UNITED STATES
NUCLEAR REGULATORY COMMISSION
WASHINGTON, D.C. 20555-0001

SAFETY EVALUATION BY THE OFFICE OF NUCLEAR REACTOR REGULATION

STATION BATTERY UNREINFORCED MASONRY WALLS

VERMONT YANKEE NUCLEAR POWER CORPORATION

VERMONT YANKEE NUCLEAR POWER STATION

DOCKET NO. 50-271

1.0 BACKGROUND

By letter dated March 15, 1996, Vermont Yankee Nuclear Power Corporation (the licensee) submitted information regarding the operability of two hollow block walls in the battery room at the Vermont Yankee Nuclear Power Station (VYNPS). One wall supports one rack of each of the two main station batteries. This wall is the limiting wall for seismic evaluation and is the wall addressed in the discussions herein. The second wall supports one of three racks of one of the batteries and is bounded by the first wall.

The main station batteries at VYNPS consist of two redundant systems, battery banks B-1-1A and B-1-1B. These battery banks are separated by an 8-inch thick hollow block, masonry wall. Each battery bank is vertically supported by the concrete floor slab. One of three racks in each battery bank is mainly supported in the horizontal directions (i.e., perpendicular to the wall and along the length of the wall) by the masonry wall. The wall is 12 feet 8 inches long, is free on one end and butted to a cross wall on the other end. The wall was originally built to a height of 6 feet from the bottom concrete slab; no reinforcing dowel bars were used between the concrete floor slab and the masonry wall.

As a result of NRC IE Bulletin 80-11 (IEB 80-11), "Masonry Wall Design," the wall height was extended to the full ceiling height of 8 feet 6 inches. A 3 inch x 3 inch x ¼ inch steel angle was anchored to the bottom face of the slab at the top of the wall against each face of the wall top to restrain the wall movement in the direction perpendicular to the wall surface. However, no steel angles or other lateral restraints were placed at the bottom of the wall.

During preparation of the Seismic Evaluation Report for Unresolved Safety Issue (USI) A-46, "Seismic Qualification of Equipment in Operating Plants," the licensee discovered that non-conservative assumptions were used in the original masonry wall calculations for resolution of IEB 80-11. A recalculation showed that the maximum tensile stress in the wall, including consideration of lateral support for the battery racks, as a result of wall bending generated by a safe shutdown earthquake (SSE) was 204 psi while the allowable tensile stress for the SSE (wall starts cracking) was only 35 psi.

Enclosure

The licensee contracted with EQE International, Inc. (EQE) to perform further analysis of the wall. EQE used a mathematical method jointly developed by EQE and RPK Structural Mechanics Consulting (RPK) to analyze the wall in question. EQE's calculation results indicated that the wall can withstand an earthquake load from an acceleration of 0.75g without collapse. EQE subcontracted RPK to review its calculations. RPK concurred with EQE's calculations and commented that two assumptions made by EQE were unnecessarily conservative. The first was that EQE had assumed that the entire 16,000 pound weight of the two battery racks was laterally supported by the wall. The second was that EQE assumed the weight to be uniformly distributed over the entire surface of the wall instead of concentrated at the level of the anchor bolts. By assuming that only 75% of the batteries' weight (12,000 pounds) acts on the wall at the level of the anchor line (33.5 inches from the bottom floor slab), RPK calculated that the wall spectral acceleration capacity was 1.41g. RPK also calculated the effective wall frequency to be 6.5 Hz and stated that the corresponding floor acceleration at that frequency was only 0.5g. Therefore, RPK concluded that the wall and attached battery racks would withstand the SSE.

In a letter dated March 26, 1996, the licensee provided additional information supporting the use of the EQE-RPK method. In a letter dated April 12, 1996, the licensee committed to modify the support of the battery racks no later than May 3, 1996, to restore the masonry wall and the battery rack to full conformance with the licensing basis.

