CATEGORY 1

REGULATORY INFORMATION DISTRIBUTION SYSTEM (RIDS)

ACCESSION NBR:9611260345 DOC.DATE: 96/11/15 NOTARIZED: NO DOCKET #
FACIL:50-220 Nine Mile Point Nuclear Station, Unit 1, Niagara Powe 05000220
AUTH.NAME AUTHOR AFFILIATION
TERRY,C.D. Niagara Mohawk Power Corp.
RECIP.NAME RECIPIENT AFFILIATION
Document Control Branch (Document Control Desk)

SUBJECT: Responds to 960820 trip rept re audit of plant reactor & turbine bldg blowout planel calculations.

DISTRIBUTION CODE: A001D COPIES RECEIVED:LTR 1 ENCL 1 SIZE: 6
TITLE: OR Submittal: General Distribution

NOTES:

	RECIPIENT ID CODE/NAME	COPIES LTTR	COPIES ENCL	RECIPIENT ID CODE/NAME	COPIES LTTR	COPIES ENCL
	PD1-1 LA	1	1	PD1-1 PD	1	1
	HOOD,D	1	1			
INTERNAL:	ACRS	1	1	FILE CENTER 01	1	1
	NRR/DE/EMCB	1	1	NRR/DRCH/HICB	1	1
	NRR/DSSA/SPLB	1	1	NRR/DSSA/SRXB	1	1
	NUDOCS-ABSTRACT	1	1	OGC/HDS3	1	0
EXTERNAL:	NOAC	1	1	NRC PDR	1	1

C A T E G O R Y 1 D O C U M E N T

NOTE TO ALL "RIDS" RECIPIENTS:
PLEASE HELP US TO REDUCE WASTE. TO HAVE YOUR NAME OR ORGANIZATION REMOVED FROM DISTRIBUTION LISTS OR REDUCE THE NUMBER OF COPIES RECEIVED BY YOU OR YOUR ORGANIZATION, CONTACT THE DOCUMENT CONTROL DESK (DCD) ON EXTENSION 415-2083

TOTAL NUMBER OF COPIES REQUIRED: LTTR 13 ENCL 12

NIAGARA MOHAWK
GENERATION
BUSINESS GROUP

NINE MILE POINT NUCLEAR STATION/LAKE ROAD, P.O. BOX 63, LYCOMING, NEW YORK 13093/TELEPHONE (315) 349-7263
FAX (315) 349-4753

CARL D. TERRY
Vice President
Nuclear Engineering

November 15, 1996
NMP1L 1155

U. S. Nuclear Regulatory Commission
Attn: Document Control Desk
Washington, DC 20555

RE: Nine Mile Point Unit 1
Docket No. 50-220
DPR-63

Subject: *Response to Trip Report for August 20, 1996 Audit of Reactor and Turbine Building Blowout Panels (Tac. No. M94858)*

Gentlemen:

The NRC's October 7, 1996 Trip Report regarding an audit of the Nine Mile Point Unit 1 (NMP1) Reactor and Turbine Building Blowout Panel Calculations, included a list of action items identified during the audit to be completed by Niagara Mohawk. Enclosure 1 of this letter provides Niagara Mohawk's response to each of these action items.

Very truly yours,

C. D. Terry
Vice President - Nuclear Engineering

CDT/AFZ/lmc
Enclosures

xc: Mr. H. J. Miller, Region I Administrator
Mr. S. S. Bajwa, Acting Director, Project Directorate I-1, NRR
Mr. B. S. Norris, Senior Resident Inspector
Mr. D. S. Hood, Senior Project Manager, NRR
Records Management

A001 1/1

ENCLOSURE 1
NIAGARA MOHAWK POWER CORPORATION
NINE MILE POINT UNIT 1

Response to Action Items from August 20, 1996
NRC Audit of Blowout Panel Calculations

Action Item

- Adjust the linear scaling method used to obtain pressure capacities of both the Reactor and Turbine Buildings and submit the revised analysis results.

- Eliminate the use of strain rate effect in establishing the Fy value used in capacity evaluation for the upcoming USAR.

Response

As discussed during the audit, Niagara Mohawk has revised the Reactor and Turbine Building superstructure calculations by eliminating the use of the 1.05 dynamic load factor (strain rate factor for increasing Fy of material) for computing the building capacities. In addition, this revision to the calculations also included the adjustment for the dead load scaling factor as per the NRC's recommendations. A separate loading case for the "dead load" was analyzed for both Reactor and Turbine Buildings using the same COSMOS models. The forces and moments due to the "dead load" case were separately accounted for in determining the actual failure internal pressures for the superstructures. The revised lower bound failure internal pressures for superstructures are as follows:

Reactor Building:	Secondary member, roof bracing (117 psf) and Primary member, critical column (124 psf)
Turbine Building:	Secondary member, roof truss (135 psf) and Primary member, critical column (150 psf)

NOTE: A reduction factor of 0.9 (ϕ) was also used in computing these capacities of superstructures, which is in conformance with AISC LRFD methods used to compute the failure loads.

