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SUBJECT: Forwards response to questions re Insp Rept 50-220/96-05 & technical questions re engineering calculations & analyses related to problem.

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CARL D. TERRY
Vice President
Nuclear Engineering

July 3, 1996
NMP1L 1096

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 Questions in Enclosure 2 of Inspection Report 50-220/96-05*

Gentlemen:

Inspection Report 50-220/96-05, which addressed a deficiency in the construction of Reactor and Turbine Building Blow-out panels at Nine Mile Poit Unit 1, also contained an Enclosure 2 with a number of technical questions regarding the engineering calculations and analyses related to this problem. Attachment 1 to this letter provides Niagara Mohawk's responses to those questions. Also included in the attachment is a description and summary results of the latest engineering analysis and responses to questions raised during the April 12, 1996 Enforcement Conference.

Very truly yours,

Carl D. Terry
Vice President - Nuclear Engineering

110053

CDT/AFZ/lmc
Attachment

xc: Regional Administrator, Region I
Mr. B. S. Norris, Senior Resident Inspector
Mr. D. S. Hood, Project Manager, NRR
Records Management

ATTACHMENT 1

NINE MILE POINT UNIT 1
REACTOR AND TURBINE BUILDING BLOW-OUT PANELS
NRC SPECIAL INSPECTION REPORT NO. 50-220/96-05, 50-410/96-05
RESPONSE TO TECHNICAL QUESTIONS

Introduction

Inspection Report 50-220/96-05 resulted from the NRC review of Niagara Mohawk's Licensee Event Report (LER) 96-05. The Inspection Report also contained an Enclosure 2 with a number of technical questions regarding the engineering calculations and analyses related to the deficiency in the construction of Reactor Building (RB) and Turbine Building (TB) Blowout panels at Nine Mile Point Unit 1 (NMP1).

As a result of discussions with the NRC's technical reviewers, Niagara Mohawk determined that a revision to the previous calculations was necessary in order to establish an upper-bound value for the currently installed configuration. A description and summary results of our most recent analyses is provided in this attachment, followed by responses to the original NRC questions.

Results of Current Evaluation (June 1996)

NMPC has recalculated the maximum blow-out pressures for the Reactor and Turbine Buildings. The results of this new calculation and the results of the superstructure minimum capability calculations are summarized below for purposes of reference in discussing the importance of various effects.

Values of Maximum Blow-out Pressure (psf)			Superstructure Minimum Capability (psf)
Building	Shear Bolts at 24" (*)	Shear Bolts at 12" (**)	
Reactor (RB)	65	128	143
Turbine (TB)	62	122	135

(*) Current configuration (1/4" diameter bolts @ 24" O.C.)
(**) Pre-existing condition (from initial construction until the 1995 Refueling Outage) (1/4" diameter bolts @ 12" O.C.)

The results in the above table are based on the following parameters:

1. shear bolt failure determines capacity;
2. 24" wide panels span horizontally
3. tensile capacity of bolts is equal to the highest value obtained from the RB and TB bolt samples (this is higher than the mean plus sigma from the 13 bolts tested)
4. shear capacity is 0.62 of the tensile capacity
5. elliptic shear tension interaction equation defines the failure
6. incorporated walkdown information on the gage of 3"x2"x ¼" angle
7. considered the effect of dead weight
8. incorporated the minor effect of large-displacements and boundary angle deformation on the bolt tension accompanying panel shear, based on finite element analysis using computer program ADINA.

In order to ensure that the results represent the upper-bound value for the blow-out panel function, Niagara Mohawk has employed conservative assumptions in the calculation, including:

- basing bolt shear strength on the highest tested tensile strength for a sample of bolts removed from the buildings; and

- using an elliptic shear tension interaction equation.

Furthermore, Niagara Mohawk has confirmed the suitability of the one-way span analysis with a specific evaluation of the effect of membrane action and a finite element analysis using the ADINA computer program.

Relative to the calculated blow-out pressures of 65 psf for the RB and 62 psf for the TB, NMPC has determined through a calculation that the effect of membrane action in the vertical direction is insignificant. The calculations indicate that the membrane could carry a pressure of 1.5 psf. However, this is not believed to be realistic in view of the fact that as the orthotropic plate bends under pressure, it is estimated that at a pressure of about 1.8 psf (starting from zero), the 16 gage plate at the joint will reach its fully plastic moment capacity. Pressure beyond this will tend to rotate the joint and open it up. Therefore, the 1.5 psf potential membrane contribution to the Blow-out pressure has not been included in the maximum Blow-out pressure values reported above. ***Based on these results, NMPC has determined that the panels will act principally on the horizontal span, and the effect of any two-way action is not significant.***

NMPC recently completed a finite element analysis of 24" FKX panel spanning horizontally. The ADINA computer program was used for this analysis. The large displacement option in ADINA was used to calculate the bolt tension as a function of applied pressure. In addition to the 24" wide panel, the analysis model also included the two steel angles which are used with the shear bolts to attach the panels to the superstructure. The results of this evaluation compared the ratio of the tension in the shear bolt as calculated by ADINA to the tension as it

would be calculated by simple analysis. The results of this analysis have been incorporated into the calculation of blow-out pressures that are listed above.

