Telephone (617) 872-8100
TWX 710-380-7619

YANKEE ATOMIC ELECTRIC COMPANY

YANKEE

1671 Worcester Road, Framingham, Massachusetts 01701 DCC 86-134
2.C2.1

September 5, 1986
FYR 86-084

United States Nuclear Regulatory Commission
Washington, DC 20555

Attention: Ms. Eileen McKenna
PWR Project Directorate No. 1
Division of PWR Licensing - A

References: (a) License No. DPR-3 (Docket No. 50-29)
(b) Letter, USNRC to YAEC, dated March 14, 1986
(c) Letter, USNRC to YAEC, dated April 17, 1986
(d) Letter, USNRC to YAEC, dated June 6, 1986
(e) Letter, YAEC to USNRC, dated July 9, 1984

Subject: Response to Requests for Additional Information, SEP Topics III-2 and III-4.A

Dear Ms. McKenna:

Attached please find Yankee Atomic Electric Company's (YAEC) responses to the NRC's requests for additional information as presented in References (b), (c), and (d).

Enclosure 1 of this letter is the response to Enclosure 2 of Reference (b). Enclosure 2 of this letter is the response to Reference (c). Enclosure 3 of this letter is the response to Enclosure 3 of Reference (d). Enclosure 4 of this letter is the response to Enclosure (4) of Reference (d).

Questions 3, 5, 6a, and 6d of Enclosure 1; Question 11 of Enclosure 2; and Questions 2 through 4, and 6 through 9 of Enclosure 3 were considered resolved at the conclusion of the May YAEC-NRC meeting in Framingham.

All other responses incorporate the comments and additional information requested by the NRC during the May meeting and subsequent telephone conversations.

As a result of the evaluations performed for Topic III-2 YAEC will perform the following work:

1. Exposed main steam and feedwater piping will be evaluated for a 178 mph tornado. Any upgrades required by this evaluation will be integrated with the seismic upgrades for this piping.

A001
1/1

2. The west wall of the upper level Primary Auxiliary Building (PAB) between column Lines F_b and E_c will be evaluated for a 134 mph wind/121 mph tornado, and upgraded if required.

3. The connection of the upper level PAB metal roof deck to the supporting steel between column Lines 6 and 8 will be evaluated. If necessary, the connections will be upgraded to the capacity of the roof deck.

4. The Cable Spreading Room will be upgraded as stated in Reference (e).

We believe the attached enclosures provide sufficient information for the resolution of SEP Topics III-2 and III-4.A. As stated in Reference (e), the results of our evaluations establish the 10^{-4} upper 95% windspeed (85 mph tornado/110 mph straight wind) as the design basis event for YNPS. Further, our evaluations conclude that for the 10^{-4} or 10^{-5} (165 mph tornado) wind event, tornado-generated missiles are not an issue.

YAEC's commitment to make plant modifications is contingent upon acceptance of the enclosed material. It is our understanding that the staff will issue an SER for SEP Topics III-2 and III-4.A, along with a summary resolution of the topics in a future supplement to NUREG-0825. If this is not the case, please advise us.

Very truly yours,

YANKEE ATOMIC ELECTRIC COMPANY

George Papanic Jr.

G. Papanic, Jr.
Senior Project Engineer
Licensing

GP/hja

Attachments

ENCLOSURE 1

Response to Enclosure 2 of Letter, J. W. Clifford (NRC) to G. Papanic, Jr. (YAEC), dated March 14, 1986 (NYR 86-055)

QUESTION 1

No values for wind speed capacities corresponding to the maximum APC have been provided; all wind speed capacities were based on dynamic wind pressure. No engineering calculations demonstrating that sufficient venting would occur have been provided. Provide a discussion of this subject.

RESPONSE

For purpose of the tornado cost-benefit analysis, the ultimate lateral pressure load (or failure load) on critical structures and components was calculated. Analyses of these structures included previously designed and/or conceptualized modifications for seismic and design wind events. Failure evaluation criteria is presented in the Cost/Benefit Evaluation, Section 4.3 (Reference 1-1).

The conversion of ultimate lateral pressure loads to straight wind and tornado wind speeds is presented in the Cost/Benefit Evaluation, Section 4.2 (Reference 1-1). For reasons stated therein, the atmospheric pressure drop loading was assumed as not producing the controlling load for structures and components in the cost-benefit analysis.

QUESTION 2

The chimney is a steel stack, 5'-0" in diameter and 1/4" thick. It was identified in NRC's IPSAR (NUREG-0825) as a concern, but was not addressed in the licensee's submittals. Provide schedules to submit the results of the chimney's analysis.

RESPONSE

The primary vent stack is approximately 130 feet tall and is supported at two elevations. The base of the stack is restrained vertically and laterally at Elevation 1056'-2". The support at Elevation 1131'-0" provides lateral restraint. An elevation view of the stack is shown in Figure 1-3. The support at Elevation 1056'-2" is shown in Figure 1-4. Figure 1-5 presents the support at Elevation 1131'-0".

The vent stack and structural steel are A7 steel (F_y = 33 ksi). Bolted connections are made using A325 bolts. Embedded anchor bolts are A307 steel. Concrete expansion anchors ("cinch bolts") are Star Slug-Ins.

The vent stack and supports were evaluated for dead load plus 165 mph (10^{-5}) wind/tornado loadings. Wind/tornado loadings were applied in both the X and Y directions. Thermal loads were not considered since the vent stack is at ambient temperature during normal operation.

The dynamic effects of the tornado wind (vortex shedding, flexural vibration) were not considered for the following reasons. Periodic vortex shedding is not expected to occur at velocities over 60 mph. The projections (vent piping, ladders, supports) present on the vent stack would prevent vortex shedding. Further, the location of the vent stack between the PAB and vapor container would also reduce the possibility of periodic vortex shedding.

The ANSYS computer code was used for the vent stack evaluation. The analytical model is shown in Figures 1-6 and 1-7.

Allowable stresses for structural steel, including the vent stack, were obtained from the AISC Manual, 8th Edition. A one-third increase in allowable stresses per 1.5.6 of AISC was used.

Embedded anchor bolts were evaluated in accordance with Appendix B, Steel Embedments, of ACI 349-76.

The results of the evaluation are presented in the table below. All stress ratios and interaction coefficients are less than 1.0. Therefore, the primary vent stack will remain within design code allowable limits when subjected to a 165 mph (10^{-5}) tornado.

		Stress Ratio/Interaction Coefficient
Node Number	Element	Dead Load + 165 mph
54	Vent Stack	0.18
21	W21 X 62	0.11
13	W18 X 50	0.66
5	W16 X 36	0.20
9	W8 X 24	0.41
2	L3-1/2 X 3-1/2	0.59
54	6" Diameter Pipe Brace	0.27
33	7/8" Diameter Bolt-Vent Stack to Support Beams	0.64
17	1" Diameter Anchor Bolts	0.32

QUESTION 3

Provide a discussion regarding foundation and soil capacities. Confirm whether they are limiting in the evaluation of structures for wind and tornado loadings. This was not addressed in the licensee's submittals [1, 2].

RESPONSE

For the tornado cost-benefit analysis, the foundation and soil capacities were not specifically addressed when determining the ultimate lateral pressure loadings. However, it can reasonably be stated that the tornado wind loads will not increase the original foundation loads by greater than a 1.6 factor (i.e., the permitted stress increase in the structural system delivering the load). Since foundation design criteria typically have safety factors greater than 2.0 for bearing capacity, 1.5 for overturning and sliding, and 1.25 for uplift, the existing foundations do not limit tornado wind loadings. The safety factors for overturning, sliding, and uplift are based on a 1/3 stress increase permitted during the original design for wind loading.

QUESTION 4

Explain how roof decks were analyzed, i.e., possible modes of failure, what type of load, and also provide wind speed capacities for all roof decks.

RESPONSE

The steel roof decks were assumed to fail in bending at 1.6 times the elastic limit stress allowable. The net uplift pressure calculated is the uniform load capacity of the roof deck (based on assumed simple span moment and allowable of 1.6 S) minus the nominal roof deadweight. The corresponding wind/tornado velocities were then calculated based on the ultimate capacity.

Minimum ultimate wind speed capacities for the roofs are summarized below:

	Straight Wind	Tornado
PAB	122 mph	126 mph
DGB	160 mph	167 mph
Upper Pipe Chase	>165 mph*	>165 mph*
Cable Spreading Room	>165 mph*	>165 mph*

---

*Roofs of Upper Pipe Chase and Cable Spreading Room will be modified to be within design allowables when subjected to a 10^{-5} tornado event.

No equipment located in the DGB is credited after the failure of Wall D1X1 at 134 mph for straight wind/121 mph for tornado. Therefore, since the failure of the DGB roof occurs at higher speeds, the roof failure will have no effect on the Cost/Benefit Evaluation.

Equipment/piping located in the upper level PAB which is credited in the evaluation is the dedicated Safe Shutdown System piping, the blowdown header, safety injection piping, and EFW piping.

The blowdown header is located directly below the Upper Pipe Chase (UPC), against the PAB north wall (Figure 1-1). The roof deck could not impact the blowdown header.

The SSS piping is located against the Valve Room wall and is directly below an elevated walkway and the UPC (Figure 1-1). The SSS piping could not be impacted by the roof deck.

The Safety Injection piping and Emergency Feedwater (EFW) piping are located approximately 7 feet south of the PAB north wall and run parallel to the wall. This piping is shielded from the roof deck by the structural steel roof framing as well as conduit running beneath the roof framing.

The failure wind speed (122 mph) noted above is for failure of the roof at corners. Failure of the main roof would not occur until 207 mph. Also, the roof would not fail downwards, but would "peel off." Therefore, failure of the PAB roof will not affect equipment credited in the Cost/Benefit Evaluation.

QUESTION 5

Explain how loads were combined in the analysis including snow, piping, thermal, and attachment loads.

RESPONSE

With the exception of the Non-Return Valve (NRV) structure, there are no significant piping or thermal loads on the structures evaluated. For the tornado wind evaluation, the snow loading is not a consideration, and, therefore, was neglected. In addition, wall attachment loads are considered minimal, and, therefore, excluded for purposes of determining ultimate lateral pressure loads.

The NRV structure's original design accounted for the significant piping forces. The ultimate lateral pressure load of 26.5 psf calculated for this structure was conservatively derived by multiplying the average design wind load of 20.7 psf by a factor of (1.70/1.33).

QUESTION 6

Regarding the masonry block walls:

a. Indicate whether shear stress was considered in the analysis.

b. Indicate whether connections at the boundary were checked for their structural adequacy. Provide the method and results of the analysis.

c. Provide the technical basis for crack simulation in the reinforced and unreinforced walls, respectively. It is noted that once cracks are developed, it is questionable that the wall could carry any loads. Provide a sample calculation regarding this subject.

d. Indicate whether there are multi-wythe walls at the plant. If so, provide the analytical method used to qualify them. Justify allowable stresses used (if any) for the collar joint.

<u>RESPONSE</u>

a. Shear stresses (flexural and in-plane) were considered in accordance with the allowables established in Attachments 11.0-1, <u>Allowable Stresses in Unreinforced Masonry (Existing)</u>, and Attachment 11.0-2, <u>Allowable Stresses in Reinforced Masonry (Existing)</u> to MAIN Document No. DCD-2648-6-1 (Reference 1-2), dated February 8, 1984. Analysis methods and results are contained in applicable calculation sets which were available for NRC audit during the May meeting.

b. Boundary conditions are selected to be representative of actual conditions to assure validity of the boundary assumptions in accordance with Section 5.0 of Document No. DCD-2648-6-1 (Reference 1-2). Methods used to satisfy validity of boundary assumptions and data substantiating these methods (structural steel angles in combination with toggle bolts, masonry sleeve anchors, and/or expansion bolts) are contained in the applicable calculation sets which were available for NRC audit during the May meeting.

c. For existing unreinforced masonry, the allowable mortar tensile stress is 14 psi normal to the bed joint and 41 psi parallel to the bed joint. Because of the substantially greater mortar tensile stress allowable in the horizontal direction, structural steel spanning vertically is generally provided to reinforce the masonry walls. When cracking occurs in the horizontal direction (i.e., the mortar tensile stress of 14 psi normal to the bed joint is exceeded resulting in zero moment capacity), the masonry spans horizontally between the vertical structural steel supports. In effect, the structural steel reinforcement is designed to carry the lateral load into the supporting structure. For either existing or new reinforced masonry, the analysis and design were performed in accordance with the requirements of ACI 531-79, "Building Code Requirements for Concrete Masonry Structures" (Revised 1981), using the allowable stress criteria established in MAIN Document No. DCD-2648-6-1 (Reference 1-2).

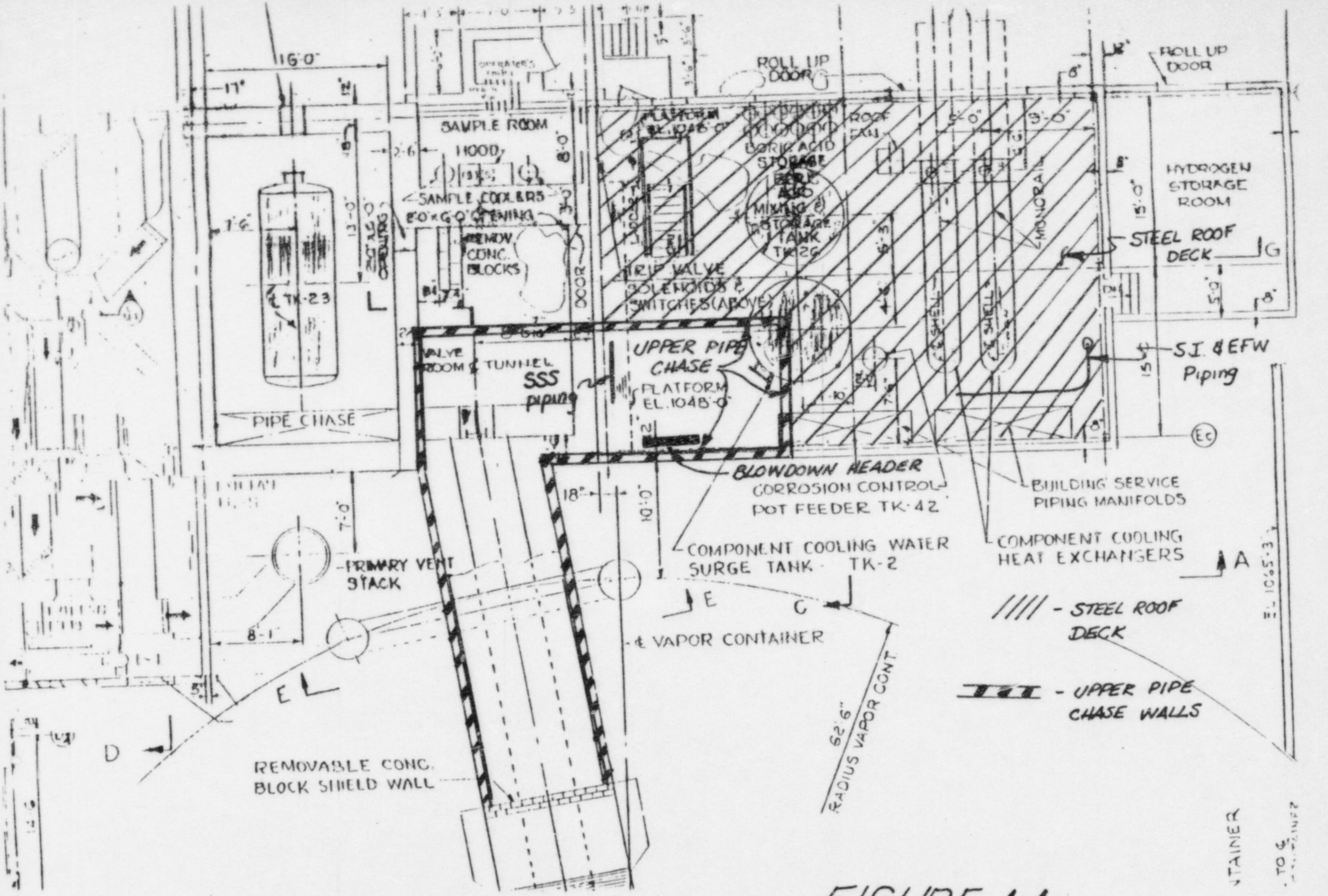
Calculations for the analysis and design of both unreinforced and reinforced masonry walls are contained in applicable calculation sets which were available for NRC audit.

d. No multi-wythe walls are present in the walls evaluated for the cost-benefit analysis.

REFERENCES

1-1 Letter, J. A. Kay (YAEC) to J. Zwolinski (NRC) (FYR 85-01), SEP Topics III-2 and III-4.A, dated December 31, 1984.

1-2 Charles T. Main, Inc., Document No. DCD-2646-6-1, Revision 0, Structural Design Criteria for Evaluation and Modification of Existing Masonry Block Walls, dated February 8, 1984.



