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ComEd

March 10, 2000

United States Nuclear Regulatory Commission
Attention: Document Control Desk
Washington, D.C. 20555

LaSalle County Station, Units 1 and 2
Facility Operating License Nos. NPF-11 and NPF-18
NRC Docket Nos. 50-373 and 50-374

Subject: Response to Request for Additional Information
NRC Inspection Report 50-373/99020, 50-374/99020

References: (1) Letter from J. L. Caldwell (U.S. NRC) to O. D. Kingsley (ComEd), "Reply to Non-Cited Violation for NRC Inspection Report 50-373/99020; 50-374/99020," dated February 8, 2000.

(2) Letter from J. A. Benjamin (ComEd) to U.S. NRC, "Reply to Non-Cited Violation for NRC Inspection Report 50-373/99020; 50-374/99020," dated December 21, 1999.

This letter provides information requested of the Commonwealth Edison (ComEd) Company by the NRC letter dated February 8, 2000, Reference 1. In Reference 1, the NRC acknowledged receipt of the ComEd letter dated December 21, 1999, Reference 2, that contested a non-cited violation contained in NRC Inspection Report 50-373/99020, 50-374/99020 for the LaSalle County Station. The NRC in Reference 1, requested additional information to address the design control issue for anchor bolt stiffness values used in pipe support calculations, and to resolve the fundamental issue related to the appropriateness of modeling the structural attachments to base plates as pinned connections. Based on discussions with Mr. Gary Shear of the NRC Region III Staff and ComEd's Mr. Rod Krich on March 9, 2000, it was agreed that ComEd could delay our response until March 10, 2000.

IE01

This letter provides, as an attachment, a response to the request for additional information (RAI), attached to the Reference 1 letter. Based on discussions with Mr. Gary Shear of the NRC Region III Staff and ComEd's Mr. Rod Krich on March 9, 2000, it was also agreed that we could delay our response to the issues described in the body of the Reference 1 letter until after a technical meeting between ComEd and the NRC is held, tentatively scheduled for the first week in April 2000. Based on the results of that meeting, ComEd will submit any additional information considered necessary to resolve this matter.

Should you have any questions concerning this letter, please contact Mr. Frank Spangenberg, III, Regulatory Assurance Manager, at (815) 357-6761, extension 2383.

Respectfully,

Charles G. Pardee
Site Vice President
LaSalle County Station

Attachment

cc: Regional Administrator – NRC Region III
NRC Senior Resident Inspector – LaSalle County Station

1. Demonstration that Bending Moments for Anchor Bolts Analysis are Conservative

Calculation No. L-002424, Rev. 0, Page 12, is a flow chart for the semi-rigid analysis of a pipe support anchorage assembly at LaSalle County Station. The pipe support anchorage assembly consists of a steel member and an anchorage. One end of the steel member is attached (welded) to a steel plate, and the plate is attached to concrete by concrete expansion anchor bolts. The other end of the steel member is attached to another steel member(s) or another anchorage. The semi-rigid analysis divides the pipe support anchorage assembly into two sub-structures at the junction of the base plate and the steel member. This technique is usually called "sub-structuring." The semi-rigid analysis is to solve the two sub-structures (an anchorage and a frame) separately for the angle of rotation at the cut location, which is common to both sub-structures, and set the two angles of rotations being equal as a condition to find the correct bending moment at the cut location. Your procedure was based on applying an assumed bending moment to the anchorage, and calculating a corresponding angle of rotation, θ, of the anchorage by a proprietary computer code APLAN. You used a computer code STADD-III, which is in the public domain, to analyze the frame with an hinged end subjected to external loads, in order to calculate an angle of rotation, θpin, at the junction of the steel member and the steel plate. You then concluded that the bending moment that you had assumed in the APLAN code analysis would be conservative, if θ is equal to or greater than θpin.