Considering the above, Niagara Mohawk is confident that the results presented in the above table represent the maximum internal pressure at which the blow-out function will occur. In addition, Niagara Mohawk has calculated the minimum internal pressure which could cause failure of the building superstructure and determined that, for both the current and original configurations, there is substantial margin between the blow-out pressure for the panels and the failure point of the superstructure.

Answers to Design/Licensing Basis Questions on NMP1 Reactor/Turbine Building Blowout Panels identified in Enclosure 2 of Inspection Report 50-220/96-05.

QUESTION 1: The LER discusses use of blow-out panels to protect the secondary containment structure. However, as discussed in "GE Design Specifications for Reactor Containment," blow-out panels are also sometimes used to protect primary containment from excessive reverse differential pressure that could occur in the event of a high energy line break in a compartment adjacent to the primary containment.

Is this generic design objective applicable to the NMP-1 facility and what is the documented basis if it is?

ANSWER 1: No. The postulated situation (i.e., a double-ended line break outside primary containment) is not a design basis event for NMP-1. There is no documented basis that the blow-out panels were designed to protect primary containment. However, the drywell was designed for an external pressure of 2 psig (288 psf), and the torus was designed for an external pressure of 1 psig (144 psf).

QUESTION 2: With respect to page 11 of Nine Mile Point 1 License Event Report (LER), titled, "Building Blow-out Panels Outside Design Basis Because of Construction Error," what are the key assumptions and engineering analysis/calculations performed in late October 1993 which led to the initial determination that the Turbine and Reactor Building panels would blow-out at 60 and 53 psf, respectively, to relieve internal pressure?

ANSWER 2: Calculations performed in late October 1993 that indicated the pressures of 60 psf and 53 psf are incorrect. These numbers were based on failure of the top and bottom connections of the panels as a result of sheet metal tearing. The results are based on the assumption of two-way distribution of load. The calculations performed in 1993 have been superseded. Further explanation of the assumptions used in the 1993 calculations is provided in Supplement 1 of LER 95-05.

QUESTION 3: Two design internal pressures of 40 and 45 psf are shown in different locations of the FSAR for Nine Mile Point 1 pressure relief panels (PRPs) for the Reactor and Turbine Buildings.

Please clarify the ambiguity about the two pressure values and indicate the correct licensing basis design pressure for the PRPs.

ANSWER 3: We are not sure why the ambiguity exists between 40 psf and 45 psf. The numbers appear in different sections of the FSAR and in some cases with the adjective "approximately." Niagara Mohawk believes that the ambiguity has existed from the original preparation of the FSAR in the 1960s, however, there are no design documents available to explain these numbers. A description of Niagara Mohawk's actions to resolve the discrepancy is provided in Supplement 1 of LER 95-05.

QUESTION 4: Has an assessment been made on the error in the design assumptions for load distribution which was identified during the March 1995 refueling outage?

Also, explain what assumptions for load distribution were used in conjunction with the consideration of the 1/4" bolts with higher ultimate strength (78 ksi) which led to the determination of the revised panel blow-out pressures of 92 and 88 psf for the Reactor and Turbine Buildings, respectively (pages 11 and 13 of DER NO. 1-93-2526).

ANSWER 4: The error was a result of a human performance deficiency. A formal root cause analysis was completed. The cause of the error was a failure on the part of structural engineers (preparer, checker, and approver of calculations) to clearly state in the conclusion statement of the calculation which span analysis (one-way or two-way) caused the failure mechanism of the blow-out panels and at which specific internal pressure load the blow-out panels fail. As a result the two-way span analysis results were incorrectly used in preparing the Operability Determination in 1993. This Operability Determination was used to justify the original panel configuration.

In the 1995 calculation, load distribution was considered to act one-way in horizontal span because of the manufacturer's recommendation and the panel stiffness in the horizontal direction. This calculation resulted in blow-out pressure values of 92 and 88 psf. See the answers to Question 5 for additional discussion of assumptions used in the 1995 calculations. Current calculations quantify the fact that the panels do not have any significant capability to resist load in the vertical span and that the panels span one-way (horizontal direction) only. Refer to results of current evaluation in the introduction.