16'-0"
SAMPLE ROOM
HOOD
SAMPLE COOLERS
TK-23
PIPE CHASE
ROLL UP DOOR
PLATFORM EL.1048'-0"
BORIC ACID STORAGE
BORIC ACID MIXING & STORAGE TANK
TRIP VALVE SOLENOIDS & SWITCHES (ABOVE)
REMOV. CONC. BLOCKS
DOOR
ROOF FAN
MONORAIL
HYDROGEN STORAGE ROOM
STEEL ROOF DECK
S.I. & EFW Piping
VALVE ROOM & TUNNEL
SSS piping
UPPER PIPE CHASE
PLATFORM EL.1048'-0"
BLOWDOWN HEADER
CORROSION CONTROL POT FEEDER TK-42
BUILDING SERVICE PIPING MANIFOLDS
COMPONENT COOLING WATER SURGE TANK TK-2
COMPONENT COOLING HEAT EXCHANGERS
PRIMARY VENT STACK
℄ VAPOR CONTAINER
62'-6" RADIUS VAPOR CONT.
REMOVABLE CONC. BLOCK SHIELD WALL
//// - STEEL ROOF DECK
- UPPER PIPE CHASE WALLS

FIGURE 1-1

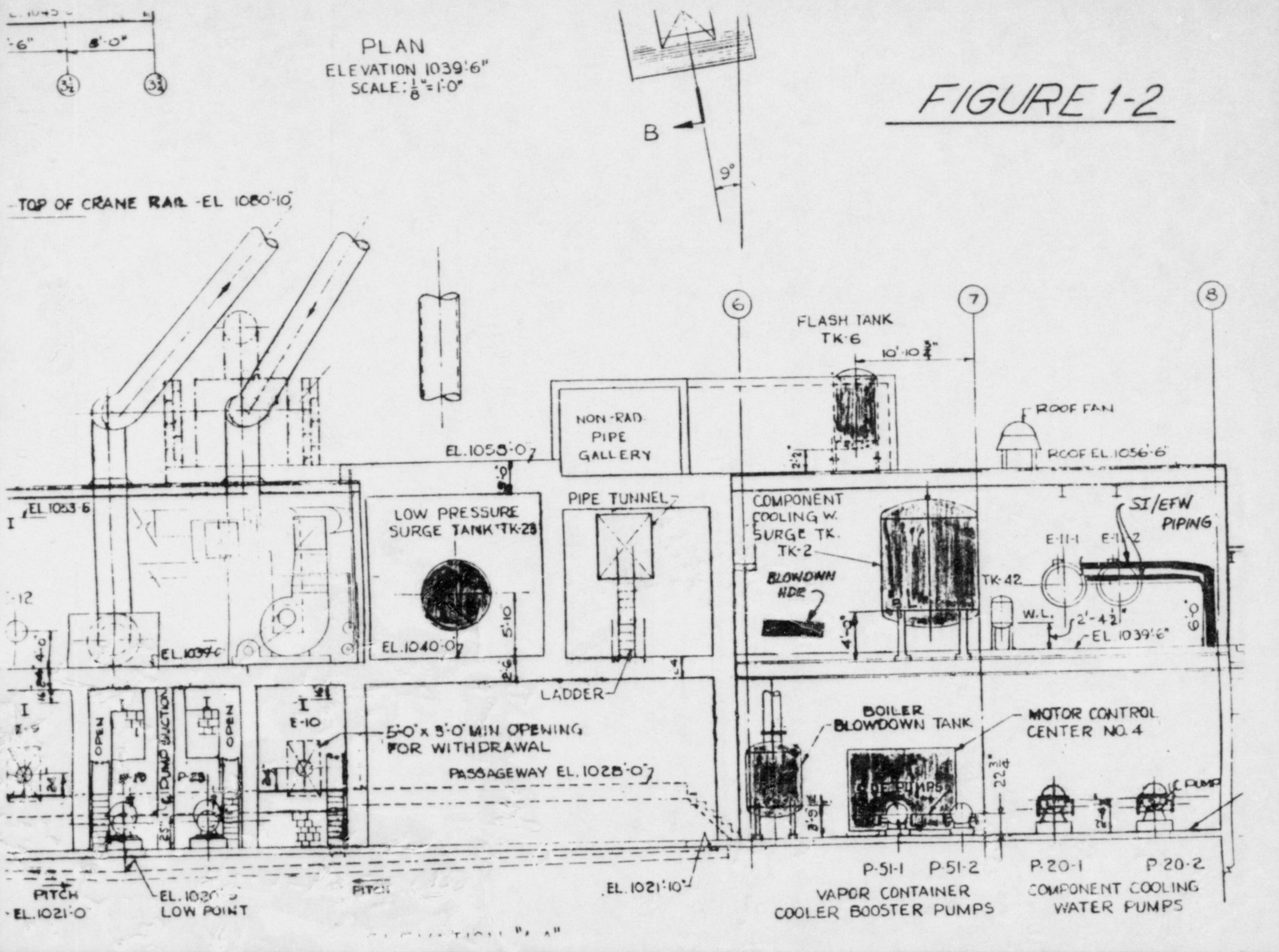
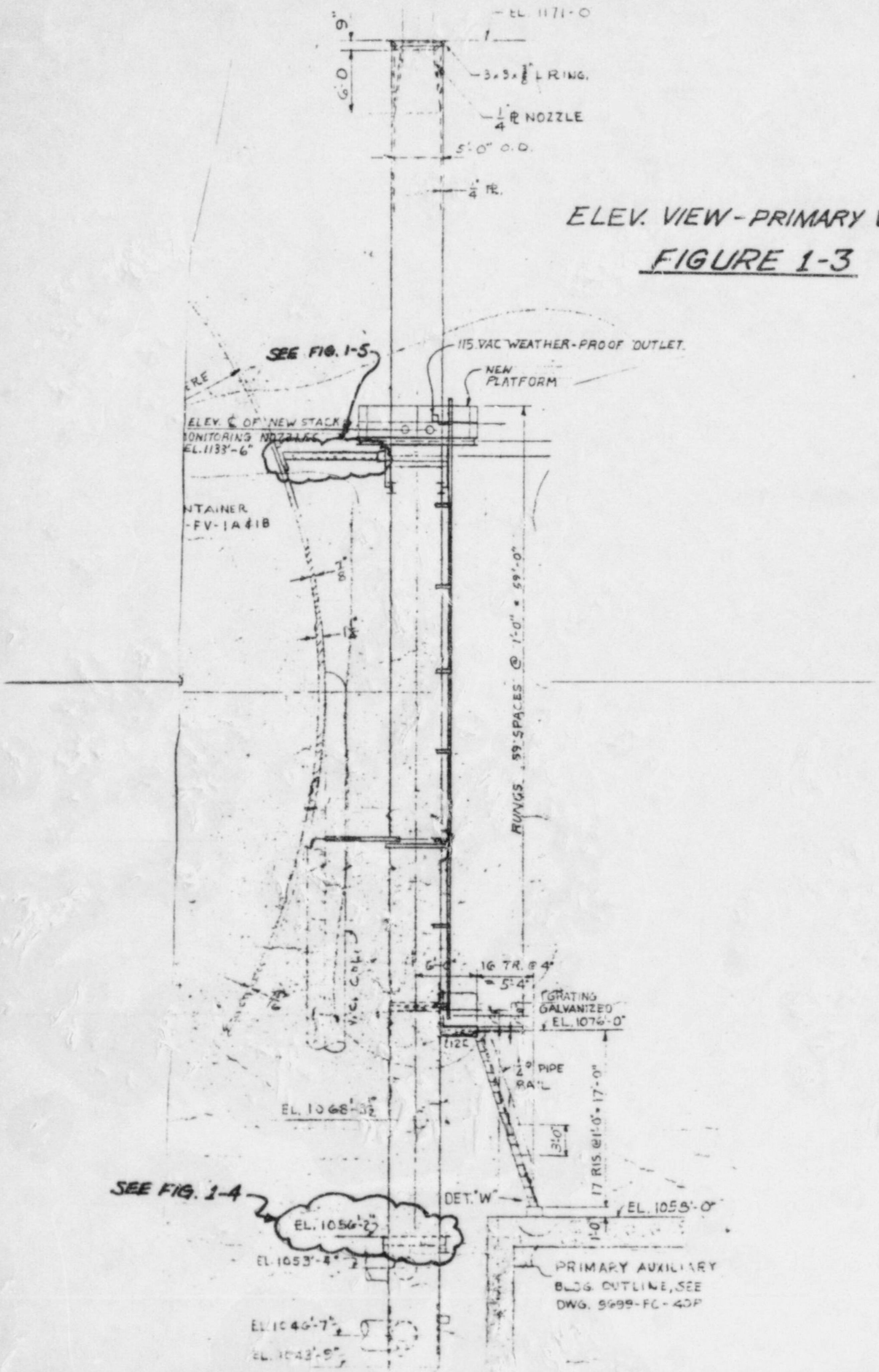
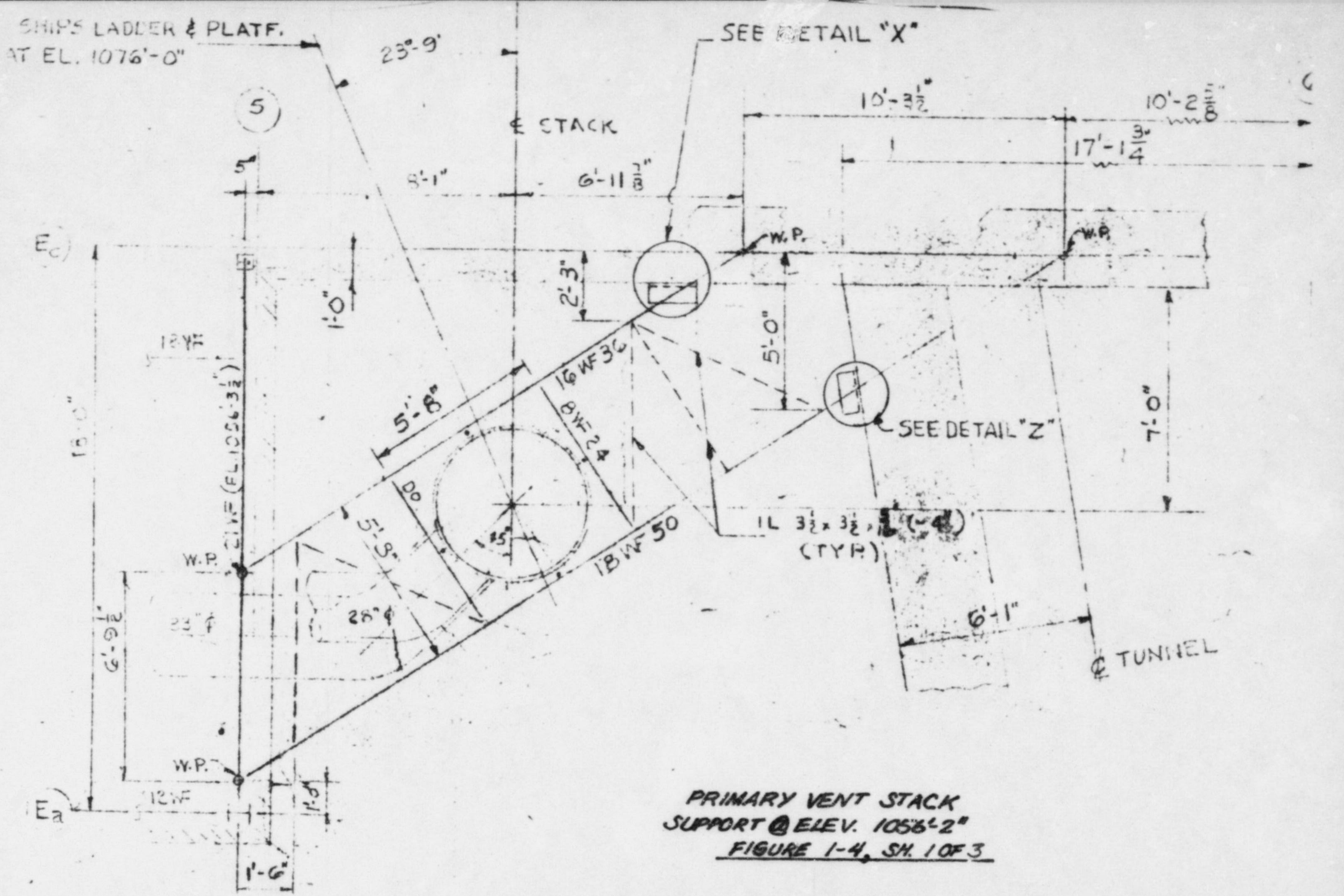

PLAN
ELEVATION 1039'-6"
SCALE: 1/8" = 1'-0"
B
9°
TOP OF CRANE RAIL - EL 1080'-10"
FLASH TANK
TK-6
10'-10 3/4"
NON-RAD
PIPE
GALLERY
ROOF FAN
ROOF EL 1056'-6"
EL. 1055'-0"
EL. 1053'-6
LOW PRESSURE
SURGE TANK TK-23
PIPE TUNNEL
COMPONENT
COOLING W.
SURGE TK.
TK-2
BLOWDOWN
HDE
SI/EFW
PIPING
E-11-1
E-11-2
TK-42
W.L.
2'-4 1/2"
EL. 1039'-6"
EL. 1039'-0
EL. 1040'-0"
5'-10
LADDER
E-10
5'-0" x 3'-0" MIN OPENING
FOR WITHDRAWAL
PASSAGEWAY EL. 1028'-0"
OPEN
P-23
BOILER
BLOWDOWN TANK
MOTOR CONTROL
CENTER NO. 4
P-51-1
P-51-2
P-20-1
P-20-2
VAPOR CONTAINER
COOLER BOOSTER PUMPS
COMPONENT COOLING
WATER PUMPS
PITCH
EL. 1021'-0
LOW POINT
EL. 1021'-10"
6
7
8

FIGURE 1-2

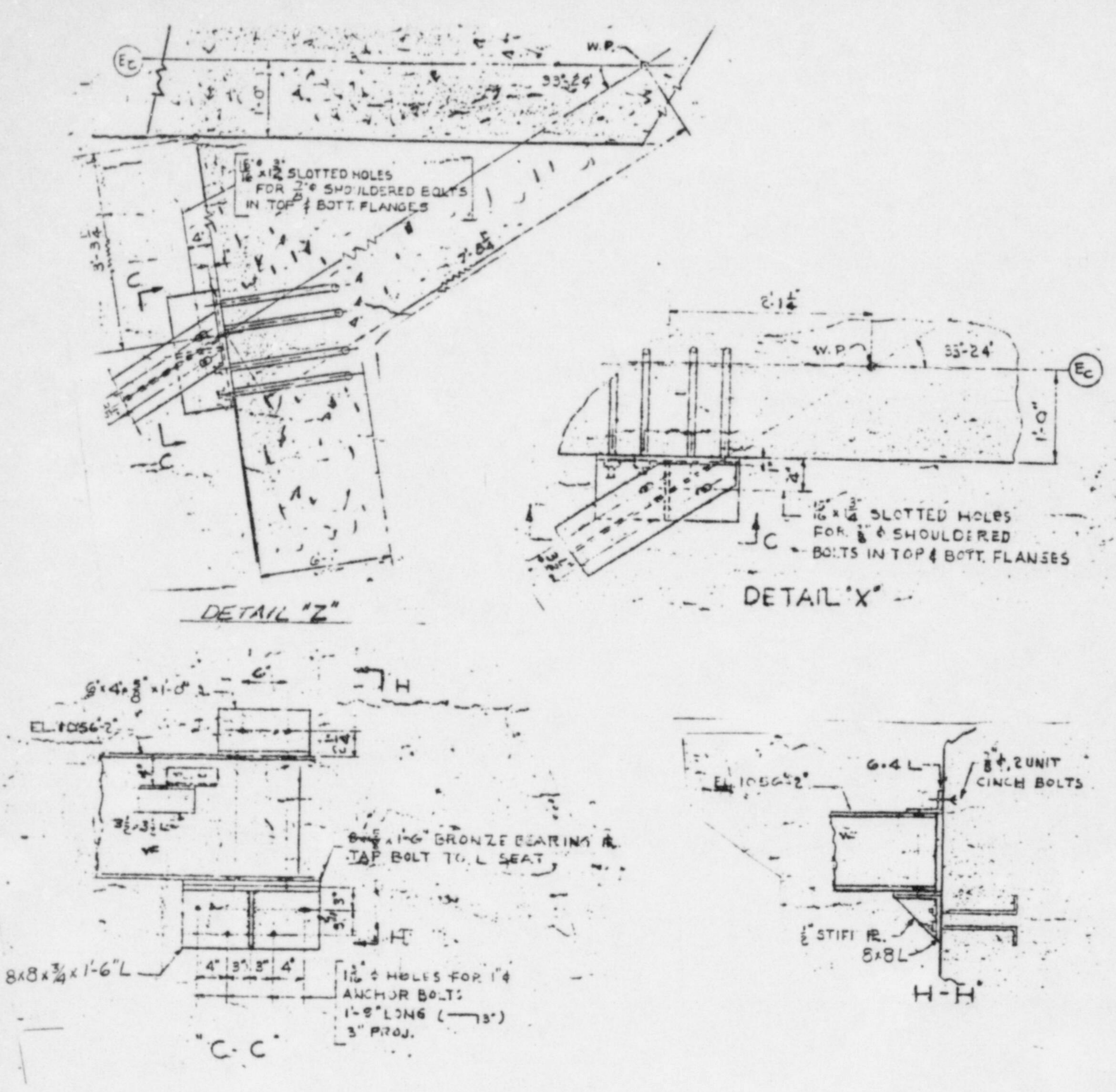


ELEV. VIEW - PRIMARY VENT STK.

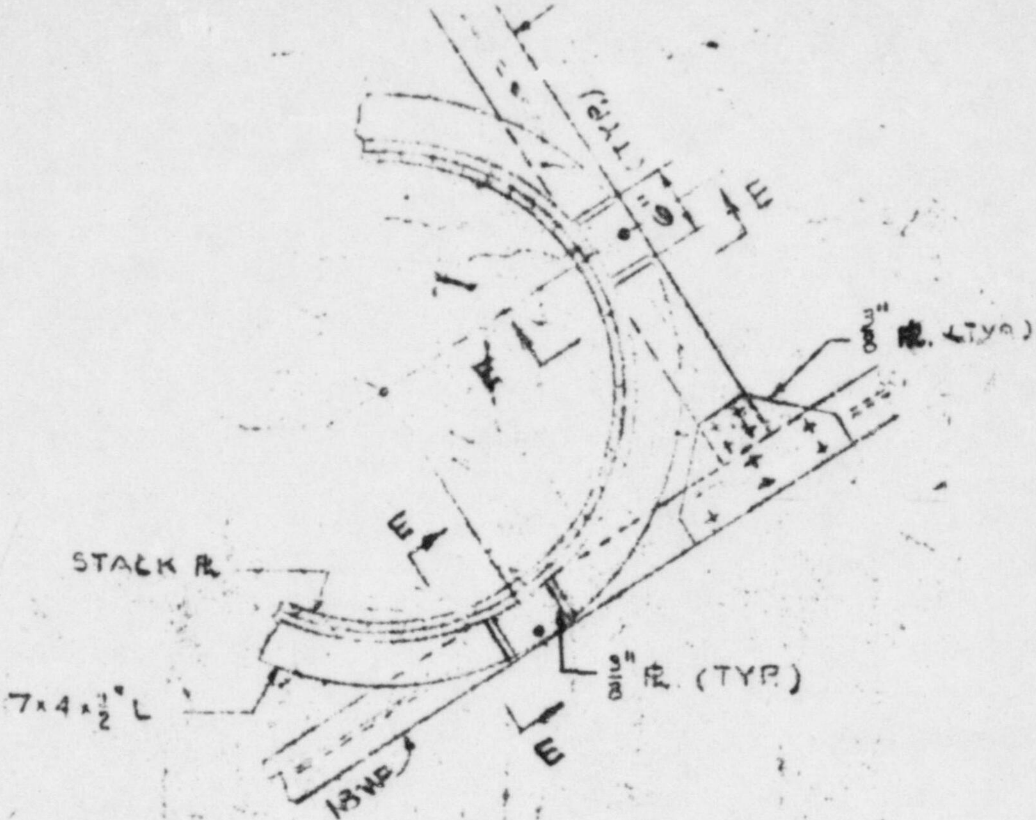
FIGURE 1-3



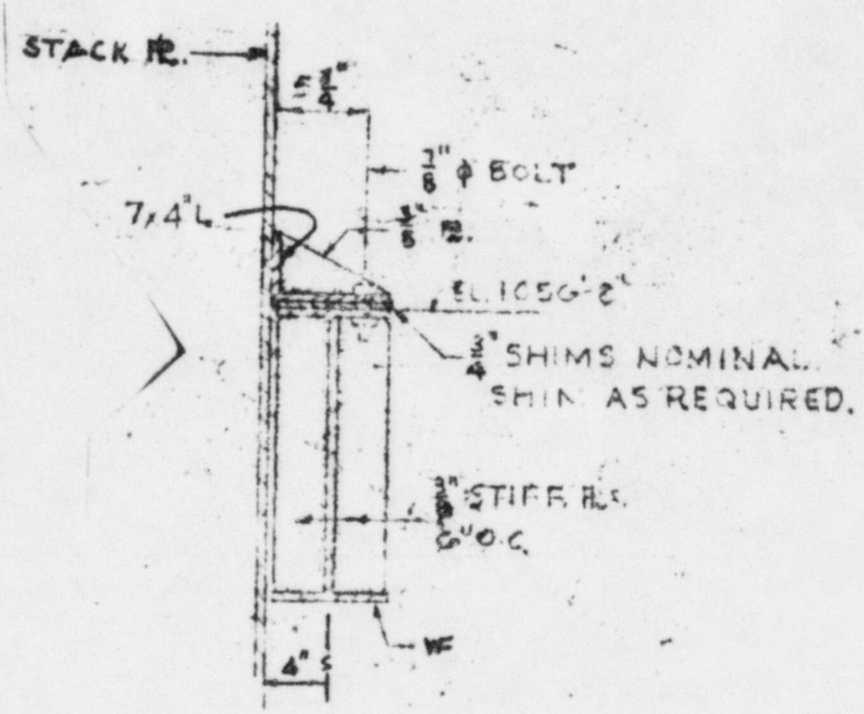
PRIMARY VENT STACK
SUPPORT @ ELEV. 1056'-2"
FIGURE 1-4, SH. 1 OF 3