The staff finds that you did not perform the sub-structuring technique properly. The proper sub-structuring technique requires that the continuity be maintained at the cut location. Therefore, if there is a bending moment acting on the sub-structure of the anchorage at the cut location, the same bending moment must also apply to the frame at the same cut location to satisfy the continuity requirement. However, you did not apply that bending moment to the frame. Due to the omission of the bending moment acting on the cut location of the frame, the value of the θpin that you calculated may in some cases be less than the correct value had the bending moment been applied to the frame. Consequently, the bending moment that you had assumed or obtained through your sub-structuring technique may also be less than the correct bending moment had the sub-structuring technique been properly performed. Therefore, you are requested to demonstrate that your sub-structuring technique has yielded the correct or a conservative bending moment at the cut location for anchor bolts analysis, preferably with numerical examples.

<u>Response to Question 1</u>

The analysis in calculation L-002424, Rev. 4, was not intended to perform a structure sub-structure analysis. The rationale described below, which is the basis for the methodology presented in calculation L-002424, is not to show or prove convergence between the frame model (STAAD) and the anchorage model (APLAN). Rather, the rationale demonstrates that a simplified, conservative, bounding analysis can be performed to show adequate anchorage design. The discussion below clarifies the method presented on pages 9 through 12 of calculation L-002424.

The θ_{pin} value described on page 12 of calculation reflects the maximum angular rotation that the supporting member will experience given a pinned boundary condition on the frame analysis. As the degree of fixity is increased, the angular rotation will decrease. The angular rotation will be zero for a theoretically fixed condition of the member.

The θ_{bp} value reflects the angular rotation that the base plate will experience at the attachment point of the framing member.

Based on experience and past analyses (reference calculation L-002379, Rev. 1), 20% of the fixed end moment (FEM) of the frame was chosen as the starting point in the iteration process. The purpose of this iteration process is not to reach convergence between the two models, but to reach a conservative value of the percentage of the FEM where the simplified, bounding analysis method will apply.

The 20% of the FEM was applied to the base plate and via the APLAN analysis, the resulting angular rotation, θ_{bp}, of the base plate was calculated.

If θ_{bp} exceeds θ_{pin}, then 20% of the FEM is bounding, and a smaller percentage of the FEM is required to ensure compatibility between the frame model, and the anchorage (i.e. APLAN baseplate) model and thus achieve an exact solution. Further iterations are not required because the true moment that results in convergence will have to be less that the 20% of FEM value. Accordingly, the application of 20% of the FEM to the APLAN analysis is conservative for the anchorage design.

If θ_{pin} exceeds θ_{bp}, then 20% of the FEM may not be bounding. Accordingly, a greater percentage value of FEM is needed to ensure compatibility between the frame model and the anchorage model. The iteration process requires a higher value of moment be applied to the base plate until θ_{bp} equals or exceeds θ_{pin}. When θ_{bp} exceeds θ_{pin}, a bounding value for the applied bending moments onto the anchorage (baseplate) is achieved, as described in the previous paragraph.

The flowchart on page 12 of the calculation L-002424 reflects the above logic. Based on the above, the methodology used is sound, and ensures an adequate baseplate design.

2. Qualification of APLAN Code in the Calculation of the Rotational Stiffness of an Anchorage

   APLAN code calculated the rotational stiffness of an anchorage being 86.3 kip-in/degree when a bending moment of 102.89 kip-in was applied to the anchorage (Calc. No. L-002379, Rev. 1, Page 9). APLAN code calculated the rotational stiffness of the same anchorage being 48.8 kip-in/degree when a bending moment of 17.72 kip-in was applied to the anchorage (Calc. No. L-002379, Rev. 0, Page 11). The rotational stiffness of the anchorage at the lower bending moment is about 57 percent of that at the higher bending moment. This relationship between applied bending moments and corresponding rotational stiffness of an anchorage is not supported by test data. Test data from connections of steel or concrete have indicated that the rotational stiffness of a connection (joint) will remain the same in a region with low bending moments and then start decreasing as bending moments increase. Therefore, we request a technical justification for the results obtained by the APLAN code.