QUESTION 5: With respect to the above referenced DER, provide a detailed discussion of the key assumptions, panel/bolt configurations (including pertinent drawings), and bolt ultimate strength used in concluding that the revised panels would blow-out at about 45 psf with the use of the 1/4" diameter bolts and the removal of every other bolt from the existing panels.

ANSWER 5: The 1995 modification objective was to correct the construction error by changing the bolt configuration, such that the panel blow-out pressure would be equivalent to that specified on the original construction drawing. The construction drawing specified 3/16" bolts on 12" centers. When the increase in ultimate bolt tensile strength and increase in diameter were considered, the panels had approximately twice the capacity of 3/16" bolts on 12" centers. Therefore, by removing every other bolt, the panel/bolt configuration would match that specified on the construction drawing. While there are no available design documents (calculations) to explain the original design, Niagara Mohawk has concluded, based on the results of recent analyses, that the same design assumptions and methods were used in 1995 as for the original design. These methods consist of the following key criteria: (1) Use of mean ultimate tensile bolt strengths, (2) Use of ultimate shear stress as 60% of mean ultimate tensile strength, (3) based on the design and manufacturer's recommendations, consider the FKX panels as one-way, simply-supported beams, and (4) Use of unity shear-tension interaction at failure. The 1995 calculation has been revised. Refer to results of current evaluation in the introduction.

QUESTION 6: The Reactor Building Blow-out panel revised calculations are based on certain assumptions, the conservatism of which cannot be determined unless compared to a more rigorous analysis or test.

It is not clear how NMPC demonstrated conformance with FSAR commitments by re-evaluating the panel pressure capacity using more exact methodology, or revising the analysis and stating clearly the conservatism of each assumption used in the analysis.

The staff has identified the following effects which appear to have not been considered in the analysis or are not clearly stated:

QUESTION 6a: The calculation of the pressure capacity of the top and bottom connections do not consider the membrane effect of the panel in the longitudinal (vertical) direction and the effect of the flexibility of the side connections between the flutes and the columns. In addition, the effect of friction between the bolts and the sheet metal surfaces, due to bolt pre-loading, has not been considered.

ANSWER 6a: Membrane effects of the panel in the vertical direction were not considered for the following three reasons:

1) The panels consist of 24" wide, H. H. Robertson, FKX 16/16 metal decking, which runs horizontally from building column to column. The panels are of ASTM A245 steel and are 16 gauge. Each plank is fastened to the adjacent planks with side joints crimped together at 18" spacing. The crimped side joints are not considered capable of accommodating any significant membrane or shear force transfer from plank to plank. Transfer of an in-plane membrane force or shear force between panels would depend on the load carrying capacity of the caulking and the interlocking of the crimped spot located on 18" centers. The load carrying capacity of these elements is considered insignificant. This assumption has been substantiated by an Engineering calculation.

2) Each Reactor Building panel bay is approximately 19'-2" side by 44'-1½" high. Assuming that the panel is an isotropic flat plate, simple-supported or fixed-end around the edges, then from geometry for a rectangular plate where a = 44.125' and b = 19.167' the a/b ratio is 2.30. With this aspect ratio, the majority of the applied pressure load is transferred across the short dimension. The distribution of edge reactions is approximate parabolic, with the highest shear loads in the center of the long span. These center bolts would experience shear loads approximately equal to the same load if vertical load transfer were neglected. In other words, the supports along the short sides (top and bottom) have little affect on the action of the plate due to the high aspect ratio, and the plate acts approximately as a simple beam having a span of 19.167'. For the Turbine Building, there are no connections on top. As shown by calculations and with the absence of top connections, the panels span one-way only.

3) The FKX 16/16 panels do not have the same stiffness in the vertical and horizontal directions. The FKX panels are designed as simple beams spanning from column line to column line, using allowable spans based on allowable bending stress of 20 ksi, per the design procedure of the H. H. Robertson catalog. With the flutes running horizontal, the panels have a section modulus of 2.49 in.^{3}/ft. in width. However, in the vertical direction, only one layer of 16 gauge steel is available to resist bending, for a section modulus of about 0.000596 in.^{3}/ft. ($S = T^2/_6 = 0.0598^2/6$).