PRIMARY VENT STACK
SUPPORT @ ELEV. 1056'-2"
FIGURE 1-4, SH. 2 OF 3

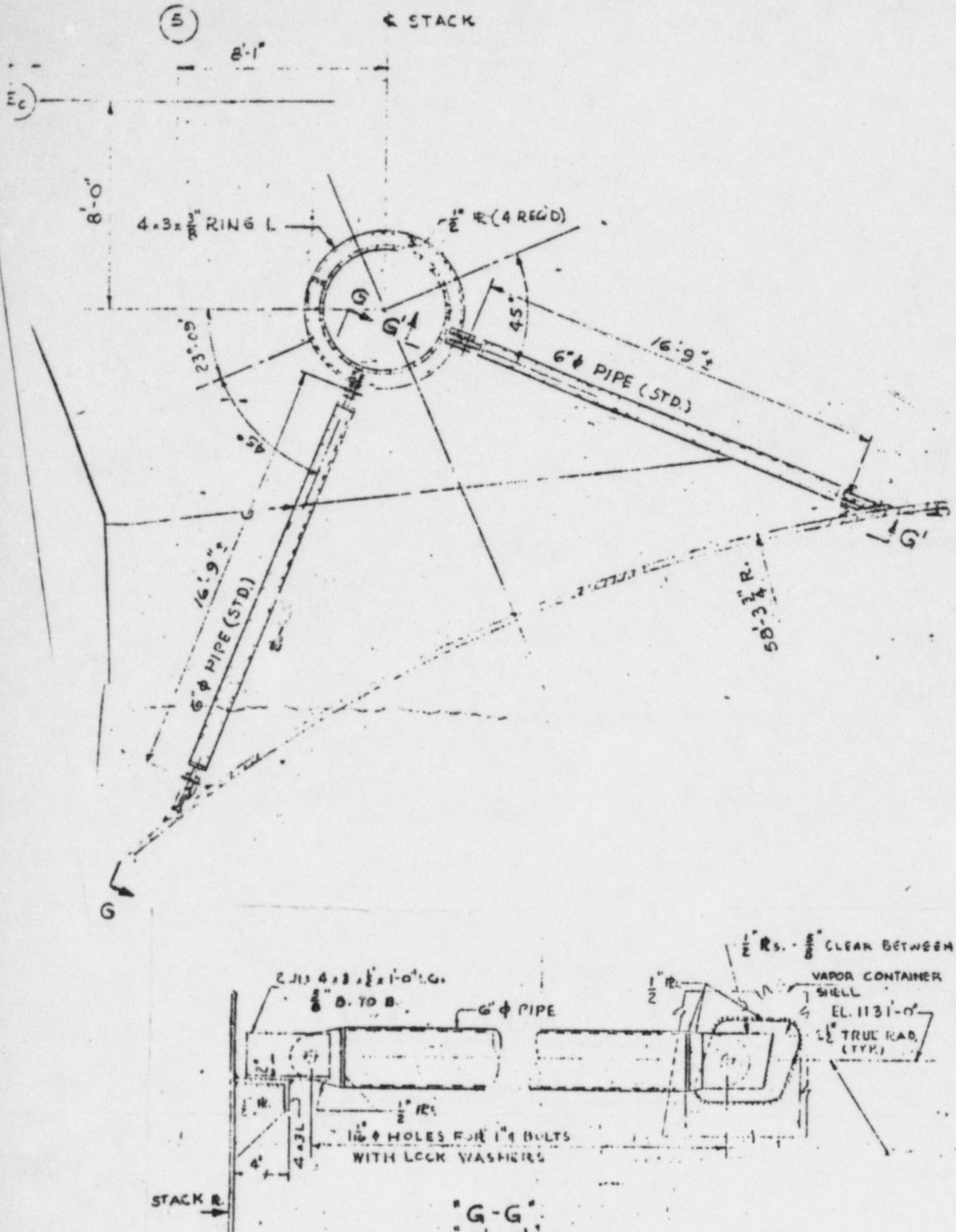


PLAN-STACK SUPPORT, EL. 1056'-2"



E-E

PRIMARY VENT STACK
SUPPORT @ ELEV. 1056'-2"
FIGURE 1-4, SH. 3 OF 3



PRIMARY VENT STACK
SUPPORT @ ELEV. 1131'-0"
FIGURE 1-5

MATHEMATICAL MODEL OF PRIMARY VENT STACK

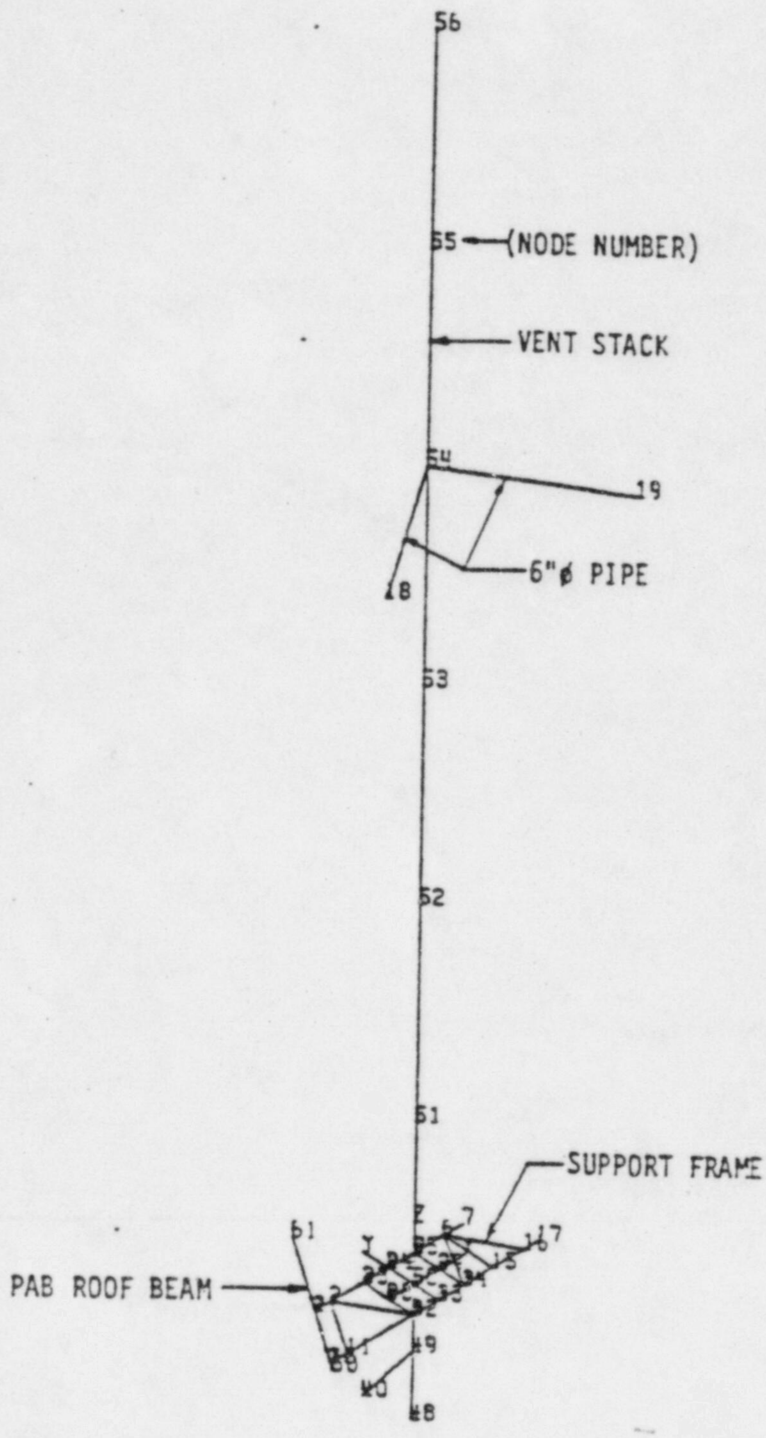
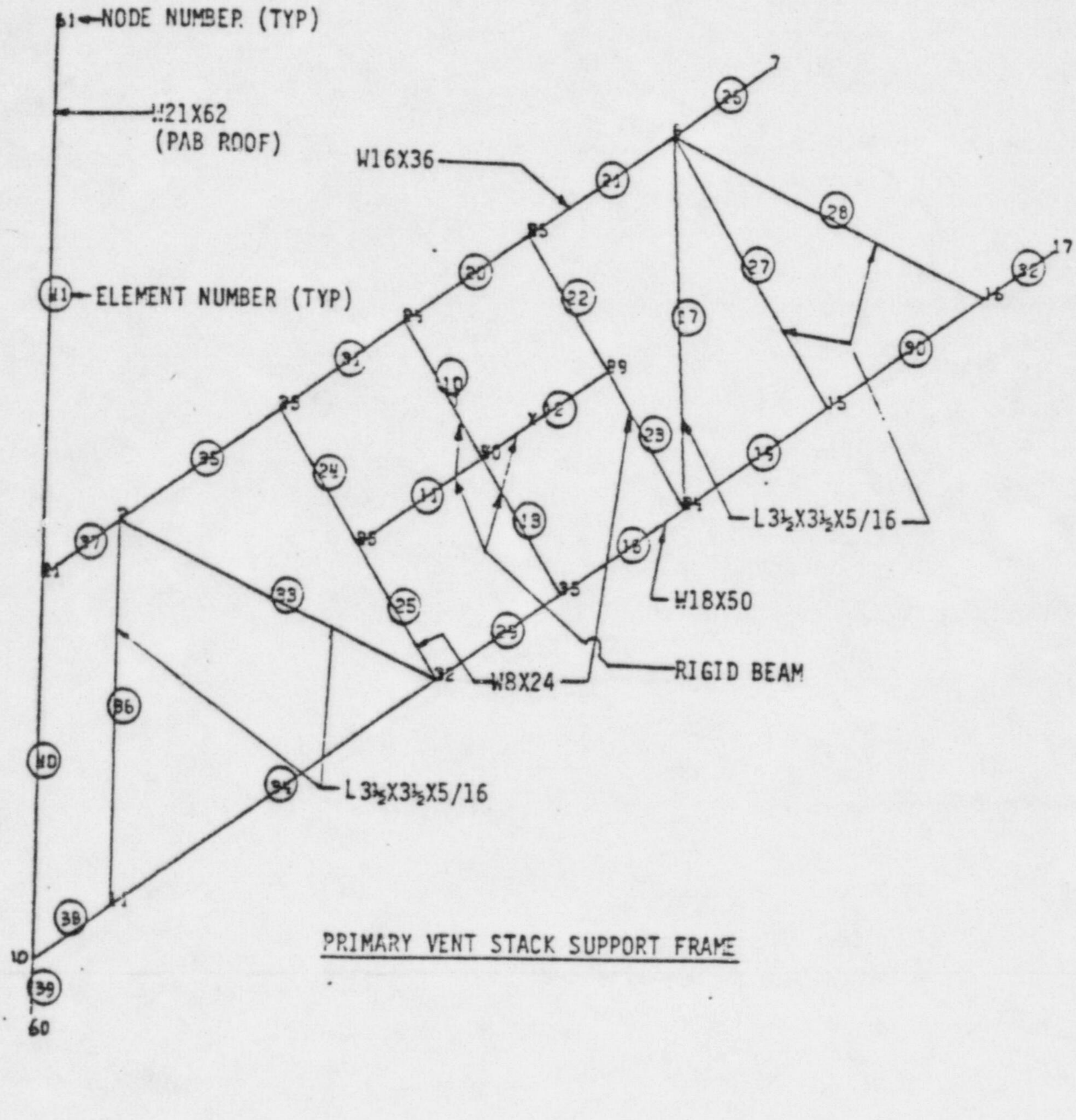


FIGURE 1-6

MATHEMATICAL MODEL OF PRIMARY VENT STACK (CONT'D)



PRIMARY VENT STACK SUPPORT FRAME

FIGURE 1-7

ENCLOSURE 2

Response to Enclosure 2 of Letter, J. W. Clifford (NRC) to G. Papanic, Jr. (YAEC), dated April 17, 1986 (NYR 86-73)

QUESTION 1

The random failure rate in the risk analysis for the Safe Shutdown System, 0.01 for a 24-hour period, was based on the preliminary design information available at the time the study was performed. Yankee Atomic Electric Company (YAEC) is requested to re-evaluate the failure rate for the Safe Shutdown System based on the as-built design. The human factor evaluation should explicitly address the time requirements on remote manual actions and the associated stress levels.

RESPONSE

In response to this question, YAEC performed two additional calculations.

For the first calculation, as a sensitivity analysis, the cost-benefit results were requantified assuming a Safe Shutdown System random (nonhazard induced) failure rate of 0.1, or ten times the failure rate used in the analysis. This did not change our overall conclusion that the Cable Tray House (CBT) should be upgraded for the wind and tornado loading. Even for this conservative sensitivity case, results clearly indicate that once the CBT has been improved, no further plant changes are cost-justified.

The following provides further information on the sensitivity calculation:

As explained in the above-referenced report (YAEC-1428) (Reference 2-1), the Safe Shutdown System (SSS) was credited as a source of feedwater in both the relief valve failure LOCA and non-LOCA models (see Pages 82 and 85, respectively), as well as a source of instrumentation (see Page 88). A review of these model logic expressions and their quantification indicates that the effect of increasing the SSS failure rate on demand by a factor of 10 is to increase the chance of feedwater failure (non-LOCA) and instrumentation failure by a factor of 10 for each hazard interval where the SSS is <u>not</u> already unavailable due to the hazard. The LOCA model contribution to interval core melt probability would increase slightly for certain cases. The LOCA model was reviewed on a cutset-by-cutset basis, the contribution of any cutset identified which included the SSS was increased by a factor of 10.

Attachment 2-1 contains marked up copies of the appropriate pages from YAEC-1428 showing how core melt quantification and the cost-benefit calculations would change for this sensitivity case. Both the base case and the Cable Tray House upgrade are presented. As the attachment shows, even if the SSS failure rate on demand was as high as 0.1, the Cable Tray House would still not be cost-justified for the best estimate case, although it would be for the conservative 95% confidence case. Since YAEC has committed to install this upgrade, this does not change our overall conclusion that only the Cable Tray House should be upgraded for wind and tornado loading.

The most important conclusion to recognize about this sensitivity case is the following:

As shown on the marked up copies of Pages 148 and 151 (Attachment 2-1), once the CBT upgrade is installed, the total person-rem exposure per year due to wind and tornado-related hazards is conservatively calculated to be 0.19 at the 50% hazard confidence and 3.07 at the 95% hazard confidence. This means that with the CBT upgrade installed, the remaining justified cost to reduce person-rem to zero (the absolute maximum justified cost for any upgrade) is only $30.7K at the 95% confidence level. It should be recognized that even this small amount is very conservative. Clearly, this amount is not enough to justify any reasonable plant upgrade. Therefore, even for this conservative sensitivity case, results clearly indicate that once the CBT has been improved, no further plant changes are cost-justified.

For the second calculation, a fault tree based system failure model was developed and quantified for the as-built Safe Shutdown System. The overall random failure rate of the SSS was calculated to be 3.2×10^{-2}, including human error which was treated as follows:

The fault tree included all human actions related to alignment, startup, and operation of the system. In order to determine the failure rate for each specific operator task, human error rates, as well as stress factors and operator interdependency factors were taken from the tables of NUREG/CR-1278, "Handbook of Human Reliability Analysis with Emphasis on Nuclear Power Plant Applications." Since the SSS would not normally be initiated unless several other feedwater sources were unavailable, it was assumed that the operator(s) stress level would be moderately high. Due to the large steam generator inventory at YNPS, operators have at least one hour to restore feedwater. The model recognizes that some time may have elapsed before a decision is made to abandon the other feedwater systems and initiate feedwater supply via the SSS. The Safe Shutdown System is relatively easy to operate and two operators would be available to initiate feed from the SSS. The operators have been trained on the system and on its operating and surveillance procedures. The operator failure probabilities for the tasks were increased by a factor of 0.15 (accounting for dependence between the operators from Table 7-3 of the handbook) to account for either operator detecting and correcting the other's error. The overall SSS random failure probability of 3.2×10^{-2} includes a total operator error probability of 1.7×10^{-2}. The logic used in developing the operator error probability is provided in Attachment 2-2.

To verify the accuracy of these assumptions, operator interviews were conducted in order to estimate how much time would elapse before the operators would start the SSS. Each operator was walked through the event and appraised of the indication of inaction of each system as he attempted to restore heat removal. Upon realizing that no normal emergency feedwater was available, each operator went directly for the Safe Shutdown System. The operator was then asked to give an estimate of the time this action would take. The response ranged from 10 to 30 minutes. Most operators related similar events they had experienced and noted the fact that there is a lot of time available. When they were reminded that they had 45 to 60 minutes to establish feedwater they expressed that there would be no chance that it would take them anywhere near that time, despite the severity of the initiating event. The analyst performing this interview is confident that this response is accurate based on his knowledge of the operator training and experience and on the complexity of the actions required.

QUESTION 2

The report states that top event LG (and OF), that is, failure of steam removal via the atmosphere dump valves or safety valves, is negligible. YAEC is requested to clarify the basis for this assumption considering the range of possible wind speeds.

RESPONSE

Steam removal from the secondary system is accomplished by any one of four Atmospheric Steam Dump (ASD) valves or any one of twelve steam generator safety valves. All sixteen valves are located on the nonreturn valve platform. The ASDs are motor-operated valves powered by emergency 480 V ac electric power and controlled from the Main Control Room. Local manual operation is also possible. The failure of sixteen valves is clearly very unlikely. It may be possible to postulate wind/tornado-related failure modes which would affect remote control of the ASDs (i.e., cable damage) and make local operation difficult; however, for the range of wind speeds involved here, direct wind-induced physical damage to the valves is not considered credible. The failure of the nonreturn valve platform is not an important contributor to failure as discussed below for Question 11. As a minimum, steam removal would be available through the safety valves throughout and after the wind/tornado event, and local operation of the ASDs would be available following the event. There is some probability of failure of remote ASD operation; however, a total failure of steam removal capability is negligible.

The wind capacity of the atmospheric steam dump valves and steam generator safety valves is greater than 178 mph.

QUESTION 3

The development of top event LE, failure of the recirculation mode, in Section 6.3 does not include the possibility of the operator failing to initiate the realignment process in time. Further justification for this logic development is needed.

RESPONSE

As was discussed in Section 6.3, for event LA "...the only LOCA to be reasonably considered for the wind/tornado hazard is a relief valve LOCA." For such a relatively small LOCA, safety injection would operate for over four hours in the injection mode before an operator action to initiate the realignment for recirculation would be required. Because of the amount of time available to perform this action, an operator error here is not expected to be an important contributor, especially when compared to the other failures included in event LE. An operator error of failing to initiate realignment in the expression for LE would be coupled through an "OR" gate with three other failures including the failure of a diesel generator to run for 24 hours which was conservatively assigned a value of 0.1 for this analysis, rendering the operator error contribution negligible.