The UFSAR was updated to identify this revised lower bound failure load of Reactor Building superstructure as 117 psf (internal pressure) and lower bound failure load of Turbine Building superstructure as 135 psf (internal pressure).

Action Item

- Provide a paragraph justifying why it is conservative to use the mean minus one standard deviation for the value of Fy.

Response

In computing the lower bound capacities of the Reactor and Turbine Building superstructures, Niagara Mohawk has used the smallest yield strength of material from the mill certificates. This smallest Fy is also less than the mean minus a standard deviation. Additionally, for the UFSAR update, the building capacity calculations also used a reduction factor of 0.9(ϕ) as described above. Therefore, the yield strength used in the evaluation is a conservative value that determined the lower bound capacities of the superstructures.

Action Item

- Further explain the basis for using 0.62 tension allowable as the shear allowable, and further discuss the available test data.

Response

A total of 13 bolts were tested to determine their tensile failure load. However, no bolts were tested to determine their shear failure load. As the bolts in the blowout panels are subjected to both shear and tension, Niagara Mohawk used the best available industry data identifying the correlation between shear and tensile failure loads in lieu of testing the bolts for shear strength. The reference used (identified below) concludes that the average shear strength of a bolt is approximately 62% of its tensile strength. This ratio of shear strength to the tensile strength was developed from test data acquired at the University of Illinois and Lehigh University as documented in the reference.

In order to ensure that the calculations result in the upperbound values for the blowout pressures on panels, Niagara Mohawk has used the highest tested tensile strength (maximum of 13 tests) for both Reactor and Turbine Buildings, i.e., the same highest tested value of tensile strength was used for Reactor Building panels and Turbine Building panels. Similarly, 62% of this highest tested tensile strength was used as the shear strength of the bolt, as discussed earlier.

Reference:

G. L. Kulak, J. W. Fisher, and J. H. A. Struik, Guide to Design Criteria for Bolted and Riveted Joints; John Wiley and Sons, Inc., 1987, pages 47 and 48 (pages 47 and 48 are reproduced and provided as Attachment A).

Action Item

- Provide a written rationale for selecting the number of bolts tested.

Response

The blowout panels in Reactor Building are located in three column bays, and in the Turbine Building in five column bays, making a total of eight column bays. The rationale for selecting the bolts was to pick one bolt from each side of each column bay. Thus 8 x 2 = 16 bolts were removed for testing. Three bolts were damaged, therefore, the remaining 13 bolts were tested for their tensile failure load. The results of these 13 tested bolts were used in computing the failure load of blowout panels in the Reactor and Turbine Buildings.

Action Item

- Explain why the type of stitch welds used justifies the rigid assumption for the angle to column joint connection in the finite element model (ADINA).

Response

In analyzing the FKX panel using the finite element program (ADINA), the model consisted of ∠ 3 x 2 x 1/4 (to which the panels are attached) and only the 2" leg of ∠ 2 1/2 x 2 x 1/4 (which is welded to the building column). A fixed condition was assumed at this end based on the fact that the stitch weld between the ∠ 2 1/2 x 2 x 1/4 and the building column provides the necessary fixity by engineering judgement, and based on the fact that the legs of ∠ 2 1/2 x 2 x 1/4 are capable of resisting the flexural stresses and remaining within code allowables. As only one leg of ∠ 2 1/2 x 2 x 1/4 was modeled, this end has the capacity to resist moment and shear. By engineering judgement, the stitch weld was assumed adequate for these moments and shears and to provide sufficient restraint to the 2 1/2 angle leg so as to prevent any rotation of the ∠ 2 1/2 x 2 angle.

The analysis has demonstrated that the effects of the side connections are insignificant in computing the shear and tension on bolts.

Action Item

- Provide a written discussion of the code reconciliation effort previously implemented in the area of AISC design and acceptance criteria.

Response

Niagara Mohawk has used the eighth edition of the AISC Code even though the plant was constructed using the sixth edition. However, it should be noted that Niagara Mohawk has reconciled these two editions of the AISC Code, and this reconciliation is documented in Chapter XVI of the UFSAR. The technical basis for reconciliation is documented in Structural Design Report SDR-004, a copy of which was provided for the NRC inspectors on August 20, 1996, during the audit.

to the tensile strength is independent of the bolt grade, as illustrated in Fig. 4.13. The shear strength is plotted versus the tensile strength for various lots of A325 and A490 bolts. The average shear strength is approximately 62% of the tensile strength.

The variance of the ratio of the shear strength to tensile strength, as obtained from single bolt tension shear jigs, is shown in Fig. 4.14. A frequency curve of the ratio of shear strength to tensile strength was developed from test data acquired at the University of Illinois and Lehigh University. The average value is equal to 0.62, with a standard deviation of 0.03.

Tests on bolted joints indicated that the initial clamping force had no significant effect on the ultimate shear strength.[4.5-4.7] A number of tests were performed on A325 and A490 bolts torqued to various degrees of tightness and then tested to failure in double shear. The results of tests with A490 bolts are shown in Fig.