Response to Question 2

Calculation L-002379, Revision 1 was prepared as a study to evaluate the modeling of concrete expansion anchor baseplates for support VG01-0024X. In the calculation, the subject hanger was analyzed with semi-rigid connections at the concrete expansion anchor baseplate connections. Several iterations of the hanger frame were run until convergence was achieved between the member rotations and the Concrete Expansion Anchor (CEA) assembly rotations.

The NRC question states that "Test data from connections of steel or concrete have indicated that the rotational stiffness of a connection (joint) will remain the same in the region with low bending moments and then start decreasing as bending moments increase." NRC has not provided the reference to the test or the actual test results. Thus, we can not determine the applicability of the test data to the CEA assemblies subjected to axial tension and moments.

The CEA assemblies for the hangers in question are subjected to a combined axial tension and moment. For assemblies subjected only to moments, the test behavior as stated by the NRC is plausible. As the moment is increased, less and less of the plate is in contact with the concrete, resulting in lower rotational stiffness. However, when significant axial tension is present, the behavior is more complex. When the tension is large compared to the bending moment, the CEA assembly has a smaller rotational stiffness because the tension load causes the CEA assembly to lift off the concrete resulting in a lower rotational stiffness. As the moment becomes large, the moment causes part of the plate to come in contact with the concrete resulting in a larger rotational stiffness. The computed values in calculation L-002379 are consistent with the behavior expected when both axial tension and moment are present.

Also, it should be noted that the program used for baseplate analysis (APLAN) is a fully validated and controlled program. The validation consisted of running several baseplate problems with APLAN, and comparing these results against analysis results of the same baseplates analyzed with ADINA.

3. Justification for Using the Lower Acceleration Values of Earthquakes for Support Analysis

   You indicated that the acceleration values for support M09-VG01-0024X resulting from OBE are higher than that resulting from SSE (Calc. L-002291, Rev. 0, Page 8). You used the acceleration values of SSE for support analysis (Calc. L-002291, Rev. 0, Page 8). Provide your explanation as to why the OBE loads are higher than the SSE loads at the support location. Also provide your rationale for using the lower earthquake acceleration values for support analysis.

Response to Question 3

The SSE allowable stresses are equal to or less than 1.6 times the OBE allowable stresses. Thus, based on the relatively large SSE pipe reaction load on the hanger, Calculation L-002291, Revision 0, determined that the SSE load case governed the design of the hanger (6673 lbs. pipe load for the SSE case compared to 2117 lbs. pipe load for the OBE case). The self-weight excitation loading for OBE is larger than SSE. However, the self-weight excitation loading is considerably smaller than the piping loads and does not change the conclusion that SSE loading combination is the governing load combination. It is appropriate to use the SSE g-values for self-weight seismic effects on the pipe support for the SSE load combination. Therefore, the calculation is correct.

In regards to why OBE g-values exceed SSE g-values, LaSalle County Station's plant is founded on soil. A Soil-Structure Interaction (SSI) analysis was used to compute the seismic response and floor response spectra for the plant. The SSI model uses strain dependent soil properties. This analysis is fully described in LaSalle County Station's UFSAR. The higher ground motions for SSE leads to higher soil strains resulting in lower soil shear modulus for the SSE when compared to the OBE. Thus, the resulting SSI motions at the base mat for SSE and OBE have different frequency content. These motions are amplified differently within the structure for OBE and SSE because the frequency content of the SSI motions is different. For the spectra in question, the floor slab response is more highly amplified by the OBE motions than by the SSE motions. This together with the lower structural and lower floor response spectra damping for OBE when compared to the SSE, results in the OBE spectra peaks that are higher than the SSE spectra peaks. The damping values used are in accordance with LaSalle County Station's licensing basis. The floor spectra for both SSE and OBE are correct.