QUESTION 4

Section 6.5 states that the Turbine Building west staircase was not modeled explicitly because failure of this area does not directly impact any system credited in the analysis. Section 3.4.2 indicates that cabling for operation of vital equipment, such as the nonreturn valves and atmospheric steam dump valves, passes through this area. Please clarify this situation.

RESPONSE

The analysts did not consider the failure of cabling for essential equipment located in the west staircase a credible failure mode since only the south and west walls of the staircase are exterior walls and both are constructed of reinforced concrete. The south wall is 3'-0" thick. The west wall is 12" thick and is designed as a shear wall used for resisting seismic loadings. Winds in the range of interest in this analysis are not capable of breaching the integrity of these walls. The north and east staircase walls are interior walls and would not be exposed to direct wind loading as was demonstrated to NRC representatives during the plant walkdown in February 1986. Design loads for recent modifications to the north and east staircase walls considered both seismic and, where appropriate, tornado delta P loads.

QUESTION 5

It does not appear that the consequences of failure of the "upper level Primary Auxiliary Building south wall" was explicitly addressed in the risk analysis. Please provide the basis for exclusion or an assessment of its wind capacity and consequences of failure.

RESPONSE

Although not explicitly modeled in the risk analysis, the south wall of the upper level Primary Auxiliary Building (PAB) was not excluded from the analysis. Failure of this wall was evaluated. It was found that such a failure would not impact any equipment related to event mitigation nor would it cause any complication of the event.

QUESTION 6

The failure of the "lower level Primary Auxiliary west wall" was excluded from the analysis because it is bordered by an adjacent room, the Hydrogen Storage Room. Please provide an assessment of the potential for a hydrogen explosion from wind-induced damage to this area.

RESPONSE

The "lower level Primary Auxiliary west wall" is reinforced concrete and is bordered by the Safety Injection Building. The Hydrogen Storage Room stands adjacent to the southernmost portion of the Upper Level Primary Auxiliary Building (ULPAB) west wall (see attached plan and elevation drawings, Figures 2-1, 2-2, and 2-3). Equipment in the upper level PAB important to this analysis is located along the north wall and at the east end of the building. There is no equipment important to this analysis in the Hydrogen Storage Room nor in the area of the PAB adjacent to the Hydrogen Storage Room. In order for a wind/tornado event to cause a hydrogen leak in the Storage Room, it would have to fail one or more outer walls of the room; such a failure would allow significant ventilation of the room making it very unlikely, if not impossible to get a potentially explosive hydrogen mixture in an enclosed area.

If a hydrogen explosion were to occur, since the hydrogen bottles are located in the center of the room, the force could be expected to be equally distributed on all walls (one or more of which would be failed and open to the outside). Looking at the plan view of the ULPAB and Hydrogen Storage Room (Figure 2-1), it can be seen that a 15-foot distance exists between the north corner of the H2 Storage Room and building line Ec, the northwest corner of the ULPAB. The emergency feedwater line and safety injection line are located about seven feet south of line Ec, about one foot off the west wall. The lines rise about seven feet above the floor and travel east to the northeast corner of the building, Column Line No. 6 of the building, to the blowdown header. From the plan and elevation view, it can be seen that the component cooling heat exchangers are directly in front of the Hydrogen Storage Room wall. These heat exchangers are approximately four feet in diameter and 18 feet long. From elevation view "A-A", it is readily apparent that due to the elevation difference of the ULPAB and the H2 Storage Room, these heat exchangers alone, not crediting the six to eight inch service water lines located adjacent to the wall, would shield the remaining equipment in the room from direct impact from a spalling concrete block wall. Additionally, if a block, or some portion of it, were to strike the eight-inch, Class 302, Schedule 40, seamless stainless steel safety injection line or the Class 601, four-inch, Schedule 80, carbon steel emergency feed line, damage such as to interrupt flow or breach the piping wall is not credible.

QUESTION 7

Section 6.5 states that failure of the south and/or west walls of the Safety Injection Building could cause damage to the fire water tank heater, a potential flooding hazard. Please provide a discussion of the effects of failure of this tank on equipment in the area credited in the risk analysis and not already assumed to be failed by failure of the south and west walls.

RESPONSE

Failure of the south and/or west walls could impact a small heater which provides fire water tank heating. The fire water tank itself is located outside in the southwest yard. It was determined that the potential failure of the tank heater would result in a less than 10 gpm leak onto the SI Building floor. Given the size of the berms and curbs for equipment in the SIB, along with the building drains (and the wall opening caused by the initiating event), the flow rate is insufficient to impair any of the equipment credited in this analysis.

QUESTION 8

During the plant site visit of February 4, 1986, YAEC verbally agreed to upgrade the wind capacity of the upper level Primary Auxiliary Building west wall. Please provide the details of the modifications and what wind capacity will be afforded by the proposed upgrades.

RESPONSE

Design of the modifications to the ULPAB west wall is scheduled to be done in 1988. If required, additional structural steel will be used to reinforce the wall in a manner similar to that used for the PAB north wall. Any required modifications will be designed in accordance with DCD-2648-6-1 (Reference 2-4) for a wind/tornado speed of 134 mph, resulting in an ultimate capacity somewhat greater. The EFW piping which runs along the west wall is part of the Emergency Feedwater System, which is assumed to fail at 134 mph (straight wind)/121 mph (tornado). Since the ultimate wind/tornado capacity of the modified west wall will be greater than 134 mph/121 mph, the failure of the wall will not contribute to core melt frequency.

QUESTION 9

The term ULPAB is used to represent location failure of the blowdown header either in the upper level Primary Auxiliary Building or nonradioactive pipe tunnel. From an examination of the logic expression used to represent failure to supply feedwater assuming no off-site power, ULPAB will, by itself, lead to failure of safety injection and Safe Shutdown System supply to the feedwater lines, but that an additional failure (e.g., LLPAB) is needed before electric emergency feedwater and charging pump supply to the feedwater lines would be lost. Since electric emergency feed and charging feed paths are also located in the upper level Primary Auxiliary Building, an explanation is needed for the logic expression used in the model.

RESPONSE

As discussed on Pages 84 and 85 of YAEC-1428 (Reference 2-1), the only path credited for feeding the steam generators from safety injection or from the Safe Shutdown System was the blowdown header. For both charging and electric emergency feedwater both the blowdown header and the main feedwater lines were credited as possible feed paths for these two systems. Discharge piping to the blowdown header is normally isolated. More importantly, both charging and electric emergency feedwater require ac power. The analysis considers all ac power failed at 134 mph straight wind or 121 mph tornado hazard, well below the predicted failure of either the ULPAB or the LLPAB.

The modifications to the ULPAB and LLPAB north wall are being designed for a 10^{-5} (165 mph) tornado event.

QUESTION 10

Wind/tornado loads for interior walls and systems have not been assessed in general. However, in selected areas, a more detailed evaluation of the possible loading and wall capacity is needed given that the exterior wall has failed. For instance, the wind capacity for the Diesel Generator Building west wall is significantly lower than the wind capacity for the Diesel Generator Building north wall. This disparity in wind capacities was not found to be significant in the risk study because the failure of the west wall was assumed to damage only one diesel generator, while the failure of the north wall results in the loss of all three emergency diesel generators. The staff will need further assurance that the interior Diesel Generator Building walls will not fail given that the west wall has experienced failure.

Furthermore, the licensee is requested to address the above staff concern about the interior walls near the following walls:

Auxiliary Boiler Room South Wall T1J2
Lower Level Primary Auxiliary Building Walls P1E1 and P1E2
Upper Level Primary Auxiliary Building Walls P2F1 and P2F2
Safety Injection Building South Walls D1Z1 and D1Z2
Safety Injection Building West Walls D11051 and D11052
Safety Injection Building North Wall D1X1
Diesel Generator Building West Wall D11053
Diesel Generator Building North Walls D1V1, D1V2, and D1V3

These walls were chosen because their failure under wind loads may have a significant impact on the core melt frequency.

RESPONSE

The treatment of exterior block walls and the assumption of no failure of interior walls are concluded to be reasonable due to the conservatisms present in the cost-benefit analysis. These conservatisms are present in the calculation of the ultimate lateral pressure of the block walls, in the assumed failure mode of the block walls, and in the calculation of straight and tornado wind speed capacities given the wall's ultimate lateral pressure.

Each of the above conservatisms is discussed in detail below.

Calculation of Ultimate Lateral Pressure Load and Wall Failure Mode

The failure of the unreinforced masonry block walls is governed by the tensile strength of the mortar. When the tensile strength is exceeded, the block wall is assumed to fail in its entirety and fall into the building (or room) a distance equal to the wall height. This assumption is conservative for two reasons. First, failure of the mortar in tension at one point in the wall does not necessarily lead to complete wall collapse. Second, winds can produce wall loadings from windward, leeward, or parallel directions.

In the windward case, the loading on the wall acts inward to the building, thus wall failure would also be inward. In the leeward and parallel cases, the loading acts outward from the building, thus walls would tend to fail outward. Conservatively, the walls were assumed to fall inwards towards equipment for windward, leeward, and parallel wind loadings.

Also, wall failure was based solely on wind speed capacity. At a specific wind speed level, all walls with capacities at or below that wind speed were assumed to fail inward without regard to wall alignment or shielding from other buildings and the surrounding terrain. This is a conservative assumption and, if accounted for, would tend to increase the capacities of some walls. Additionally, some walls would have an increased wind capacity, given that walls facing some other direction had failed.

Additional conservatism is present in the assumed failure mode for the block walls. The walls are assumed to fall in an arc equivalent to the height of the wall. Any equipment located within the arc was assumed to be rendered inoperative.

Conversion of Lateral Pressure to Wind Capacity

Wind capacities of the block walls are given in Table 5-2 of the cost-benefit analysis (Reference 2-1). The wind capacities were developed from the ultimate lateral pressure loads as discussed in Section 5.0 of the study. The conservatisms included in the conversion are discussed below.

o Straight Winds

Wind capacities for straight winds were determined per ANSI A58.1-1982. In Section A6.7 of A58.1, it is noted that the pressure coefficients provided, "...represent the upper bounds of the most severe values for any wind direction. The reduced probability that the design wind speed may not occur in the particular direction for which the worst pressure coefficient is recorded has not been included in the values of the tables." As noted above, these pressure coefficients are conservative and, therefore, the straight wind capacity of the wall is also conservative. Median-based pressure coefficients would be more appropriate for this analysis.

As previously noted, a direct windward wind on a wall is most critical. Extreme straight winds at the site are expected to show no preference in direction and, therefore, the probability of the wind with a worst case direction is somewhat below that from the hazard curves which have no direction preference.

Finally, in calculating straight wind capacities, gust response factors are explicitly accounted for in the analysis. As previously noted, the failure criteria for the block walls is exceedance of ultimate lateral pressure. Predicted wall failure is for the intermittent gust loading which is less likely to lead to total failure of the wall than if constantly loaded at the gust-factored wind level.

In applying the design methodology of ANSI A58.1-1982 to determine ultimate wind capacities of the block walls, some modifications were made to the design procedures. The ANSI A58.1 methodology was developed to be applicable to all types of buildings and structures. By its nature as a design document, certain conservatisms are "built-in" throughout the ANSI A58.1 procedures. In the cost-benefit risk assessment, realistic capacities for the structures, systems, and components are the desired input. As noted in the sample calculations previously submitted to the staff (Reference 2-3), the internal pressure component was not included in the wind capacity calculations. Internal pressure coefficients are given in ANSI A58.1. Depending on the external load on the wall due to wind pressure, the appropriate positive or negative value of the internal pressure coefficient is used in determining the controlling load requirements. For example, in the case of a windward loading on an exterior wall which produces an external pressure acting toward the wall, ANSI A58.1 would call for a negative internal pressure which would increase the overall pressure on the wall. This is a conservative procedure in that upper bound external and internal pressure coefficients are combined even though they are not necessarily produced by the same wind. Note in the above example that a positive internal pressure would tend to offset an external windward pressure on the wall.

Ventilation, elevation, plan, and building service drawings for the Turbine, Primary Auxiliary, and Diesel Generator Buildings were reviewed for building ventilation layouts and wall penetrations. Penetrations in the block walls analyzed in the cost-benefit analysis consist of louvers, doors, and exhaust dampers. Interior rooms bounded by these walls either have their own ventilation system (e.g., diesel generator cubicle) or are interconnected to the building ventilation system (e.g., the PAB). The effect of the ventilation systems would be to offset any tendency to over or underpressurize building interiors. Also, in most instances, the internal pressures, ignoring ventilation effects, would tend to produce a beneficial internal pressure which would offset the external pressure.

In conclusion, a review of each wall for windward and leeward wind loadings was performed, and it was concluded that use of an internal pressure component of zero, in lieu of those required by ANSI A58.1, was reasonable in the determination of the expected ultimate failure capacity for both straight winds and tornadoes.

o Tornadoes

The tornado hazard wind speeds used in the cost-benefit analysis are referenced to 30 feet above ground level and are the maximum horizontal wind speeds. Although not credited in the analysis, there is a reduction in tornado wind speeds near ground level due to boundary layer effects. In Appendix J to the Seabrook Station SSER 3 (Reference 2-2), the NRC consultant finds the reduction of tornado wind speeds at ground level to 75% of the reference level speed (at 33 feet) to be reasonable. Taking no credit for this reduction in the cost-benefit study, is a conservative feature which, if included, could increase tornado wind capacities of the Diesel Generator Building walls on the order of 25%.

The predicted tornado capacity for the block walls is based on centering the tornado speed distribution on each side of the building or building complex and determining the average pressure on each side of the building. This conservative approach assumes that all building sides are simultaneously hit by the tornado and applies the average pressure on the building side to each block wall component. All buildings and structures in the cost-benefit analysis were treated in this manner.

Finally, although not explicitly accounted for in the tornado hazard analysis, the location of the site, nestled into the side of the steep river valley, provides some measure of topographical shielding from tornadoes.

Interior Walls

Below is a discussion of the specific interior walls listed in Question 10:

Auxiliary Boiler Room South Wall T1J2 - Failure of this exterior wall is assumed in the cost-benefit model to fail the steam-driven emergency feedwater pump. This is the only equipment in the ABR important to the model. Failure of interior walls near this wall is, therefore, of no importance to this analysis.

Lower Level Primary Auxiliary Building Walls P1E1 and P1E2 - The only interior walls in this area are the east and west walls of the building. The east wall separates the LLPAB from the charging pump cubicles and is constructed of reinforced concrete. The west wall is also reinforced concrete and forms the boundary with the Safety Injection Building, and is discussed below. Additionally, no LLPAB exterior wall failure is predicted to occur unless wind speeds reach 197 mph (or 222 mph tornado).

Upper Level Primary Auxiliary Building Walls P2F1 and P2F2 - The only interior walls in this area are the east wall which forms the boundary with the Valve Room and is constructed of reinforced concrete and the southern part of the west wall which was discussed above in response to Question 6.

Diesel Generator Building West Wall, D11053, and North Walls D1V1, D1V2, and D1V3 - A plan view of the Diesel Generator Building (DGB) is presented in Figure 2-4. This figure shows equipment locations and wall designations. The effect of the failure of each wall section in the DGB is discussed below:

D1V1, D1V2, D1V3 - Wall failure is assumed to disrupt/destroy diesel generator cooling air/water for the respective diesel generator.

D1X1 - Wall failure is assumed to disable <u>ALL</u> power and control cables to and from Emergency 480 V ac Buses 1, 2, and 3 and their associated loads.

D1Z1 and D1Z2 - Wall failure is assumed to disable Battery No. 3 and low pressure safety injection pump motors and instrumentation.

D11051 - Wall falls on the No. 3 LPSI pump, fire water tank heater, and low pressure to high pressure safety injection header division valve.

D11052 - Wall falls on No. 3 HPSI pump and motor.

D11053 - Wall failure disables the No. 3 emergency 480 V ac diesel generator.

From Table 5-2 of the cost-benefit analysis (Reference 2-1), the capacities of the west wall, D11053, for straight wind and tornado are [illegible]1 mph and 6[illegible] mph, respectively.

Even though the wind capacity for the Diesel Generator Building west wall is significantly lower than the north wall (D1X1), the controlling wall for loss of all three diesel generators is Wall D1X1. Wind capacities of Wall D1X1 are 134 mph for straight wind and 121 mph for the tornado. Wall D1X1 controls, since, upon failure vital cabling necessary for diesel generator operation is assumed lost.

Treatment of the diesel generators in the cost-benefit analysis is concluded to be reasonable for the following reasons. The interior diesel generator cubicle walls, D11043 and D11054, are similar in dimensions and construction to the west wall, D11053. However, assuming the complete or even major destruction of the west wall, there are mitigating factors which would prevent the collapse of the interior walls. These factors are: shielding from remaining wall segments, the roof, north wall, and equipment; potential for outward directed wind forces; short duration of maximum tornado loadings; the unlikelihood of a direct high sustained windward wind on the west wall, given no preference for wind direction, so as to progressively fail Walls D11054 and D11053; and assumed loss of all three diesel generators with failure of Wall D1X1, which itself is a conservative assumption.

From the hazard curves, Figure 4-1 of the cost-benefit study, the annual exceedance probabilities of these failure wind speeds for D11053 at the 50th and 95th confidence levels can be determined. For the straight wind of 91 mph, the probabilities at the 50th and 95th confidence levels are 1×10^{-4} and 1×10^{-3}, respectively. For the tornado wind of 69 mph, the equivalent probabilities are 3×10^{-5} and 2×10^{-4}. Therefore, the use of 95% confidence level wind speeds in the determination of potential plant modifications is conservative.

Safety Injection Building South Walls D1Z1 and D1Z2,
Safety Injection Building West Walls D11051 and D11052,
Safety Injection Building North Wall D1X1 - Interior walls of the SIB are those related to the diesel generator cubicles which were discussed above in detail; a portion of the north wall forming a boundary with the PICS Building which is constructed of reinforced filled block and seismically designed; the block walls of the northeast corner which separate and are protected from wind loads by the SI accumulator; and the east wall which forms the boundary with the LLPAB and is constructed of reinforced concrete.

<u>QUESTION 11</u>

Please provide an assessment of the wind capacity of the main steam/main feedwater lines and their support structures.

RESPONSE

Yankee has committed to upgrade the supporting structures of the main steam and feedwater (MS/FW) piping for wind and seismic loads. The engineering will be performed in 1987 with plant modifications to follow in 1988.

In order to eliminate MS/FW piping failure from any contribution to the cost-benefit analysis of core melt frequency, the piping must have an ultimate wind capacity of 163 mph and 178 mph for tornado. Although lower ultimate wind capacities for this piping would not necessarily be a significant contributor to risk, Yankee will include the loading from the above wind speeds in the forthcoming MS/FW analysis. These wind loads will be applied to the exposed MS piping between the vapor container penetrations and the Turbine Building. All exposed (outdoors) FW piping will be evaluated for the same wind loads.

Any supports necessary to resist these wind loads will be included with the seismic upgrades previously committed to.