Furthermore, to demonstrate that the effect of membrane action in the vertical direction is insignificant, Niagara Mohawk conducted the following analysis. A load of 65 psf was applied on a horizontally spanning 20 ft. panel, and its center deflection was calculated as 1.06". If this center deflection is imposed on a vertically spanning membrane of 20 ft. length, the membrane tension is 1685 lbs. for a 24" width. At this amount of tension, the membrane can carry a pressure of 1.5 psf. This assumes that the horizontal J-joints between panels can transfer a membrane tension of

1685 lbs. However, this is not realistic since, as the orthotropic plate bends under pressure, the 16 gage panel at the joint will reach its fully plastic moment capacity at a pressure of about 1.8 psf (starting from zero). Pressures beyond this will tend to rotate the joint and open it up. By the time the load of 65 psf is reached, the J-joint will lose its capacity to transmit axial tension, therefore, the 1.5 psf potential membrane contribution to the blow-out pressure has not been included in the current calculation of maximum blow-out pressure values as described in the introduction.

Regarding the effect of flexibility of the side connection, Niagara Mohawk completed a finite element analysis of a 24" FKX panel spanning horizontally using the ADINA computer code. The panel was represented by plate elements. The mesh size was fine enough to model the 3/8" bolts holding the panels to the angle irons. The 1/4" shear bolt was also included in the model. The isometric view of the model that was analyzed is attached. (Figure 1)

The large displacement option in ADINA was used to calculate the bolt tension as a function of applied pressure. The result of this evaluation compared the ratio of the tension in the shear bolt as calculated by ADINA to the tension as it would be calculated by simple analysis (from the moment equilibrium of the 3"x2"x1/4" angle for a given bearing force on the angle leg resulting from the pressure load). The resulting ratio depends on the value of applied pressure. At 60 psf, tension ratio of the ADINA to the simple method is 1.02; at 120 psf this ratio is 1.06. This analysis demonstrates that the effects of the side connections are insignificant, however, they have been incorporated, through the use of the ADINA program, into the results presented in the introduction.

The effect of friction between bolts and sheet metal surfaces is not considered significant for the following reasons. The ¼" shear bolts are A307 bolts. Their installation did not have a controlled pre-load. Moreover, it is known that for high strength bolts which have higher pre-load than A307, the pre-load does not affect the bolts shear strength. (See Pages 48 and 49 of Reference 1). Therefore, the effect of friction on shear strength of the A307 bolts of concern is not significant.

QUESTION 6b: The calculation of the pressure capacity of the side connections are based on the assumption of a simply supported beam. This is not valid if the panel is rigidly bolted to the angle members, in which case the analysis should be based on a beam with elastically built-in ends.

ANSWER 6b: FKX panels are designed as simply supported beams per the manufacturer's recommendations and load tables. To design them otherwise would be non-

conservative when considering wind loads. The results of the analysis are not affected by this choice, since the same shear load would be applied to the shear bolts for either case. The panels cannot be considered to be rigidly bolted to the attachment angles, since the bolts are through one sheet metal layer only. The low bending stiffness of the 16 gauge sheet metal under the head of one bolt is not sufficient to carry the moment capacity of the entire flute section. This local flexibility would allow the ends of the FKX panels to rotate, thereby precluding a fixed moment capacity. Even if the panel was rigidly connected, the end moments would increase the prying tensile load on the shear bolts, reducing the pressure loading at which blow-out would occur.

In addition, please refer to the discussion of modeling with the ADINA Program in the answer to question 6a. That study showed that including the flexibility of the boundary angles had very little effect on the calculated pressure capacity. Also, note that the observed small effect has been incorporated in the blow-out pressures that are reported in the results of the current evaluation.

QUESTION 6c: The calculation of the pressure capacity of the side connections was determined from the analysis of a typical flute, without considering the in-plane membrane forces acting on the flutes.

ANSWER 6c: Please refer to our quantification of the effect of membrane action in the vertical direction, discussed in Response to Question 6a.

QUESTION 6d: The calculation of the sheet metal shear capacity is based on one shear area. It should be based on two shear areas.

ANSWER 6d.: NMPC concurs with this observation related to the 1993 calculation. However, shear tearing of metal is not in the load path to failure. For this reason, NMPC has recalculated the blow-out pressures based on the failure of the ¼" shear bolts.

QUESTION 6e: The effect of the panel deadweight on the connections has not been considered.

ANSWER 6e: The shear due to deadweight needs to be vectorially combined with the shear due to pressure. The results of the current evaluation considered this dead load effect. The effect of deadweight causes increase on the shear load, thereby lowering blow-out pressure by approximately 1.0 psf.