2.0 EVALUATION

2.1 Seismic Capacity Evaluation

The EQE-RPK method postulates arching action of the wall under SSE load to calculate a collapse load of the masonry wall. The arching action phenomenon of an unreinforced masonry wall has been observed experimentally. During static tests, masonry walls are simply supported at both ends of the wall height. The ends can rotate but will not move in the direction perpendicular to the face of the wall. Air bags are usually used in between a reaction frame and the wall surface to supply the load, and wind pressures can be simulated by pumping a proper amount of air into the bags. Hydraulic jacks are used for creating concentrated loads. The wall bends as pressure increases. When the bending stress in the wall reaches the tensile strength of the wall, the wall usually cracks along a horizontal line. Once the horizontal crack is formed, the section of the wall above the crack line and the piece below the line rotate slightly because the supporting ends are free. Since the wall has a finite thickness and because of the geometry, the rotation will cause an upward movement for the upper section and a downward movement for the lower section of the wall. If the supporting ends of the wall are restrained in the vertical direction, then the vertical movements of the wall sections create vertical forces in the wall. When the applied pressure increases further, the wall deflects more horizontally, and the crack size, rotation of the wall, and vertical forces in the wall all increase as well. Thus, the wall behavior subjected to loads acting perpendicular to the plane of the wall surface is similar to that of a three-hinge-arch bridge subjected to loads acting perpendicular to the surface of the bridge.

Based on the observation of the arching action from tests, theories have been developed to predict the collapse load of unreinforced masonry walls. Most theories are based on two criteria: a stability limit and a strength limit. The EQE-RPK method considers the strength limit only. The strength limit needs an assumption on the thickness of the wall being allowed to be crushed locally before the entire wall collapses. In the EQE-RPK method, this thickness was assumed to be ¼ of the original wall thickness, or 1 inch. In addition to this important assumption, there were several other assumptions made in the derivation of the EQE-RPK method and in the application of the method. Another important assumption made in the VYNPS calculations is that the vertical restraining force on the wall is dependent on the shape and magnitude of deflections of the floor slabs above and below the wall when they are being pushed by the wall. This requires calculation of the stiffness of the floor slabs above and below the wall considering the location and weight of equipment and other objects on the slabs. In the EQE calculation, the location and weight of equipment and other objects on the slabs were ignored. EQE assumed that the concrete floor slabs were uncracked and calculated the floor slab stiffness under that assumption. EQE also assumed that the vertical restraining force on the wall was limited by the flexural cracking strength of the concrete floor slabs. EQE used the equation provided for calculating tensile strength of plain concrete in the Uniform Building Code for that purpose. Since the floor slabs are supported by beams, the beam stiffnesses were also included in the process of calculating the vertical restraining force on the wall at VYNPS. The staff considers that the flexural cracking strength of plain concrete varies greatly and is unreliable because most floor slabs are actually cracked due to shrinkage or loads, and the use of uncracked concrete sections for calculating the restraining force on the wall is improper.

Arching action has also been observed on a single story masonry house tested on a shake table for seismic responses at the University of California at Berkeley (Report No. UCB/EERC-79/23, September 1979). The wall was constructed from 6-inch thick unreinforced hollow masonry blocks with a height of 8 feet 8 inches. The bottom of the wall was positively anchored to the shake table, and roof trusses were positively anchored to the top of the wall. Actual earthquake records were input to the shake table. It was reported that during the test a horizontal crack formed at ¾ of the wall height. At an acceleration of 0.21g minor hinging (arching) of the wall was observed at the crack joint and horizontal deflection at the crack joint level was measured at 0.25 inches. The report further stated that significant hinging (arching) was observed at an acceleration of 0.31g, and that the horizontal deflection at the crack joint level was measured at 2 inches. This phenomenon indicates that the wall stiffness degraded substantially as indicated by the change in wall deflection from 0.25 inches to 2 inches from the stage of minor hinging to significant hinging. This significant degradation in stiffness does not appear to have been considered in the EQE-RPK calculations. EQE calculated the horizontal deflection in the wall at the hinge as being 0.46 inches, but did not calculate effective wall frequency. RPK calculated the horizontal deflection in the wall at the hinge as being 0.418 inches with an effective wall frequency of 6.5 Hz. Both the wall at VYNPS and the test wall are built from hollow masonry blocks and have about the same height. The physical differences are the wall thickness (6 inches for the test wall vs. 8 inches at VYNPS) and anchoring (positive anchoring in both the vertical and horizontal directions at both the top and bottom of the test wall vs. only horizontal restraint at only the top of the wall at VYNPS). Also, EQE and RPK did not use the actual boundary (anchorage) conditions of the VYNPS wall to perform their analyses, and instead used boundary conditions identical to that of the test wall. Therefore, the analytical models of the two walls become identical except for the wall thickness and the mass and additional lateral loads of the two batteries.