QUESTION 12

The failure under wind loads for some components and/or structures may have a significant impact on the core melt frequency. Therefore, the licensee is requested to provide more detailed information concerning the analytical technique and criteria to confirm the adequacy of the wind capacities of the following items listed in Table 5-2 of the licensee's September 1984 report (October 24, 1985 submittal):

a) Tanks - TK-1, TK-28, and TK-39

b) Nonradioactive Pipe Tunnel

c) Safe Shutdown System Pump House

d) Cable Spreading Room with Fix

RESPONSE

Detailed calculations containing the analytical techniques and criteria used for determining the tornado/wind capacities of the above structures and components were made available during the May 1986 meeting.

The analytical techniques and criteria used are also discussed below:

a. Tanks TK-1, TK-2, and Tk-29 were designed, fabricated, and constructed in accordance with API Standard 126, "Specifications for Welded Aluminum Alloy Storage Tanks" (Tentative), March 1957. Tanks were fabricated from GR 20A aluminum with fy = 9.5 ksi and f (allow) = 7.3 ksi. Each tank was designed for a 25 psf wind load on projected area.

   The "ultimate" or failure wind load was deduced through evaluation of API Standard 126 design criteria. It was found that the design methodology contained therein closely correlates to API Standard 650, "Welded Steel Tanks for Oil Storage," 7th Edition, 1980.

   The correlation between API Standards 126 and 650 was checked through review of the standards and equating the wind girder requirements (section modulus) from each standard. Proper units were used when comparing the required section modulus equations from the two standards.

   It is reasonable to assume that the above tank designs used a 1/3 increase in allowable stresses for the design wind load of 25 psf. By increasing the design wind load by 20% (to 30 psf), the allowable stresses are then proportionally increased by 1.33 x 1.20 = 1.66 factor. The Aluminum Association "Specifications for Aluminum Structures," 3rd Edition, 1976, in Table 3.3.3, indicate minimum allowable stresses equal to yield stress divided by a 1.65 safety factor. For the PRA work, it was therefore deemed justified to use 30 psf for ultimate lateral pressure loading.

The corresponding wind capacity (mph) for each tank was then calculated based on the ultimate lateral pressure loading.

b. The nonradioactive pipe tunnel or Upper Pipe Chase (UPC) was analyzed in accordance with C. T. Main, Document No. 2648-6-1 (Reference 2-4).

The UPC walls, as modified, will remain within the allowable limits in Section 11 of the Structural Design Criteria (Reference 2-4) when subjected to a 10^{-5} (165 mph) tornado event.

c. The Safe Shutdown System (SSS) Pump House was designed in accordance with ACI 318-83, "Building Code Requirements for Reinforced Concrete."

The load combinations considered were:

1. $U = 1.4D + 1.7L$

2. $U = 1.4D + 1.7L + 1.9E$

3. $U = 1.4D + 1.7L + 1.7W$

4. $U = 1.2D + 1.9E$

5. $U = 1.2D + 1.7W$

where E = seismic loads based on the Yankee Composite Spectra; and
W = wind loads due to a 85 mph tornado or 110 mph straight wind.

Wind loads were applied in accordance with ANSI A58.1-1982. The controlling load case for design of the structure was Load Case 2 (seismic). The seismic load case would continue to control even if a 165 mph tornado was used for W. It should also be noted that ACI 349-76, "Code Requirements for Nuclear Safety-Related Concrete Structures," does not require the use of load factors for load combinations which include the design tornado, thus producing further margin. Therefore, the SSS Pump House will meet design code limits with margin when subjected to a 165 mph tornado and will, therefore, have an ultimate capacity considerably greater than 165 mph.

d. The Cable Spreading Room was modified on the same bases as the Upper Pipe Chase. The modified Cable Spreading Room structure meets the allowables of Section 11 of DCD 2648-6-1 (Reference 2-4) when subjected to a 10^{-5} (165 mph) tornado event.

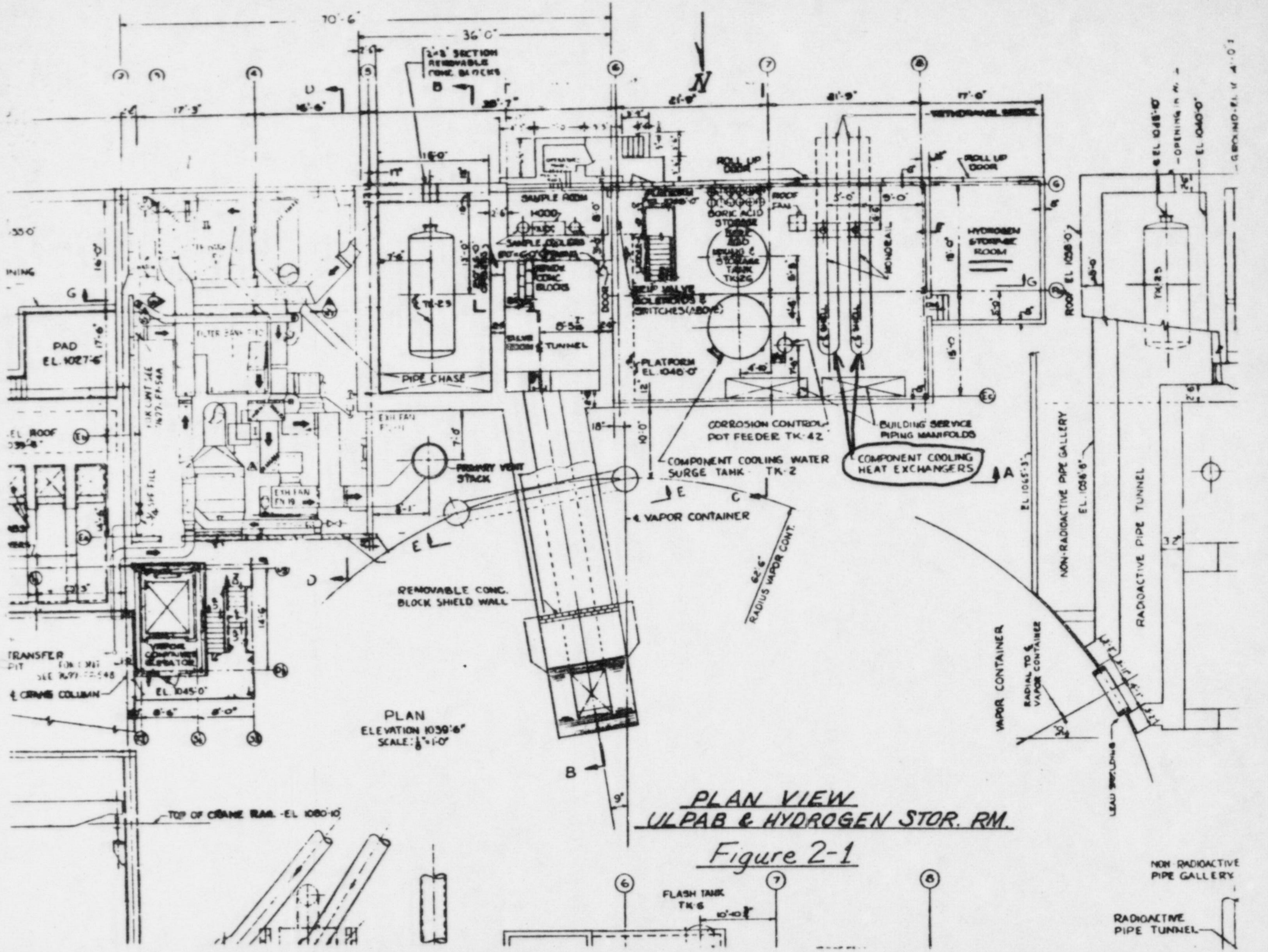
REFERENCES

2-1. "Tornado Cost-Benefit Analysis for Proposed Backfits at Yankee Nuclear Power Station," YAEC-1428, September 1984.

2-2. "Safety Evaluation Report Related to the Operation on Seabrook Station," USNRC, NUREG-0896, Supplement No. 3, July 1985.

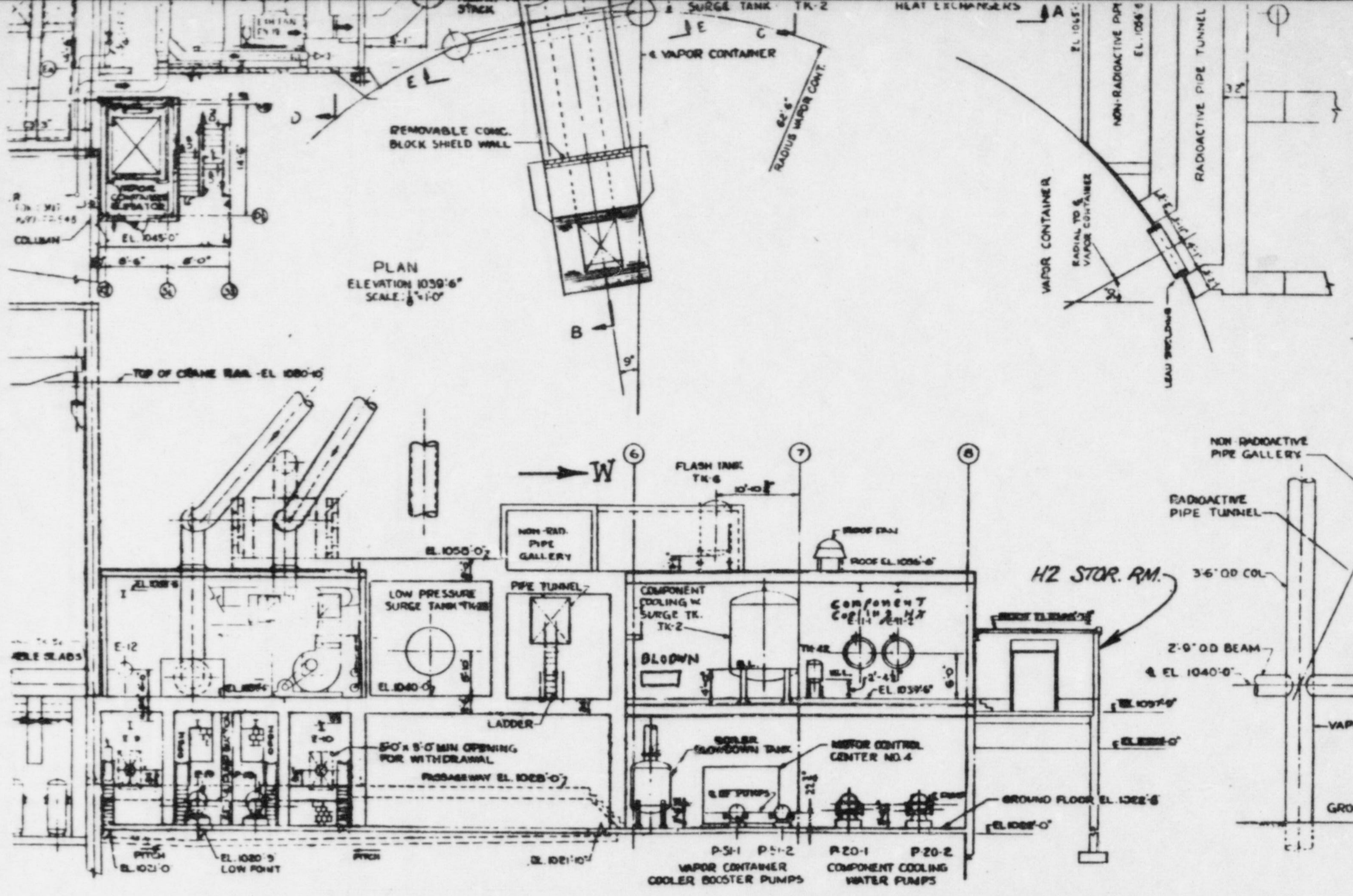
2-3 Letter, J. A. Kay (YAEC) to J. Zwolinski (NRC), February 13, 1985 (FYR 85-14), SEP Topics III-2 and III-4.A, Tornado Cost/Benefit Evaluation.

2-4 Chas. T. Main, Inc., Document No. DCD-2648-6-1, Revision 0, Structural Design Criteria for Evaluation and Modification of Existing Masonry Walls.



PLAN VIEW
ULPAB & HYDROGEN STOR. RM.

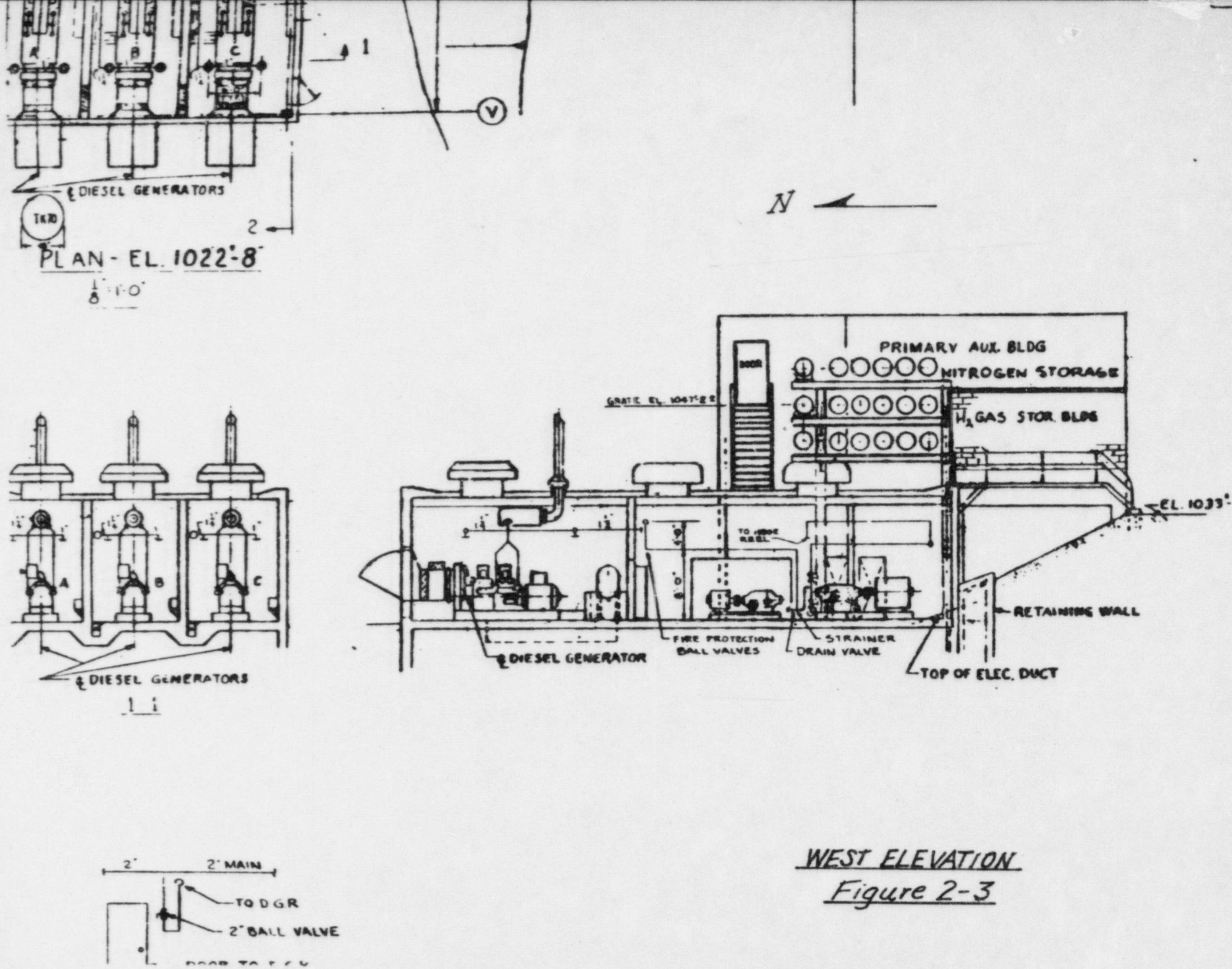
Figure 2-1



NORTH ELEVATION

Figure 2-2

ISSUE	DESCRIPTION					ISSUE	DESCRIPTION	BY	
9	REVISED PER EDCR 81-356	[illegible]	[illegible]	PAG					
8	AS BUILT PER EDCR 79-22	[illegible]	[illegible]	[illegible]		12	GENERAL UPDATE	[illegible]	
7	GENERAL UPDATE & REVISED TO AS BUILT CONDITIONS (EDCR 74-3)	E.V.		T.L.	[illegible]	11	REVISED PER EDCR 79-[illegible]	[illegible]	
6	AS BUILT CHANGES	CBS		DEV		10	REVISED PER EDCR 81-34	GC	
		CHKD.		APPR.	APPR.				

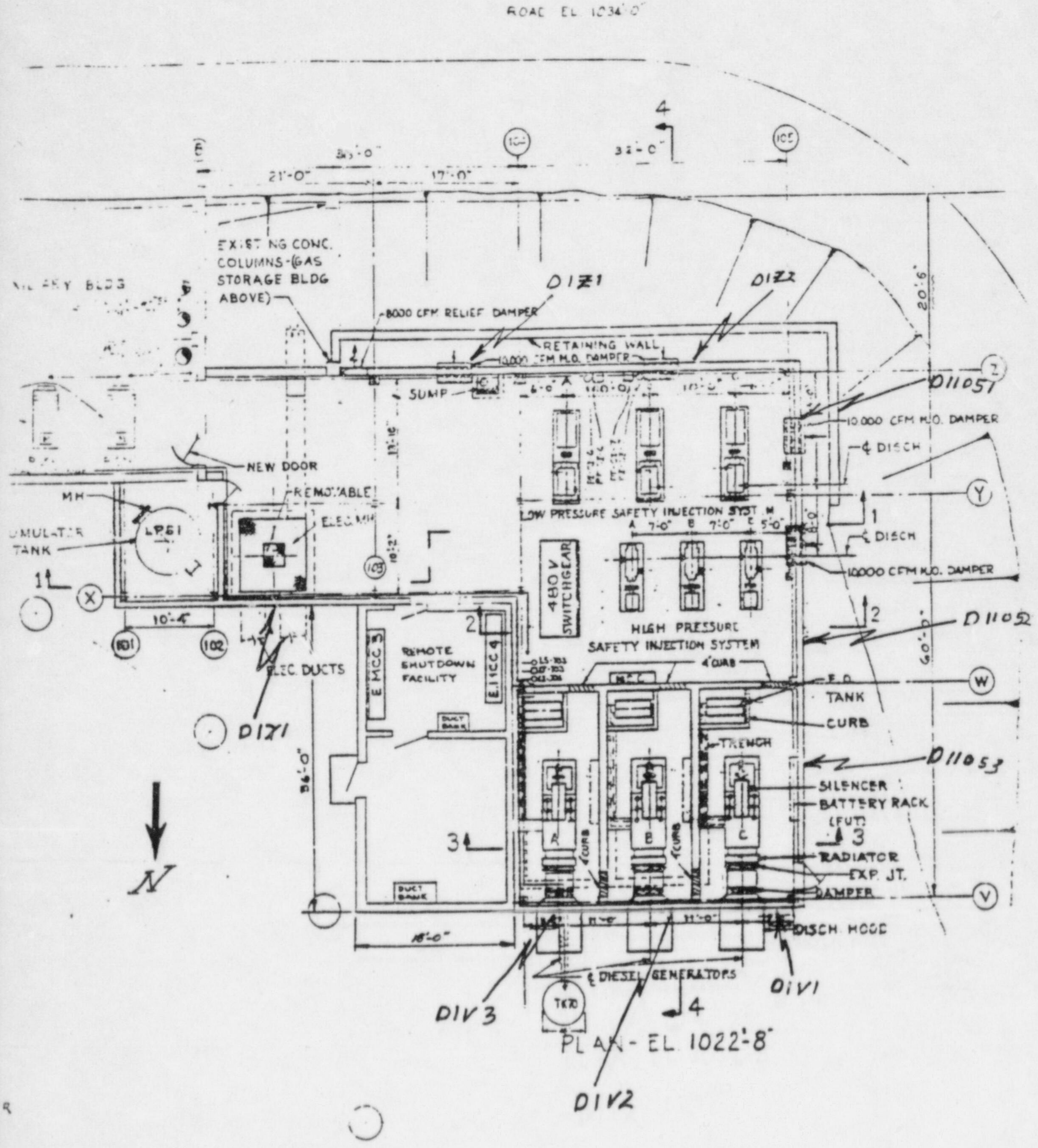


WEST ELEVATION

Figure 2-3

FIGURE 2-4

DGB - Schematic Plan View



PLAN - EL 1022'-8"

For the wind hazard (95% confidence level):

Speed Interval (MPH)	Point Frequency (95% Confidence)	Interval Frequency	Core Melt Probability	Interval Core Melt Frequency
69-91	$(3 \times 10^{-2}) - (1.8 \times 10^{-3})$	2.8×10^{-2}	$2.91 \times 10^{-\cancel{7}\,6}$	$8.1 \times 10^{-\cancel{9}\,8}$
91-103	$(1.8 \times 10^{-3}) - (3 \times 10^{-4})$	1.5×10^{-3}	$2.21 \times 10^{-\cancel{6}\,5}$	$3.3 \times 10^{-\cancel{9}\,8}$
103-122	$(3 \times 10^{-4}) - (2.5 \times 10^{-5})$	2.8×10^{-4}	$6.13 \times 10^{-\cancel{5}\,4}$	$1.7 \times 10^{-\cancel{8}\,7}$
122-135	$(2.5 \times 10^{-5}) - (2 \times 10^{-6})$	2.3×10^{-5}	$6.12 \times 10^{-\cancel{3}\,2}$	$1.4 \times 10^{-\cancel{7}\,6}$
135-145	$(2 \times 10^{-6}) - (2.5 \times 10^{-7})$	1.8×10^{-6}	$1.00 \times 10^{-\cancel{2}\,1}$	$1.8 \times 10^{-\cancel{8}\,7}$
145-163	$(2.5 \times 10^{-7}) - (10^{-8})$	1.5×10^{-7}	$1.00 \times 10^{-\cancel{2}\,1}$	$<1.5 \times 10^{-\cancel{9}\,8}$
163-	$(<10^{-8})$ -	$<10^{-8}$	1.00	$<10^{-8}$

So, for the wind hazard, at the 95% confidence level, the total core melt frequency due to feedwater failure (non-LOCA) is about 1.9×10^{-7}.