QUESTION 6f: What are results of these effects on the calculation of the panel pressure capacities, as demonstrated by detailed calculations, or by the conservatism of the existing calculations?

ANSWER 6f: As discussed under Responses to Question 6a through 6e above, NMPC has quantified and assessed results of these effects. NMPC has performed either conceptual models with quantitative results to indicate the impact of the effects, or has performed refined analysis to show the effects, and included these in our final result.

QUESTION 6g: Also, a weakness of the 1995 modification evaluation for blow-out panel interim corrective actions appears to be the implied assumption of imminence of failure of the ¼" diameter shear bolt with a computed "unity" value for the "shear-tension interaction" equation. What is the pertinent analytical or test-supported basis for such an assumption?

ANSWER 6g: The 1995 calculation use of a unity shear-tension interaction equation was incorrect. NMPC agrees with the use of an elliptical equation as being the appropriate equation to determine the failure of bolts. Our current calculations are based on an elliptical shear-tension interaction equation. Use of the unity shear-tension interaction equation is conservative for designing building siding to maintain Reactor Building leak integrity during high wind conditions.

QUESTION 7: (Does not exist).

QUESTION 8: With respect to Nine Mile Point 1 Calculation No. S7-RX340-W01, revision 0 and 1, in support of bolt strength:

QUESTION 8a: What is the basis for selecting the revised bolt ultimate tensile strength of 78 ksi in the latest panel blow-out capacity calculation?

ANSWER 8a: Refer to the attached Table 1, bolt test results. The mean value of RB bolt tests is 2476 lbs; the mean value of TB bolt tests is 2524 lbs. With tensile area of 0.0318 in^2 for ¼" diameter bolts, these yield mean tensile stress at failure of 77.9 ksi and 79.4 ksi, respectively. The value of 78 ksi being questioned is based on the above mean stress values at failure. This was used in our 1995 calculations.

It is noted that for blow-out pressures reported in the results of the current evaluation, the bolt tensile strength used is 2685 lbs., which is the maximum observed value of the combined sample from the TB and RB. There is no specific reason for treating these samples separately. In our view, they belong to the same population.

Note in Table 1 that the mean + 1 standard deviation value of the combined sample is 2647 lbs. This is slightly less than the observed maximum. The observed maximum sample strength is used for blow-out pressures in the results of the current evaluation. This is conservative.

QUESTION 8b: What is the test-verified ultimate shear strength of the same set of bolts tested?

ANSWER 8b: Shear tests were not performed. Adequate information on the relationship between shear and tensile strength for bolts exists. The attached Figure 2 (Page 47 from Reference 1) shows that the ratio between shear strength to tensile strength does not depend on the bolt grade.

The attached Figure 3 (Page 48 from Reference 1) shows the distribution of shear strength to tensile strength. The mean value of this ratio is 0.62 and its standard deviation is 0.03.

For the blow-out pressures in the results of the current evaluation, this mean factor 0.62 was used to obtain the shear strength from the maximum observed tensile strength.

The reason for not using mean + 1 standard deviation here is to avoid compounding conservatisms; when a function of two or more variables is being evaluated to obtain a reasonably high single value of the function, it is the practice to evaluate the function when all variables except one of them are at their mean values, and the single remaining variable is chosen at a more conservative value. In the present case, we have used a high value for tensile strength. Therefore, it is reasonable to use the mean shear strength-to-tensile strength ratio to determine a reasonably conservative shear strength value.

QUESTION 8c: Are the strengths (both the ultimate tensile and shear strengths) based on ultimate strength tests of an adequate sample size of the ¼" bolts?

ANSWER 8c: For discussion of samples and our selecting a value greater than the mean + 1 standard deviation to quantity panel blow-out pressure, refer to Answer 8a.

QUESTION 8d: Is the 78 ksi a mean ultimate tensile strength?

ANSWER 8d: Yes. This value was used in our 1995 calculations to determine failure load of bolts. However, please refer to Answer 8a and discussion of the statistics of the sample tests.. The 78 ksi is nearly the mean ultimate tensile strength, but this value is not used in blow-out pressures for the results of the current evaluation. A value higher than 78, i.e., 84.4 ksi, is used in our current (1996) calculations.

QUESTION 8e: What is the corresponding mean ultimate shear strength used in the assessment?

ANSWER 8e: Please refer to Answer 8b. Mean ultimate shear strength is not calculated. A conservative higher value than the mean is used. This means shear strength-to-tensile strength ratio times the highest observed tensile strength, i.e., 0.62 x 2685 = 1665 lbs., is used as shear strength.

QUESTION 8f: If they represent mean ultimate strengths, what are their corresponding standard deviations?