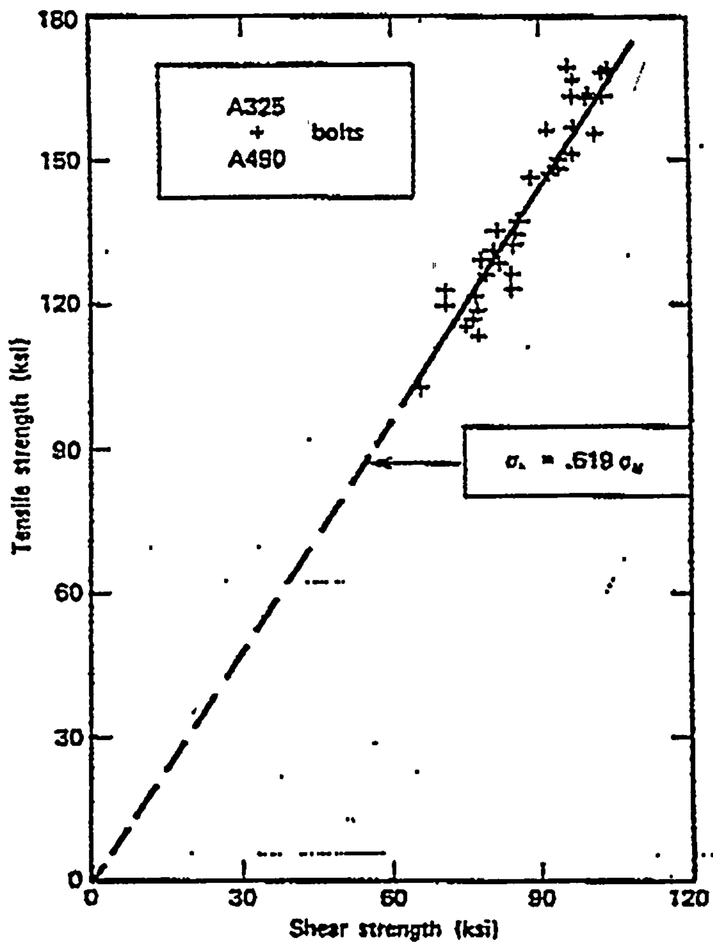

Fig. 4.13. Shear strength versus tensile strength. *Note.* Each point represents the average values of a specific bolt lot. The shear strength is computed on the relevant area, depending on the location of the shear plane.

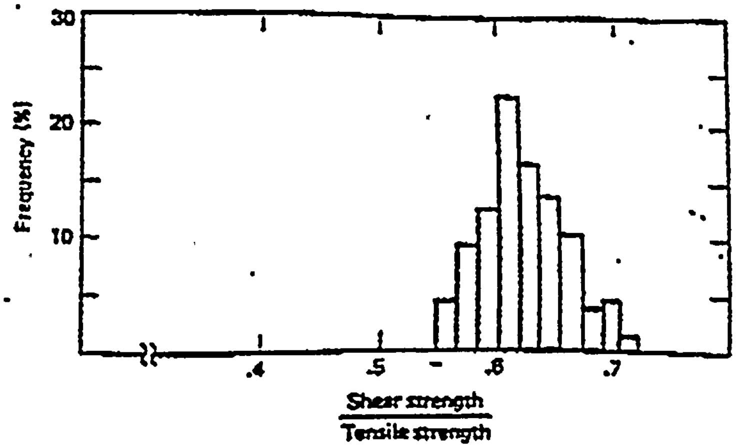

Fig. 4.14. Frequency distribution of ratio of shear strength to tensile strength for A325 and A490 bolts. Number of tests, 142; average value, 0.62; standard deviation, 0.03.

4.15.[4.4] The lower portion shows the relationship between bolt shear strength and the initial bolt elongation after installation. The bolt preload was determined from measured elongations and the torqued tension relationship given in the upper portion of Fig. 4.15. The results confirm that no significant variation of shear strength occurred when the initial bolt preload was varied.

There are two sources of tensile load in the bolt that should, theoretically, interact with the shear load and result in a failure load that is less than that from shear alone. These are (1) the bolt preload induced during the installation procedure, and (2) bolt tension resulting from prying action in the plates.

Measurements of the internal tension in bolts in joints have shown that at ultimate load there is little preload left in the bolt.[4.6,4.7,4.25] The shearing deformations that have taken place in the bolt prior to its failure have the effect of releasing the rather small amount of axial deformation that was used to induce the bolt preload during installation.

At any level of load producing shear in the bolts, prying action of the plates can also produce an axial tensile load in the bolts. In most practical situations, however, the tensile stress induced by prying action will be considerably below the yield stress of the bolt; therefore, it has only a minor influence. Studies of bolts under combined tension and shear have shown that tensile stresses equal to 20 to 30% of the tensile strength do not significantly affect the shear strength of the bolt.[4.8]

The shear resistance of high-strength bolts is directly proportional to the available shear area. The available shear area in the threaded part of a bolt is equal to the root area and is less than the area of the bolt shank. For most commonly used bolts, the root area is about 70% of the nominal area. The influence of the shear plane location on the load versus deformation characteristics of A325 and A490 bolts is reported in Ref. 4.4. Figure 4.16 shows the influence of the shear plane