1.9×10^{-6}

For the wind hazard (50% confidence level):

Speed Interval (MPH)	Point Frequency (50% Confidence)	Interval Frequency	Core Melt Probability	Interval Core Melt Frequency
69-91	$(1 \times 10^{-2}) - (2 \times 10^{-4})$	9.8×10^{-3}	$2.91 \times 10^{-\cancel{7}\,6}$	$2.9 \times 10^{-\cancel{9}\,8}$
91-103	$(2 \times 10^{-4}) - (2.5 \times 10^{-5})$	1.8×10^{-4}	$2.21 \times 10^{-\cancel{6}\,5}$	$4.0 \times 10^{-\cancel{10}\,9}$
103-122	$(2.5 \times 10^{-5}) - (4 \times 10^{-7})$	2.5×10^{-5}	$6.13 \times 10^{-\cancel{5}\,4}$	$1.5 \times 10^{-\cancel{9}\,8}$
122-135	$(4 \times 10^{-7}) - (1 \times 10^{-8})$	3.9×10^{-7}	$6.12 \times 10^{-\cancel{3}\,2}$	$2.4 \times 10^{-\cancel{9}\,8}$
135-145	$(1 \times 10^{-8}) - (<10^{-8})$	ε	$1.00 \times 10^{-\cancel{2}\,1}$	$<10^{-\cancel{10}\,9}$
145-163	$(<10^{-8}) - (<10^{-8})$	ε	$1.00 \times 10^{-\cancel{2}\,1}$	$<10^{-\cancel{10}\,9}$
163-	$(<10^{-8})$ -	$<10^{-8}$	1.00	$<1 \times 10^{-8}$

Then, for the 50% confidence level wind hazard, the total core melt frequency due to feedwater failure (non-LOCA) is less than 10^{-8}.

$< 8.4 \times 10^{-8}$

For the tornado hazard (95% confidence level):

Speed Interval (MPH)	Point Frequency (95% Confidence)	Interval Frequency	Core Melt Probability	Interval Core Melt Frequency
65-93	$(1.7 \times 10^{-4}) - (8.5 \times 10^{-5})$	8.5×10^{-5}	$2.21 \times 10^{-\cancel{6}}$ 5	$1.9 \times 10^{-\cancel{10}}$ 9
93-120	$(8.5 \times 10^{-5}) - (4 \times 10^{-5})$	4.5×10^{-5}	$6.13 \times 10^{-\cancel{5}}$ 4	$2.8 \times 10^{-\cancel{9}}$ 8
120-156	$(4 \times 10^{-5}) - (1.6 \times 10^{-5})$	2.4×10^{-5}	$1.00 \times 10^{-\cancel{4}}$ 3	$2.4 \times 10^{-\cancel{9}}$ 8
156-162	$(1.6 \times 10^{-5}) - (1.1 \times 10^{-5})$	5.0×10^{-6}	$1.00 \times 10^{-\cancel{4}}$ 3	$5.0 \times 10^{-\cancel{10}}$ 9
162-176	$(1.1 \times 10^{-5}) - (7.0 \times 10^{-6})$	4.0×10^{-6}	$1.00 \times 10^{-\cancel{2}}$ 1	$4.0 \times 10^{-\cancel{8}}$ 7
176-	(7.0×10^{-6}) -	7.0×10^{-6}	1.00	7.0×10^{-6}

The total core melt frequency due to (non-LOCA) feedwater failure is 7.0×10^{-6} for the 95% confidence tornado hazard.
7.5×10^{-6}

And for the tornado hazard (50% confidence level):

Speed Interval (MPH)	Point Frequency (50% Confidence)	Interval Frequency	Core Melt Probability	Interval Core Melt Frequency
65-93	$(4.5 \times 10^{-5}) - (2 \times 10^{-5})$	2.5×10^{-5}	2.21×10^{-6}	$5.5 \times 10^{-\cancel{11}}$ 10
93-120	$(2 \times 10^{-5}) - (7 \times 10^{-6})$	1.3×10^{-5}	6.13×10^{-5}	$8.0 \times 10^{-\cancel{10}}$ 9
120-156	$(7 \times 10^{-6}) - (2.0 \times 10^{-6})$	5.0×10^{-6}	1.00×10^{-4}	$5.0 \times 10^{-\cancel{10}}$ 9
156-162	$(2.0 \times 10^{-6}) - (1.7 \times 10^{-6})$	3.0×10^{-7}	1.00×10^{-4}	$3.7 \times 10^{-\cancel{11}}$ 10
162-176	$(1.7 \times 10^{-6}) - (9.2 \times 10^{-7})$	7.8×10^{-7}	1.00×10^{-2}	$7.8 \times 10^{-\cancel{9}}$ 8
176-	(9.2×10^{-7}) -	9.2×10^{-7}	1.00	9.2×10^{-7}

Then, for the 50% confidence level tornado hazard, the total core melt frequency due to (non-LOCA) feedwater failure is 9.3×10^{-7}.
1.0×10^{-6}

To summarize the non-LOCA feedwater failure part:

	Core Melt Frequency	
Hazard	**50% Hazard Confidence**	**95% Hazard Confidence**
Wind	$<8.4 \times 10^{-8}$ $<10^{-8}$	1.9×10^{-6}
Tornado	1.0×10^{-6} ~~9.3×10^{-7}~~	7.5×10^{-6}

For wind at 95% hazard confidence level:

Speed Interval (MPH)	Point Frequency (95% Confidence)	Interval Frequency	Core Melt Probability	Interval Core Melt Frequency
69-91	$(3 \times 10^{-2}) - (1.8 \times 10^{-3})$	2.8×10^{-2}	~~7.24~~ 9.04 x 10^{-5}	2.~~0~~5 x 10^{-6}
91-103	$(1.8 \times 10^{-3}) - (3 \times 10^{-4})$	1.5×10^{-3}	~~4.84~~ 5.02 x 10^{-4}	7.~~3~~7 x 10^{-7}
103-	(3×10^{-4}) -	3.0×10^{-4}	2.0×10^{-2}	6.0×10^{-6}

The total annual core melt frequency due to relief valve LOCA due to a wind event (95% hazard confidence) is 8.7 x 10^{-6}.
9.3 x 10^{-6}

At 50% confidence for wind:

Speed Interval (MPH)	Point Frequency (50% Confidence)	Interval Frequency	Core Melt Probability	Interval Core Melt Frequency
69-91	$(1 \times 10^{-2}) - (2 \times 10^{-4})$	9.8×10^{-3}	~~7.24~~ 9.02 x 10^{-5}	~~7.1~~ 8.9 x 10^{-7}
91-103	$(2 \times 10^{-4}) - (2.5 \times 10^{-5})$	1.8×10^{-4}	~~4.84~~ 5.02 x 10^{-4}	~~8.7~~ 9.0 x 10^{-8}
103-	(2.5×10^{-5}) -	2.5×10^{-5}	2.00×10^{-2}	5.0×10^{-7}

Then, for the wind hazard, at the 50% confidence level, the total core melt frequency due to relief valve LOCA is 1.~~3~~5 x 10^{-6}.
1.5 x 10^{-6}

For tornados at 95% hazard confidence,

Speed Interval (MPH)	Point Frequency (95% Confidence)	Interval Frequency	Core Melt Probability	Interval Core Melt Frequency
65-93	$(1.7 \times 10^{-4}) - (8.5 \times 10^{-5})$	8.5×10^{-5}	~~4.8~~ 5.02 x 10^{-4}	4.~~1~~3 x 10^{-8}
93-	(8.5×10^{-5}) -	8.5×10^{-5}	2.00×10^{-2}	1.7×10^{-6}

For the tornado hazard, at the 95% confidence level, the total core melt frequency due to relief valve LOCA is 1.7 x 10^{-6}. ✓
No change

And finally for tornados with a 50% hazard frequency confidence,

Speed Interval (MPH)	Point Frequency (50% Confidence)	Interval Frequency	Core Melt Probability	Interval Core Melt Frequency
65-93	$(4.5 \times 10^{-5}) - (2 \times 10^{-5})$	2.5×10^{-5}	5.02 ~~4.84~~ $\times 10^{-4}$	1.3 ~~1.2~~ $\times 10^{-8}$
93-	(2×10^{-5}) -	2.0×10^{-5}	2.00×10^{-2}	4.0×10^{-7}

For the 50% hazard confidence tornado, the total core melt frequency due to relief valve LOCA is 4.1×10^{-7}. ✓

No change

To summarize the LOCA failure (base case):

	Core Melt Frequency	
Hazard	50% Hazard Confidence	95% Hazard Confidence
Wind	1.~~1~~5 $\times 10^{-6}$	9.3 ~~8.7~~ $\times 10^{-6}$
Tornado	4.1×10^{-7} ✓	1.7×10^{-6} ✓

CABLE TRAY HOUSE UPGRADE

The ultimate failure speeds of a Cable Tray House designed to 110 mph (10^{-4} annual frequency) are predicted to be 196 mph wind and 186 mph tornado.

From the hazard curve,

Speed MPH	Exceedance Frequency 50% Wind	50% Tornado	95% Wind	95% Tornado
186	--	6.5×10^{-7}	--	5.0×10^{-6}
196	$<10^{-8}$	--	$<10^{-8}$	--

A logic model review indicates clearly that the Cable Tray House does not impact non-LOCA feedwater but does affect relief valve LOCA as well as instrumentation.

Instrumentation, and the relief valve LOCA are considered below for the upgraded Cable Tray House.

For Instrumentation:

Conservatively, no credit is taken for batteries if there is no ac power available to charge them. Then instrumentation failure probability is 10^{-4} with no hazard failures; 10^{-2} when all ac power or the Cable Tray House are lost and 10^{-1} when the Safe Shutdown System is also lost. (See Sections 6.3 through 6.5).

For wind events, ac power is lost at 135 mph when the SI Building North Wall (SIBN) fails; the SSS can withstand at least 250 mph wind.

For tornados, ac power fails with the SIBN at 120 mph; the SSS fails at 178 mph.

So for wind at 95% confidence level:

Speed Interval (MPH)	Point Frequency (95% Confidence)	Interval Frequency	Core Melt Probability	Interval Core Melt Frequency
69-91	$(3 \times 10^{-2}) - (1.8 \times 10^{-3})$	2.8×10^{-2}	$10^{-\cancel{5}\,4}$	$2.8 \times 10^{-\cancel{7}\,6}$
91-135	$(1.8 \times 10^{-3}) - (2 \times 10^{-6})$	1.8×10^{-3}	$10^{-\cancel{4}\,3}$	$1.8 \times 10^{-\cancel{7}\,6}$
135-	(2×10^{-6}) -	2.0×10^{-6}	$10^{-\cancel{3}\,2}$	$2.0 \times 10^{-\cancel{9}\,8}$

For tornados at the 95% level:

Speed Interval (MPH)	Point Frequency (95% Confidence)	Interval Frequency	Core Melt Probability	Interval Core Melt Frequency
65-120	$(1.7 \times 10^{-4}) - (4.0 \times 10^{-5})$	1.3×10^{-4}	$10^{-\cancel{4}\,3}$	$1.3 \times 10^{-\cancel{8}\,7}$
120-178	$(4.0 \times 10^{-5}) - (7.0 \times 10^{-6})$	3.3×10^{-5}	$10^{-\cancel{3}\,2}$	$3.3 \times 10^{-\cancel{8}\,7}$
178-	(7.0×10^{-6}) -	7.0×10^{-6}	10^{-1}	7.0×10^{-7} ✓

Then, for the combined wind/tornado hazard at the 95% confidence level the total core melt frequency due to instrumentation failure, is 1.2 $\times 10^{-6}$, if the Cable Tray House is upgraded to 110 mph design.
5.8×10^{-6}

Now, for wind at the 50% level:

Speed Interval (MPH)	Point Frequency (50% Confidence)	Interval Frequency	Core Melt Probability	Interval Core Melt Frequency
69-91	$(1 \times 10^{-2}) - (2 \times 10^{-4})$	1.0×10^{-2}	$10^{-\cancel{5}}$ 4	$1.0 \times 10^{-\cancel{7}}$ 6
91-135	$(2 \times 10^{-4}) - (1 \times 10^{-8})$	2.0×10^{-4}	$10^{-\cancel{4}}$ 3	$2.0 \times 10^{-\cancel{8}}$ 7
135-	(1×10^{-8}) -	10^{-8}	$10^{-\cancel{3}}$ 2	$1.0 \times 10^{-\cancel{11}}$ 10

And for tornados at the 50% confidence level:

Speed Interval (MPH)	Point Frequency (50% Confidence)	Interval Frequency	Core Melt Probability	Interval Core Melt Frequency
65-120	$(4.5 \times 10^{-3}) - (7.0 \times 10^{-6})$	3.8×10^{-5}	$10^{-\cancel{4}}$ 3	$3.8 \times 10^{-\cancel{9}}$ 8
120-178	$(7.0 \times 10^{-6}) - (9.2 \times 10^{-7})$	6.1×10^{-6}	$10^{-\cancel{3}}$ 2	$6.1 \times 10^{-\cancel{9}}$ 8
178-	(9.2×10^{-7})	9.2×10^{-7}	10^{-1}	9.2×10^{-8} ✓

Then for the combined wind/tornado hazard at the 50% confidence level, the total core melt frequency due to instrumentation failure, is 2.2×10^{-7}, if the Cable Tray House is upgraded to 110 mph design.

1.4×10^{-6}

<u>For relief valve LOCA</u> with the Cable Tray House 110 mph design modification:

At the 95% hazard confidence for wind:

Speed Interval (MPH)	Point Frequency (95% Confidence)	Interval Frequency	Core Melt Probability	Interval Core Melt Frequency
77-91	$(1 \times 10^{-2}) - (1.8 \times 10^{-3})$	8.2×10^{-3}	4.53 $\cancel{4.40} \times 10^{-5}$	3.7×10^{-7} ✓
91-103	$(1.8 \times 10^{-3}) - (3 \times 10^{-4})$	1.5×10^{-3}	4.38 $\cancel{4.34} \times 10^{-4}$	$6.\cancel{5}$ 6 $\times 10^{-7}$
103-122	$(3 \times 10^{-4}) - (2.5 \times 10^{-5})$	2.8×10^{-4}	2.07×10^{-3} ✓	5.8×10^{-7}
122-135	$(2.5 \times 10^{-5}) - (2.0 \times 10^{-6})$	2.3×10^{-5}	2.07×10^{-3} ✓	4.8×10^{-8}
135-	(2.0×10^{-6}) -	2.0×10^{-6}	2.00×10^{-2} ✓	4.0×10^{-8}

For the wind hazard, at the 95% confidence level, the total core melt frequency due to relief valve LOCA excluding instrumentation failure is 1.7×10^{-6} ✓, if the Cable Tray House is upgraded to the 110 mph design.

No change

At the 95% confidence level for tornados:

Speed Interval (MPH)	Point Frequency (95% Confidence)	Interval Frequency	Core Melt Probability	Interval Core Melt Frequency
70-93	$(2.0 \times 10^{-4}) - (8.5 \times 10^{-5})$	1.2×10^{-4}	$4.3\cancel{4}8 \times 10^{-4}$	$5.\cancel{2}3 \times 10^{-8}$
93-120	$(8.5 \times 10^{-5}) - (4.0 \times 10^{-5})$	4.5×10^{-5}	2.07×10^{-3} ✓	9.3×10^{-8}
120-	(4.0×10^{-5}) -	4.0×10^{-5}	2.00×10^{-2} ✓	8.0×10^{-7}

For the tornado hazard, at the 95% confidence level, the total core melt frequency due to relief valve LOCA, excluding instrumentation failure is 9.5×10^{-7}, if the Cable Tray House is modified to 110 mph design.

✓ No change

At the 50% hazard level for wind:

Speed Interval (MPH)	Point Frequency (50% Confidence)	Interval Frequency	Core Melt Probability	Interval Core Melt Frequency
75-91	$(4 \times 10^{-3}) - (2.0 \times 10^{-4})$	3.8×10^{-3}	$4.\cancel{49}53 \times 10^{-5}$	1.7×10^{-7} ✓
91-103	$(2.0 \times 10^{-4}) - (2.5 \times 10^{-5})$	1.8×10^{-4}	$4.3\cancel{4}8 \times 10^{-4}$	$7.\cancel{8}9 \times 10^{-8}$
103-122	$(2.5 \times 10^{-5}) - (4.0 \times 10^{-7})$	2.5×10^{-5}	2.07×10^{-3} ✓	5.2×10^{-8}
122-135	$(4 \times 10^{-7}) - (1 \times 10^{-8})$	3.9×10^{-7}	2.07×10^{-3} ✓	8.1×10^{-10}
135-	(1×10^{-8}) -	1×10^{-8}	2.00×10^{-2} ✓	2.0×10^{-10}

For the wind hazard, at the 50% confidence level, the total core melt frequency due to relief valve LOCA, excluding instrumentation failure is 3.0×10^{-7} if the Cable Tray House is upgraded to the 110 mph design.