Due to the many similarities between the analytical models, the staff used the results from the shake table test as a rough measure for judging the acceptability of the analytical method developed by EQE and RPK. While the geometrical models of the two walls are about the same, except for wall thickness, the seismic inertia load and the wall responses of the two walls are quite different. While the seismic inertia load of the test wall was generated only by its own mass (weight), the VYNPS wall has the battery weights, which are about three times heavier than its own wall weight, attached to the wall. While the test wall had a wall deflection of 2 inches at 0.31g, RPK reports that the VYNPS wall will only deflect 0.417 inches at 1.41g. Recognizing that the VYNPS wall is stronger than the test wall because of the wall thickness difference, and also recognizing that the seismic inertia load of the VYNPS wall is about three times greater than that of the test wall, the staff cannot reconcile the big difference in the responses of the two walls (2 inches deflection at 0.31g for the test wall vs. 0.417 inches deflection at 1.41g for the VYNPS wall in the RPK calculation.)

Aside from the mathematical assumptions associated with the EQE-RPK method as described above, the chief problem is whether the EQE-RPK method derived from the arching action can be applied to the unreinforced masonry wall in question at VYNPS. The wall was originally built for a height of 6 feet as a partition, and was later extended to the full ceiling height of 8 feet 6 inches. The wall was purposely designed and built as a partition only and not as a load bearing wall so that no vertical loads from the slab above can be transmitted to the partition wall and compress it. To achieve that purpose, it is usual to leave a gap between the top half course of masonry blocks and the bottom face of the upper slab. Mortar is often applied at the perimeter of the wall gaps for cosmetic reasons. Since steel angles were installed to sandwich the top of the wall, application of mortar along the length of the wall could not be verified because the area is concealed by the steel angles. The wall in question at VYNPS is a partition wall and not a load bearing wall by design and construction. The existence of a gap between the top of the wall and the ceiling slab makes the method developed by EQE and RPK inappropriate for the wall at VYNPS because the gap or void space cannot generate the vertical restraining force on the wall or slab which is required to create the arching action in the wall that is assumed in that method.

2.1 Corrective Action Plan

By letter dated April 12, 1996, the licensee committed to modify the battery racks no later than May 3, 1996, to ensure that both the battery racks and the masonry walls are in full conformance with their design basis requirements. The licensee stated that it has conducted visual inspections of the walls to verify that there are no structurally degrading cracks. The licensee further stated that it has analyzed the walls without the battery loads attached and with the holes remaining where the bolts attaching the battery racks will have been removed. This analysis was performed using the linear elastic method which is part of the NRC-approved design basis of the wall. The staff finds that the licensee's plan can reasonably be expected to restore the station battery racks and block wall to full conformance with their design basis requirements and is reasonably prompt. Therefore, the staff finds the licensee's commitment acceptable.

3.0 CONCLUSION

The staff has reviewed the licensee's submittals on the masonry walls at VYNPS. The licensee contracted EQE to perform an analysis for the seismic capacity of the wall. EQE used a method developed by EQE and RPK and subcontracted RPK to review its calculations. EQE and RPK concluded that the wall at VYNPS had sufficient capacity to withstand the SSE for VYNPS. The staff compared the licensee's analytical results with test results from a similar wall and cannot reconcile the differences. In addition, the method developed by EQE and RPK assumes arching behavior which requires that vertical compressive forces be transmitted from the wall to the upper and lower boundaries. This is not the case for the partition wall in question at VYNPS. While the staff has doubts about the adequacy of the method developed by EQE and RPK to predict the collapse load of unreinforced masonry walls during earthquakes in general, the staff has specifically concluded that the method developed by EQE and RPK is not applicable to the station battery masonry wall at VYNPS. However, the licensee has committed to promptly implement modifications to the battery racks which will render both the battery racks and the masonry wall in full conformance with their design basis requirements. The staff finds the licensee's commitment acceptable.

Principal Contributors: J. Ma
D. Dorman

Date: April 25, 1996