails.~~ The design of the lateral bracing system complies with the seismic requirements for concentrically braced frames given in section 9.3 of the AISC Seismic Provisions for Structural Steel Buildings. (reference 34). Quality assurance is in accordance with ASCE 7-93 (reference 35) for the lateral bracing system using the quality assurance plan for seismic Category II structures (see subsection 3.2.1.1.2).

Add references to subsection 3.7.6

34. "Seismic Provisions for Structural Steel Buildings," American Institute of Steel Construction, June 1992.

35. "Minimum Design Loads for Buildings and Other Structures," American Society of Civil Engineers, ASCE 7-93.