✓ No change

At the 50% level for tornado:

Speed Interval (MPH)	Point Frequency (50% Confidence)	Interval Frequency	Core Melt Probability	Interval Core Melt Frequency
70-93	$(4 \times 10^{-5}) - (2 \times 10^{-5})$	2.0×10^{-5}	$4.3\cancel{4}8 \times 10^{-4}$	$8.\cancel{7}8 \times 10^{-9}$
93-120	$(2 \times 10^{-5}) - (7 \times 10^{-6})$	1.3×10^{-5}	2.07×10^{-3} ✓	2.7×10^{-8}
120-	(7×10^{-6}) -	7.0×10^{-6}	2.00×10^{-2} ✓	1.4×10^{-7}

For the tornado hazard, at the 50% confidence level, the ~~total~~ core melt frequency due to relief valve LOCA, excluding instrumentation failure is 1.8×10^{-7}, if the Cable Tray house is modified to 110 mph design.

✓
No change

Not recalkulated since residual person-rem is small after CBT upgrade and since for this case, the SSS is worth less.

SSS UPGRADE

If the Safe Shutdown System design is upgraded to 165 mph design windspeed (10^{-5} event), its ultimate failure speeds will exceed 200 mph. In order to maximize benefit of this upgrade (conservative for cost benefit), it is assumed here that the upgraded system will withstand 250 mph. Note that the upgrade must include the Fire Water Storage Tank and the Upper Level Primary Auxiliary Building to be effective.

This upgrade has no effect on LOCA since for the "base case" plant, the SSS survives a stronger hazard than safety injection. Feedwater and instrumentation are both affected by an improved SSS capacity.

Feedwater

There is no change below 163 mph wind or 176 mph tornado, since for the "base case" plant, no damage to any SSS related location occurs below these speeds. Then, for wind 163 to 250 mph melt probability becomes 10^{-2} improving from 1.0; interval core melt frequency (95%) goes from less than 10^{-8} to less than 10^{-10}. For this case, total core melt frequency remains unchanged at 1.9×10^{-7}. Applying the same at the 50% level, the total core melt frequency again remains unchanged at less than 10^{-8}.

Now consider tornados for feedwater on the 95% confidence level. Above 176 mph the core melt probability changes from 1.0 to 10^{-2} so the interval core melt frequency improves to 7.0×10^{-8} from 7.0×10^{-6}. Total core melt frequency due to feedwater failure becomes 1.2×10^{-7} for the 95% confidence tornado hazard.

At the 50% level, interval melt frequency above 176 mph, becomes 9.2×10^{-9} resulting in a total frequency of 1.8×10^{-8} which is an improvement from 9.3×10^{-7}.

Instrumentation

Instrumentation failure probability is not significantly improved by this upgrade such that the value remains unchanged from the base case at 10^{-3}.

not recalculated

COMBINED UPGRADE

Finally, look at both upgrades together (110 mph Cable Tray House design and 165 mph Safe Shutdown System design).

Feedwater System results for the SSS upgrade are appropriate for this case; LOCA results for the Cable Tray House upgrade are also appropriate. Instrumentation for this combined upgrade is an improvement over the Cable tray case since here, for tornados, instrumentation never gets to 10^{-1}. (SSS does not fail below 250 mph). Then, for tornados at the 95% level, interval core melt frequency above 178 mph goes from 7.0×10^{-7} to 7.0×10^{-9} so for the combined wind/tornado hazard, total frequency due to instrumentation improves from 1.2×10^{-6} to 5.2×10^{-7}.

At the 50% confidence level, interval core melt frequency above 178 mph goes from 9.2×10^{-8} to 9.2×10^{-10} improving the combined wind/tornado total frequency of core melt due to instrumentation failure from 2.2×10^{-7} to 1.3×10^{-7}.

Then, to summarize event tree quantification results for each of the four cases,

For 95% hazard confidence:

	Core Melt Frequency Per Year Due To:		
Case	Instrumentation Wind+Tornado	Non-LOCA Wind/Tornado	Relief Valve LOCA Wind/Tornado
Base	3.0×10^{-5}	6 5 $1.9 \times 10^{-7}/7.0 \times 10^{-6}$	9.3 ~~8.7~~ $\times 10^{-6}/1.7 \times 10^{-6}$
Cable Tray House Upgrade	1.2×10^{-6}	6 5 $1.9 \times 10^{-7}/7.0 \times 10^{-6}$	$1.7 \times 10^{-6}/9.5 \times 10^{-7}$
SSS Upgrade	3.0×10^{-5}	$1.9 \times 10^{-7}/1.2 \times 10^{-7}$	$8.7 \times 10^{-6}/1.7 \times 10^{-6}$
Combined Upgrade	5.2×10^{-7}	$1.9 \times 10^{-7}/1.2 \times 10^{-7}$	$1.7 \times 10^{-6}/9.5 \times 10^{-7}$

For the 50% Hazard Confidence:

	Core Melt Frequency Per Year Due to:		
Case	Instrumentation Wind+Tornado	Non-LOCA Wind/Tornado	Relief Valve LOCA Wind/Tornado
Base	1.0×10^{-5}	$<8.4 \times 10^{-8}$ 1.0×10^{-6} ~~$<10^{-8}/9.3 \times 10^{-7}$~~	5 $1.3 \times 10^{-6}/4.1 \times 10^{-7}$
Cable Tray House Upgrade	2.2×10^{-7}	$<8.4 \times 10^{-8}$ 1.0×10^{-6} ~~$<10^{-8}/9.3 \times 10^{-7}$~~	$3.0 \times 10^{-7}/1.8 \times 10^{-7}$
SSS Upgrade	1.0×10^{-5}	$<10^{-8}/1.8 \times 10^{-8}$	$1.3 \times 10^{-6}/4.1 \times 10^{-7}$
Combined Upgrade	1.3×10^{-7}	$<10^{-8}/1.8 \times 10^{-8}$	$3.0 \times 10^{-7}/1.8 \times 10^{-7}$

The next section combines the above information to determine total core melt frequency.

6.6.3 Overall Results

The total annual core melt frequency due to the wind/tornado hazard is determined by simply adding the Event Tree results to the scram and instrumentation failure frequency.

Scram, from Section 6.4.2 has a failure probability of 1.0×10^{-5} which is constant over the hazard speed range and LOCA/non-LOCA events. The dominate event threshold frequencies are for wind, being 3×10^{-2} and 1×10^{-2} for 50% and 95% confidence levels, respectively. Then, core melt frequency due to scram failure is 3×10^{-7} (95%) or 1×10^{-7} (50%). Note that it is conservatively assumed that scram failure leads to core melt.

Instrumentation is as discussed in Sections 6.3 (base case) and 6.6.2 (upgrade cases).

Overall results are summed as follows:

For the base case and 95% hazard confidence:

Non-LOCA feedwater	-	wind	$1.9 \times 10^{-\cancel{7}\,6}$
	-	tornado	$7.\cancel{0}5 \times 10^{-6}$
Relief Valve LOCA	-	wind	9.3 $\cancel{8.7} \times 10^{-6}$
	-	tornado	1.7×10^{-6}
Scram			3.0×10^{-7}
Instrumentation			$3.0 \times 10^{-\cancel{5}\,4}$
Total annual core melt frequency			$\cancel{4.8 \times 10^{-5}}$ 3.2×10^{-4}

where instrumentation is clearly the major contributor here being 62.5% 93.8% of the total. Recall that Cable Tray house failure is the major contributor to instrumentation failure probability.

For the base case and 50% hazard confidence:

Non-LOCA feedwater	-	wind	~~$<10^{-8}$~~ $<8.4 \times 10^{-8}$
	-	tornado	~~9.3×10^{-7}~~ 1.0×10^{-6}
Relief Valve LOCA	-	wind	$1.\cancel{3}5 \times 10^{-6}$
	-	tornado	4.1×10^{-7}
Scram			1.0×10^{-7}
Instrumentation			$1.0 \times 10^{-\cancel{5}4}$
Total annual core melt frequency			~~1.3×10^{-5}~~ 1.0×10^{-4}

97%

where instrumentation contributes 77% at the 50% hazard confidence.

If the Cable Tray House is upgraded to 110 mph design, the overall results improve as follows.

For 95% hazard confidence:

Non-LOCA feedwater	-	wind	1.9×10^{-6}
	-	tornado	$7.\cancel{0}5 \times 10^{-6}$
Relief Valve LOCA	-	wind	1.7×10^{-6}
	-	tornado	9.5×10^{-7}
Scram			3.0×10^{-7}
Instrumentation			5.8 ~~1.2~~ $\times 10^{-6}$
Total annual core melt frequency			$1.\cancel{1}8 \times 10^{-5}$

32%

Instrumentation contributes ~~only~~ about 11% here with the main contributor being non-LOCA feedwater for the tornado hazard (64%).

42%

Considering the Cable Tray House upgrade at 50% hazard confidence level:

Non-LOCA feedwater	-	wind	$\cancel{<10^{-8}}$ $<8.4\times10^{-8}$
	-	tornado	$\cancel{9.3 \times 10^{-7}}$ 1.0×10^{-6}
Relief Valve LOCA	-	wind	3.0×10^{-7}
	-	tornado	1.8×10^{-7}
Scram			1.0×10^{-7}
Instrumentation			$\cancel{2.2 \times 10^{-7}}$ 1.4×10^{-6}

Total annual core melt frequency $\cancel{1.7 \times 10^{-6}}$ 3.1×10^{-6}

Instrumentation is about ~~13%~~ 45% of this total with non-LOCA feedwater for the tornado hazard being ~~almost 55%~~. 32%

For the case of the Safe Shutdown System upgrade (95% hazard confidence):

Non-LOCA feedwater	-	wind	1.9×10^{-7}
	-	tornado	1.2×10^{-7}
Relief Valve LOCA	-	wind	8.7×10^{-6}
	-	tornado	1.7×10^{-6}
Scram			3.0×10^{-7}
Instrumentation			3.0×10^{-5}

Total annual core melt frequency 4.1×10^{-5}

For the 50% hazard confidence with the Safe Shutdown System designed to 165 mph hazard:

Non-LOCA feedwater	-	wind	$<10^{-8}$
	-	tornado	1.8×10^{-8}
Relief Valve LOCA	-	wind	1.3×10^{-6}
	-	tornado	4.1×10^{-7}
Scram			1.0×10^{-7}
Instrumentation			1.0×10^{-5}

Total annual core melt frequency 1.2×10^{-5}

Finally, consider the combined modification case:

For the 95% hazard confidence case,

Non-LOCA feedwater	-	wind	1.9×10^{-7}
	-	tornado	1.2×10^{-7}
Relief Valve LOCA	-	wind	1.7×10^{-6}
	-	tornado	9.5×10^{-7}
Scram			3.0×10^{-7}
Instrumentation			5.2×10^{-7}
Total annual core melt frequency			3.8×10^{-6}

And for the 50% confidence combined upgrade:

Non-LOCA feedwater	-	wind	$<10^{-8}$
	-	tornado	1.8×10^{-8}
Relief Valve LOCA	-	wind	3.0×10^{-7}
	-	tornado	1.8×10^{-7}
Scram			1.0×10^{-7}
Instrumentation			1.3×10^{-7}
Total annual core melt frequency			7.3×10^{-7}

To summarize the overall core melt frequency results:

Total Core Melt Frequency Due to Wind/Tornado

Case	95% Confidence	50% Confidence
Base Case	3.2×10^{-4} ~~4.8×10^{-5}~~	1.0×10^{-4} ~~1.3×10^{-5}~~
Cable Tray House Upgrade	$1.\cancel{1}8 \times 10^{-5}$	3.1 ~~1.7~~ $\times 10^{-6}$
SSS Upgrade	4.1×10^{-5}	1.2×10^{-5}
Combined Upgrade	3.8×10^{-6}	7.3×10^{-7}

To put the potential modifications in some perspective, the following table presents each, in terms of reduction in core melt frequency, (at the 95% confidence level).

Improvement: From Plant Condition	Improvement: Proposed Upgrade	Melt Frequency Reduced: From	Melt Frequency Reduced: To	Annual Core Melt Frequency Reduction	
Base Case	Cable Tray	~~4.8 x 10^{-5}~~ 3.2x10^{-4}	$1.\cancel{1}8 \times 10^{-5}$	~~3.7 x 10^{-5}~~	3.0x10^{-4}
Base Case	SSS	4.8 x 10^{-5}	4.1 x 10^{-5}	0.7 x 10^{-5}	not recalculated
Base Case	Combined	4.8 x 10^{-5}	3.8 x 10^{-6}	4.4 x 10^{-5}	
Cable Tray	SSS*	1.1 x 10^{-5}	3.8 x 10^{-6}	0.7 x 10^{-5}	

*This case considers installation of the 10^{-5} SSS upgrade given that the 110 mph design Cable Tray House has been installed. The result, of course, is the combined upgrade.

The combined upgrade, of course, yields the maximum reduction in annual core melt frequency. It is important to note that 84% of this reduction can be accomplished by the Cable Tray House upgrade alone.

Core melt frequency alone is not necessarily a good indicator of plant risk and, therefore, cannot reliably indicate modification benefit. Further, any potential benefit must be weighed against its costs if an upgrade justification is to be valid. Section 9.0 provides further case comparison from a cost-benefit perspective.

6.7 Release Frequency

Section 6.6 determined annual core melt frequency for hazards up to 250 mph. The annual, release frequency for hazards up to 250 mph is the product of:

- Core melt frequency,
- Vessel failure probability given core melt, and
- Containment failure probability given core melt and vessel failure.

The probability of reactor vessel failure given core melt will, conservatively be taken as 1.0. Containment failure frequency given core melt and vessel failure was determined by the YNPS PSS. From Page 13-46 of that document:

5.27×10^{-2} "best estimate" (taken here as 50% confidence)
2.15×10^{-1} "baseline" (taken here as 95% confidence)

For hazard events greater than 250 mph, containment failure is expected and core melt is assumed. The frequency of release above 250 mph is simply taken as the hazard frequency at 250 mph. At this high speed, wind event frequency is negligible. Tornado frequency at 250 mph is 6×10^{-8} at 50% confidence and 4×10^{-7} at 95% hazard confidence.

Resulting annual release frequencies are as follows:

Case	Release Frequency < 250 MPH		Total Annual Release Frequency	
	95% Confidence	50% Confidence	95% Confidence	50% Confidence
Base Case	6.88 ~~1.03~~ $\times 10^{-5}$	5.27×10^{-6} ~~6.85×10^{-7}~~	6.92 ~~1.07~~ $\times 10^{-5}$	5.33×10^{-6} ~~7.45×10^{-7}~~
Cable Tray House Upgrade	3.87 ~~2.37~~ $\times 10^{-6}$	1.63×10^{-7} ~~8.96×10^{-8}~~	4.27 ~~2.77~~ $\times 10^{-6}$	2.23 ~~1.50~~ $\times 10^{-7}$
SSS Upgrade	8.82×10^{-6}	6.32×10^{-7}	9.22×10^{-6}	6.92×10^{-7}
Combined Upgrade	8.17×10^{-7}	3.85×10^{-8}	1.22×10^{-6}	9.85×10^{-8}

The consequences of a release are discussed in the next section. Section 8 combines the above release frequencies with the release consequences in order to assess plant risk.

TABLE 8-3

Person-Rem Exposure Development

Case Description		Core Melt and Release Frequency (yr^{-1}) < 250 MPH	Conditional Person-Rem Exposure <250 MPH	Person-Rem Exposure (yr^{-1}) < 250 MPH	Core Melt and Release Frequency (yr^{1}) >250 MPH	Conditional Person-Rem Exposure >250 MPH	Person-Rem Exposure (yr^{-1}) >250 MPH	Total Person-Rem Exposure (yr^{-1})
Base Case	50	~~6.85-7~~ 5.27-6	6.6+5	~~0.45~~ 3.48	6-8	1.3+6	7.8-2	~~0.53~~ 3.56
	95	~~1.03-5~~ 6.88-5	6.6+5	~~6.81~~ 45.41	4-7	1.3+6	0.52	~~7.33~~ 45.93
Cable Tray House Upgrade	50	~~8.96-8~~ 1.63-7	6.6+5	~~5.91-2~~ 0.11	6-8	1.3+6	7.8-2	~~0.14~~ 0.19
	95	~~2.37-6~~ 3.87-6	6.6+5	~~1.56~~ 2.55	4-7	1.3+6	0.52	~~2.08~~ 3.07
SSS Upgrade	50	6.32-7	6.6+5	0.42	6-8	1.3+6	7.8-2	0.50
	95	8.8-6	6.6+5	5.82	4-7	1.3+6	0.52	6.34
Combined Upgrade	50	3.85-8	6.6+5	2.54-2	6-8	1.3+6	7.8-2	0.10
	95	8.17-7	6.6+5	5.4-1	4-7	1.3+6	0.52	1.06

not recalculated

Enclosure 2
Attachment 2-1
Page 20 of 21

TABLE 9-1

Cost-Benefit Analysis Results

Description	Plant Capacity (1) (mph)	SSS DSN Capacity (mph)	SSS Actual Capacity (mph)	Hazard Curve (%)	Core Melt Freq. Per Year	Indiv. Risk Per Year	Societal Risk Per Year	Residual Person-Rem Per Year	Reduction in Person-Rem Per Year	(3) Just. Costs to Upgrade ($'s)	Actual Costs of Upgrade ($'s)	Ratio of Actual to Justif. Costs
Base Case	70(2)	--	175	50	1.0-4 ~~1.3-5~~	4.47-10	1.97-11	~~0.53~~ 3.56	--	--	-	-
				95	3.2-4 ~~4.8-5~~	6.43-9	2.7-10	~~7.33~~ 45.93	--	--	-	-
Cable Tray House Upgrade	160	--	175	50	3.1-6 ~~1.8-6~~	8.98-11	5.45-12	~~0.14~~ 0.19	~~0.39~~ 3.37	33.7 ~~3.9K~~	108K	~~28~~ 3.2
				95	1.8-5 ~~1.1-5~~	1.66-9	7.88-11	~~2.08~~ 3.07	~~5.25~~ 42.86	~~52.5K~~ 428.6	108K	~~2~~ 0.25
SSS Upgrade	70(2)	165	250	50	1.2-5	4.15-10	1.85-11	0.50	0.03	0.3K	296K	987
				95	4.1-5	5.53-9	2.34-10	6.34	0.99	9.9K	296K	30
Combined Upgrade	160	165	250	50	7.9-7	5.91-11	4.22-12	0.1	0.43	4.3K	404K	94
				95	4.2-6	7.3-10	4.16-11	1.06	6.27	62.7K	404K	6
Combined Upgrade Compared to Cable Tray House Upgrade	160	165	250	50	7.3-7	5.91-11	4.22-12	0.1	0.03	0.3K	296K	987
				95	4.2-6	7.3-10	4.16-11	1.06	1.02	10.2K	296K	29

(1) Excluding Safe Shutdown System.

(2) The Cable Tray House fails at ≃ 70 mph, this analysis assumes the Cable Tray House failure fails all normal plant instrumentation yielding a core melt probability of 10^{-1} above 70 mph since only local instrumentation is credited. This is extremely conservative for reasons stated in the analysis.