ANSWER 8f: For discussion of standard deviation of tensile strength, see Answer 8a. For shear, test results were not considered necessary. This is discussed in Answer 8b.

QUESTION 8g: If the values represent nominal lower-bound strengths and they were used in your latest calculation which confirmed the revised blow-out capacity of approximately 45 psf, how can one be sure that the panels would blow-out approximately at 45 psf and not at a higher value?

ANSWER 8g: NMPC has recalculated the failure load of bolts using higher-bound strengths. In our current calculations (1996), the nominal lower-bound strength values were <u>not</u> used to determine the blow-out pressure values in the results of the current evaluation. The calculated pressures are higher than 45 psf. Refer to results of current evaluation in the introduction.

QUESTION 8h: Given the above-mentioned uncertainties in ultimate tensile and shear strengths and load distribution assumptions used in the analysis, has NMPC established a conservatively determined upper-bound panel blow-out pressure and demonstrated that the computed pressure capacity is lower than the 45 psf pressure stipulated in the licensing basis document?

ANSWER 8h: As discussed in the introduction, NMPC has determined the upper-bound blow-out pressures. The conservatively established blow-out pressures are provided in the results of the current evaluation. The safety implications of these results are discussed in Supplement 1 of LER 95-05.

QUESTION 9a: With respect to the same calculations noted above, discuss the appropriateness of using conservative engineering assumptions, including conservative modeling (i.e., one-way horizontal action for Robertson's panels), and use of mean of non-upper-bound ultimate bolt tensile and shear capacities to determine a realistic upper-bound internal pressure which will cause failure of the PRPs.

Such an approach could underestimate the real panel blow-out pressure, thus, resulting in a non-conservative conclusion. Specifically, use of a lower-bound or mean bolt ultimate tensile strength and an assumed ultimate bolt shear strength of $0.6F_{\mu}$ (instead of test verified shear strength) would lead to an unrealistic PRP failure pressure and potentially unsafe conclusion.

ANSWER 9a: Revised upper-bound panel blow-out pressures have been calculated and provided in the introduction. As discussed in the introduction and in the response to several previous questions, Niagara Mohawk has used conservative assumptions to assure that the results represent the maximum internal pressure at which the panels will blow-out. Specifically, the bolt shear strength used in the current analysis is based on the maximum tested tensile strength from a sample of bolts removed from the panels. Niagara Mohawk has also established explicit justification for the use of a one-way span analysis, the exclusion of membrane effects, and the use of the elliptic shear tension interaction equation.

QUESTION 9b: Discuss the safety implications of such a practice in light of the objective of the PRP evaluation and the need to modify the evaluation and demonstrate that the physical panel disposition proposed is still valid.

ANSWER 9b: As discussed above, Niagara Mohawk has revised its analysis of the blow out panels considering the staff's comments provided in these questions. The blow-out pressure values have been recalculated, with appropriate justification for design assumptions, to ensure the results represent the maximum blow-out pressure at which the panels will fail.

Responses to questions raised during Enforcement Conference on April 12, 1996.

QUESTION 1: What is the significance of the Reactor Building blow-out panel landing on the roof of the Condensate Storage Tank Building?

ANSWER 1: The building roof over the condensate storage tanks was not originally designed for the impact load of the blow-out panels, however, Niagara Mohawk has performed calculations that indicate that in the worst case the panels may penetrate the roof covering, but would not cause a catastrophic collapse of the structure or otherwise damage the tanks. Hence, the condensate storage tanks remain operable.

QUESTION 2: What is the effect of caulking on the blow-out panel function?

ANSWER 2: The FKX panels are caulked in two locations. First, the J-joint between panels was caulked by applying a continuous beak of caulking to the J-section of the connection. Second, a continuous 1/2' bead of caulking was applied between the FKX panels and the building columns. This caulking was designated as "RP-545 Sealant." Since the original construction, additional silicon sealant has been applied to various locations to maintain leak tightness. Because the sealant has a negligible bonding strength compared to the forces involved and relative to the strength of the shear bolts, the effect of the sealants on the blow-out panel function was considered to be insignificant.

REFERENCES

1. 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.

TABLE 1

BOLT TEST RESULTS

A. Reactor Building

Sample size = 4 (original sample 6, 2 damaged, not tested)

Sample #	Tensile Failure Load, lbs.
1	2525
2	2470
3	2390
4	2518

Mean Value = 2476 Standard deviation = 62.17

B. Turbine Building

Sample size = 9 (original sample 10, 1 bolt severely rusted, conservatively not counted)

Sample #	Tensile Failure Load, lbs.
1	2475
2	2685
3	2610
4	2612
5	2500
6	2555
7	2562
8	2130
9	2595

Mean value = 2525 Standard deviation = 160.75

C. For Combined Sample (4 + 9 = 13 data points)

Mean = 2510 lbs. s = 137 lbs.

Mean + s = 2647 lbs.

Maximum of data = 2685 lbs. (sample #2 in TB)

FIGURE 1

NIAGARA MOHAWK NUCLEAR ENGINEERING

CALCULATION CONTINUATION SHEET

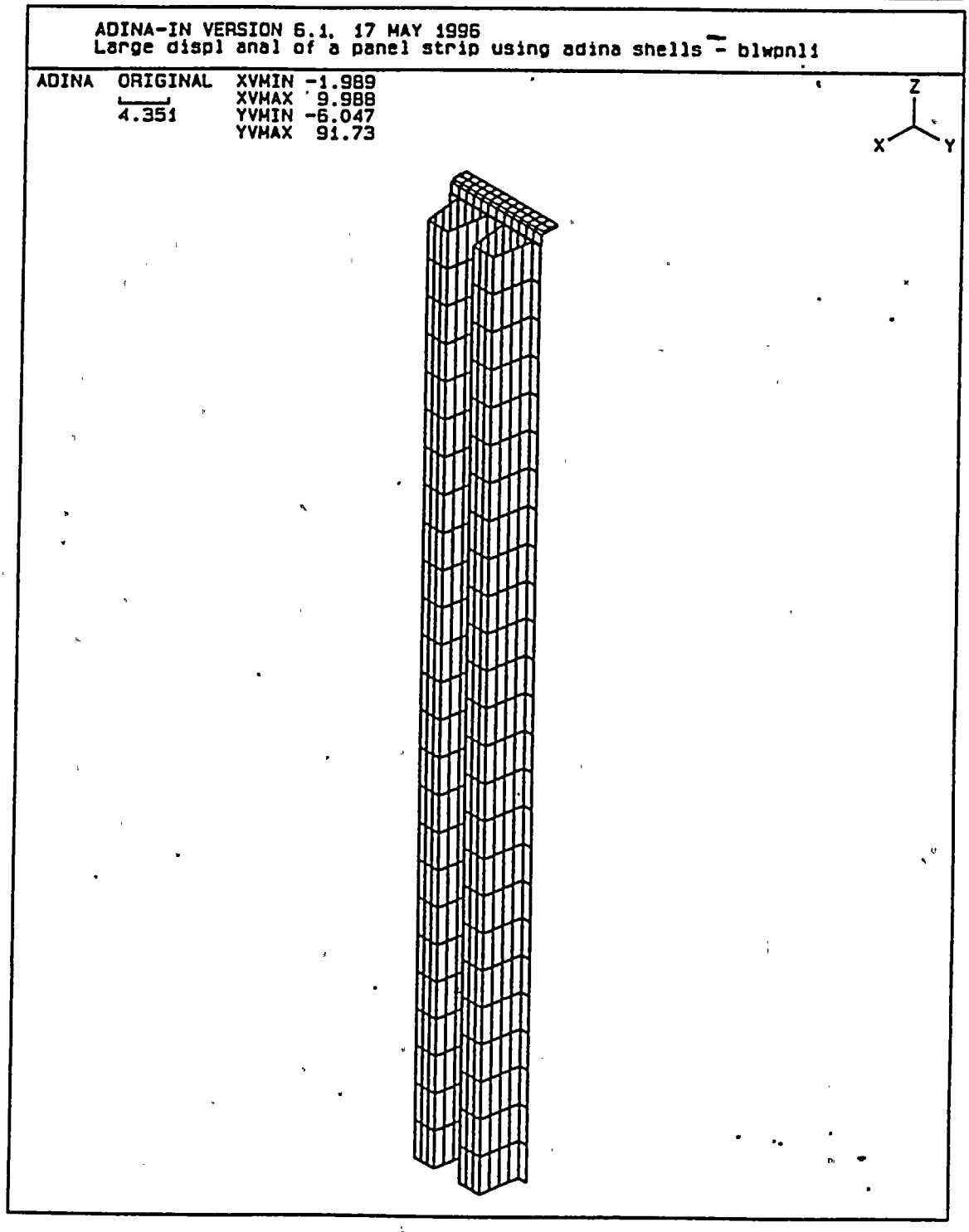
Page 42 (Next ___)

Project: *NINE MILE POINT NUCLEAR STATION* UNIT: 1 DISPOSITION ___

Originator/Date	Checker/Date	Calculation. No.	Rev
JPJ 5-20-96		S7-RX340-W01	0

Ref.

A.2



ISOMETRIC VIEW OF THE MODEL

(NEP-DES-08-F02-00)

FIGURE 2

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.

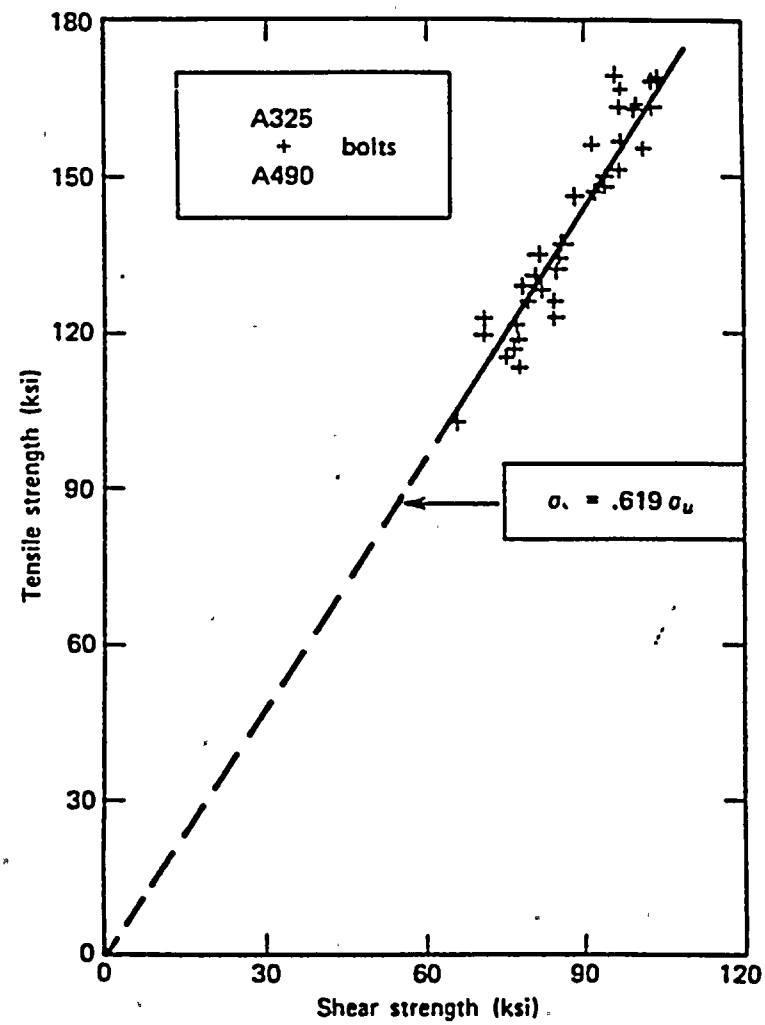


Fig. 4.13. Shear strength versus tensile strength. *Note*. Each point represents the average values of a specific bolt lot. The shear strength is computed on the relevant area, depending on the location of the shear plane.

FIGURE 3

A.4

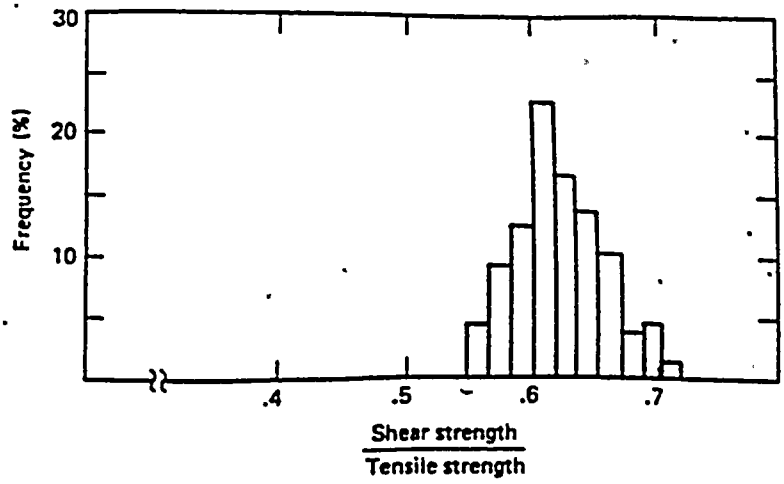


Fig. 4.14. Frequency distribution of ratio of shear strength to tensile strength for A325 and A490 bolts. Number of tests, 142; average value, 0.62; standard deviation, 0.03.

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