(3) Based on $1,000 per person-rem averted for 10 years or $10,000/person-rem.

Safe Shutdown System (SSS) Operation

If all attempts to restore feedwater to the steam generators fail, the operators are directed to establish feedwater from the Safe Shutdown System (SSS) to the steam generator blowdown header.

It has been established by draft procedure that three operators are required to operate the SSS; one to provide and control plant steam removal at the emergency Atmospheric Steam Dump (ASD) valves on the NRV platform, one to align the SSS feedwater path at the Upper Level Primary Auxiliary Building (ULPAB) blowdown feed header, and one at the SSS Building to start and run the system.

The contribution to system failure by the operator at the ASD valves is considered negligible since if he fails to open the correct valves, the steam generator safety valves will lift and remove steam. If he opens the valves fully and takes no further action to throttle them, the effect will be minimal since the valves are sized to limit steam flow within safe limits.

The contribution to system failure by the operator in the upper level PAB is set by the complexity of this task. Assuming he is at a moderate level of stress, and is well trained, there is some probability of his failing to correctly align the feed path. This is quantified separately. The operator must open the inlet valve, VD-V-1157, check the blowdown lines are isolated (they are normally closed), and check closed the SI/charging header cross-connect valve, SI-V-701.

The contribution to system failure by the operator in the SSS is determined by a review of his actions. He must start the SSS Diesel Generator (SSS-DG), open the normal 480 V ac supply to the SSS Support Systems, and close the SSS-DG output breaker and the breaker to the SSS-MCC. He then closes the breaker to the SSS Support System loads from the SSS diesel generator. He then energizes the SSS instrumentation and begins to monitor system parameters.

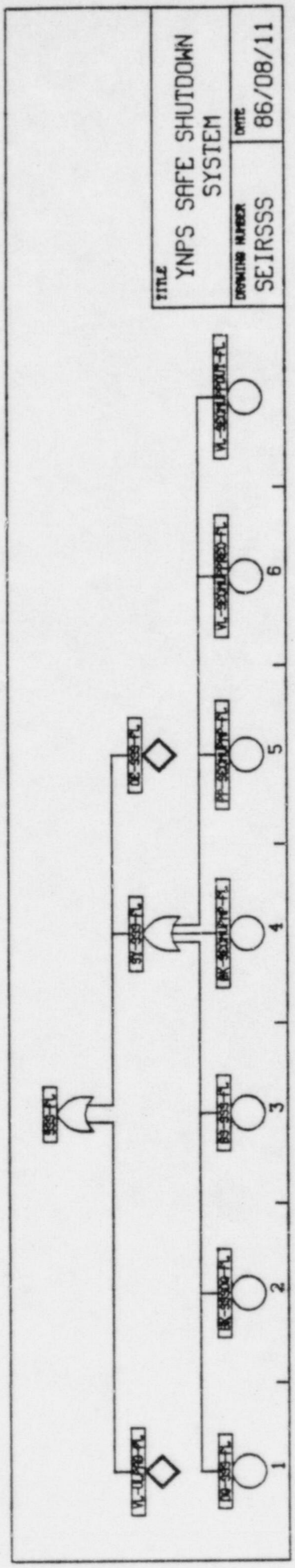
The operator, from the ULPAB (or, if he should not arrive, the SSS operator), then lines up the Secondary Make-Up Pump (SMUP) recirculation line by opening the recirculation valve and assuring that the discharge throttle valve is closed. He next fills the boron mix tank from the Fire Water Storage Tank (FWST). This step is not credited in the analysis and has no effect on the analysis. The operator then starts the SMUP and opens the SMUP discharge valve and throttles the SMUP recirculation valve to maintain or restore steam generator level while controlling Main Coolant System cooldown rate.

Local emergency atmospheric steam dump operator failure has a negligible effect on successful cooling since the discharge head of the SSS Secondary Make-Up Pump (SSS-SMUP) is greater than that of the steam generator safety valves. Therefore, steam removal is assured as long as feed is established.

The following errors of omission must be considered with respect to aligning the valves from the SSS to the steam generator blowdown header. The operator must gain access to the ULPAB and open the SSS to blowdown cross-connect valve, VD-V-1157. The remainder of his actions are checking other systems to assure other valves have been or are shut. The contribution to system failure from the valves, other than VD-V-1157, are negligible when compared to the human error of the operator failing to open the correct valve, VD-V-1157, under a moderately high stress situation. From NUREG/CR-1278, Table 20-7, the probability that the operator fails to open the correct valve is 3×10^{-3}. This is considered quite conservative since the operator is very much aware of the importance of opening this valve from the "System of Last Resort" and has been highly trained on the valves. However, moderately high stress when performing this action is assumed; from NUREG/CR-1278, Table 20-16, the operator error probability is increased by a factor of five. This raises the failure probability to 1.5×10^{-2}.

The SSS operator must start and continue to run the SSS-DG and correctly align power to the SSS-MCC and SMUP. Additionally, the operator must correctly align the SMUP to the steam generator blowdown line. The alignment may be performed by either the operator that aligned the system in the ULPAB or the SSS operator. Since there are two operators present in the building and a period of time exists to recover an incorrect startup, we will allow a factor of .15 for moderate dependence between operators from Table 20-21 of NUREG/CR-1278. The probability that the operators fail to align the system correctly is 3×10^{-3} from Table 20-7 of NUREG/CR-1278, yielding a failure probability of 4.5×10^{-4}. Finally, assuming moderately high stress (increased by a factor of five) results in 2.25×10^{-3}.

This results in a total operator error probability of $(1.5 \times 10^{-2}) + (2.25 \times 10^{-3}) = 1.73 \times 10^{-2}$.


TITLE
YNPS SAFE SHUTDOWN SYSTEM
DRAWING NUMBER
SEIRSSS
DATE
86/08/11
1
2
3
4
5
6

ENCLOSURE 3

Response to Enclosure 3 of Letter, E. McKenna (NRC) to G. Papanic, Jr. (YAEC), dated June 6, 1986 (NYR 86-119)

QUESTION 1

For all walls identified in References 2 and 3, provide the following information:

a. Number of reinforced and unreinforced walls.

b. For reinforced walls, indicate type and spacing of vertical and horizontal reinforcement. Verify that the reinforcement amount satisfies the minimum requirements of ACI 531-79 code.

RESPONSE

a. The number of reinforced and unreinforced walls identified in References 2 and 3 are indicated in Table 1 below. In addition, Table 1 provides supplemental information requested by the NRC to further clarify and categorize the identified masonry walls, and includes those masonry walls evaluated for the tornado cost-benefit evaluation.

   The "dominating" load (i.e., seismic or tornado) has not been included in the table for the following reasons. For a particular wall, a single loading may not produce the highest level for all types of stresses. For instance, tornado loading may produce the highest shear stress while seismic loads may produce the highest tensile stresses on a particular wall. Additionally, for a particular wall, one loading may control the design of the wall, while the other loading may control the design of modifications. See also the response to Question 4.

b. For the existing or modified reinforced masonry walls listed in Table 1, the vertical reinforcing required for load carrying capacity satisfies the minimum requirements of ACI 531-79.

TABLE 1

Summary of Masonry Wall Evaluations

Wall Designation	Wall Location	Existing Condition	Design Modification	Design Basis: Tornado Event	Design Basis: Ultimate Wind (psf)	Design Basis: Seismic Event	Wall Dimensions (Length x Height)
T1121	Turbine Bldg Stairwell Level 1	8" CMU	Structural Steel	10^{-4}(a)		YCS	9'-4" x 13'-0"
T1H2	Turbine Bldg Stairwell Level 1	8" CMU	None Required	10^{-4}(a)		YCS	13'-6" x 13'-0"
T2121	Turbine Bldg Stairwell Level 2	8" CMU	Structural Steel			YCS	9'4" x 13'-0"
T2H5	Turbine Bldg Stairwell Level 2	8" CMU	Structural Steel			YCS	13'-6" x 13'-0"
T3121	Turbine Bldg Stairwell Level 3	8" CMU	Structural Steel			YCS	18'-6" x 13'-6"
T3H4	Turbine Bldg Stairwell Level 3	8" CMU	Structural Steel			YCS	21'-2" x 13'-6"
T2H4	Turbine Bldg Battery Room Mezz Level	8" CMU	Grouted Reinforcing Steel	10^{-5}(b)		YCS	27'-0" x 11'-9"

Wall Designation	Wall Location	Existing Condition	Design Modification	Design Basis: Tornado Event	Design Basis: Ultimate Wind (psf)	Design Basis: Seismic Event	Wall Dimensions (Length x Height)
T2H3	Turbine Bldg Battery Room Mezz Level	8" CMU	Grouted Reinforcing Steel	10^{-5}(b)		YCS	7'-0" x 11'-9"
T2101	Turbine Bldg Battery Room Mezz Level	8" CMU	Grouted Reinforcing Steel	10^{-5}(b)		YCS	14'-0" x 7'-4"
T293	Turbine Bldg Battery Room Mezz Level	8" CMU	Grouted Reinforcing Steel	10^{-5}(b)		YCS	14'-0" x 7'-4"
T292	Turbine Bldg Battery Room Mezz Level	8" CMU	Grouted Reinforcing Steel	10^{-5}(b)		YCS	16'-0" x 13'-0"
T2G3	Turbine Bldg Battery Room Mezz Level	8" CMU	Grouted Reinforcing Steel	10^{-5}(b)		YCS	7'-0" x 7'-4"
T2G4	Turbine Bldg Battery Room Mezz Level	8" CMU	Grouted Reinforcing Steel	10^{-5}(b)		YCS	27'-0" x 7'-4"
T3G1 T3G2 T3G4	Turbine Bldg Control Room North Walls	8" CMU	Attached to Existing Reinforced Concrete Shield Wall			YCS YCS YCS	27'-0" x 13'-6" 22'-0" x 13'-6" 22'-0" x 13'-0"
T3122	Turbine Bldg Control Room West Wall	8" CMU	Remove and Replace With Steel Stud/ Gypsum Partition			YCS	16'-6" x 13'-6"

Wall Designation	Wall Location	Existing Condition	Design Modification	Design Basis: Tornado Event	Design Basis: Ultimate Wind (psf)	Design Basis: Seismic Event	Wall Dimensions (Length x Height)
P1E1	PA Bldg North Wall Lower Level	12" CMU	Structural Steel	10^{-5}		YCS	22'-9" x 16'8"
P1E2	PA Bldg North Wall Lower Level	12" CMU	Structural Steel	10^{-5}		YCS	22'-1" x 16'-8"
P2F1	PA Bldg North Wall Upper Level	8" CMU	Structural Steel	10^{-5}		YCS	22'-9" x 16'-8"
P2F2	PA Bldg North Wall Upper Level	8" CMU	Structural Steel	10^{-5}		YCS	22'-1" x 16'-8"
P363 P3E2 P371 P3F2 P361 P362	Upper Pipe Chase Walls	8" CMU Reinforced (c)	Structural Steel	10^{-5}		YCS	30'-0" x 8'-0" 19'-6" x 8'-8" 11'-4" x 8'-8" 32'-9" x 8'-8" 11'-4" x 8'-8" 31'-9" x 8'-0"

				Design Basis			
Wall Designation	Wall Location	Existing Condition	Design Modification	Tornado Event	Ultimate Wind (psf)	Seismic Event	Wall Dimensions (Length x Height)
T4J1 T4H1 T491 T4121	Cable Spreading Room Walls	8" CMU	Structural Steel	10^{-5}		YCS	67'-0" x 8'-5" 67'-0" x 8'-5" 15'-0" x 8'-5" 15'-0" x 8'-5"
T1J2	Turbine Bldg Aux Blr Room South Wall	12" CMU	None Required		22.8		22'-0" x 13'-4"
T1G2	Turbine Bldg Aux Blr Room Interior Wall	12" CMU	None Required		23.1		22'-0" x 13'-4"
D1Z1	Safety Inj Bldg-South Wall	8" CMU	None Required		9.7		17'-0" x 12'-4"
D1Z2	Safety Inj Bldg-South Wall	8" CMU	None Required		9.7		33'-2" x 12'-4"
D1X1	Safety Inj Bldg-North Wall	8" CMU	None Required		16.3		18'-0" x 12'-4"
D11051	Safety Inj Bldg-West Wall	8" CMU	None Required		10.4		15'-0" x 12'-4"
D11052	Safety Inj Bldg-North Wall	8" CMU	None Required		10.4		21'-8" x 12'-4"
D11053	Diesel Gen Bldg-West Wall	8" CMU	None Required		7.5		25'-8" x 12'-4"

Wall Designation	Wall Location	Existing Condition	Design Modification	Design Basis: Tornado Event	Design Basis: Ultimate Wind (psf)	Design Basis: Seismic Event	Wall Dimensions (Length x Height)
D1V1	Diesel Gen	8" CMU	None		26.9		11'-3" x 12'-4"
D1V2	Bldg-North		Required				11'-2" x 12'-4"
D1V3	Walls						11'-3" x 12'-4"

NOTES:

(a) Designed for 10^{-4} event tornado differential pressure equal to 24.5 psf.

(b) Designed for 10^{-5} event tornado differential pressure equal to 82.0 psf, applied to Walls T2G3, T2G4, and T292.

(c) Existing reinforcing is grouted No. 6 bars at 2'-8" o/c.

QUESTION 2

Clarify whether any multi-wythe walls exist. If so, provide and justify the allowable stress of the collar joint.

RESPONSE

No multi-wythe walls are present in the masonry walls evaluated. Walls evaluated are listed in Table 1.

QUESTION 3

In Section 1.3 of Reference 2, elaborate on the statement: "...removing deficient construction and replacing with suitably designed materials." If this is a deficiency in construction, identify the deficiency and justify the use of special inspection category for allowable stresses.

RESPONSE

The above statement uses the term "deficient construction" to categorize existing construction which cannot be reasonably modified to withstand current extreme environmental loadings. For example, Masonry Wall T3122 in the Control Room could not be reasonably reinforced for YCS seismic loading, and, therefore, was replaced with steel stud/gypsum partition to satisfy design seismic loading.

No deficiencies in the original construction of the block walls have been noted during field inspections. The field inspections found the existing masonry construction to be in very good condition.

QUESTION 4

Provide the technical basis to determine the governing load case (between seismic and tornado events).

RESPONSE

For the masonry walls evaluated for both seismic and tornado loadings (see Table 1), the stresses resulting from either loading were limited to the allowables in Section 11 of DCD-2648-6-1 (Reference 3-1).

Results of these evaluations are contained in the calculations made available for NRC review during the May meeting.

QUESTION 5

Indicate whether the modes of failure (besides tension) were investigated (i.e., compression, shear, boundary connections).

RESPONSE

For walls evaluated for design tornado and seismic events identified in Table 1, other modes of failure (compression, in-plane and out-of-plane shear, boundary connections) were evaluated in addition to mortar tensile failure. The results of the evaluations are contained in the applicable calculation sets which were available during the May meeting for NRC audit.

For determination of wall ultimate lateral loading for the tornado cost-benefit evaluation, it was prudently assumed, based on field review of wall geometry, that failure of the masonry was governed by mortar tensile failure normal to the bedding joint. Based on original construction specifications, ultimate mortar tensile values of 22.7 psi (normal to bed joint) and 45.7 psi (parallel to bed joint) were used. These values are conservative and consistent with "Recommended Guidelines for the Reassessment of Safety-Related Concrete Masonry Walls," prepared by owners and Engineering Firms Informal Group on Concrete Masonry Walls (October 1980). These values are 1.67 times the design tension allowables in ACI 531-79 (Revision 1981) for unreinforced masonry.

For designated walls, the ultimate lateral failure loading is summarized in Table 1. The calculations for these evaluations were available for NRC audit.

QUESTION 6

In Section 2.4 of Reference 3, elaborate on the statement "No evidence of vertical wall reinforcing was observed." Indicate whether vertical reinforcing was specified in the original design.

RESPONSE

The original Cable Spreading Room architectural design drawing by Stone & Webster Engineering Corporation does not indicate vertical reinforcing steel, nor was it required by the original design specification. As part of the field walkdown, an effort was made to determine if vertical reinforcing steel was installed in the original masonry construction. No evidence of vertical reinforcing was observed.

QUESTION 7

In Section 4.3 of Reference 3, the north wall frame was analyzed using an equivalent load of 100 psf. Indicate how this load was determined. Also, elaborate on the statement "This loading included the effects of an 8" thick reinforced concrete wall should it be installed at a future date."

RESPONSE

The north wall structural steel frame was designed to accommodate a potential future 8" concrete shield wall. The 100 psf equivalent lateral load is an approximation based upon the weight of the 8" shield wall and the response of the Primary Auxiliary Building to a YCS seismic event.

QUESTION 8

Provide all floor response spectra curves corresponding to walls identified in References 2 and 3.

RESPONSE

YCS floor response spectra for the walls identified in References 2 and 3 were made available for NRC audit during the May 1986 meeting.

QUESTION 9

Please make available all wall calculations for the May 20, 1986 meeting.

RESPONSE

Calculations for all walls listed in Table 1 were made available at the meeting for NRC audit.

REFERENCES

3-1 Chas. T. Main, Inc., Document No. DCD-2648-6-1, Structural Design Criteria for Evaluation of and Modification of Existing Masonry Block Walls, dated February 8, 1984.

ENCLOSURE 4

Response to Enclosure 4 of Letter, E. McKenna (NRC) to G. Papanic (YAEC), dated June 6, 1986 (NYR 86-119)

Response to Comments on Technical Background

In Table 8 of NBSIR 76-1050 (Reference 4-1), maximum speeds for a number of specified potential tornado-borne missiles are presented corresponding to a set of assumptions believed by the writers to be reasonable for design purposes. As noted in the NRC evaluation (Reference 4-2), the NBSIR 76-1050 authors acknowledge that higher missile speeds are conceivable; however, the NBSIR 76-1050 authors further note that it is their judgement that the probabilities of occurrence of such higher speeds for any given tornado strike are low. The authors further state that the assumptions used to estimate the speeds of Table 8 are sufficiently conservative for purposes of nuclear power plant design.

Based on the information presented in NBSIR 76-1050 and summarized above, it is concluded that maximum missile speeds given in Table 8 and Figure 3 of NBSIR 76-1050 are reasonable and appropriate.

Question 1

Provide technical basis for extrapolating the missile speed beyond the range of the tornado windspeed between 240 mph and 360 mph.

Response

In the tornado cost-benefit analysis the tornado windspeeds of interest are the 10^{-4} and 10^{-5} upper 95 percent confidence level values of 85 and 165 mph, respectively. Therefore, the tornado windspeeds of interest are below the 240 to 360 mph range for which maximum missile speeds are available. Also as specified by the NRC, the missiles of interest are limited to the utility pole and the steel rod.

Given missile speeds at some level of tornado windspeed, it is reasonable to assume that at lower tornado windspeeds, the maximum missile speeds will also be lower. This reduction is shown in Figure 3-1 of Reference 4-3 for the utility pole and steel rod data points labeled NBSIR. Given that the drag force on the missile is proportional to the square of the relative velocity and the reasonableness of the NBSIR 76-1050 Table 8 results, it is concluded that the extrapolation of missile speeds in Reference 3 for tornado windspeeds less than 240 mph is reasonable. The NBSIR 76-1050 authors characterize their Table 8 results as maximum missile speeds. The Figure 3-1 curves of Reference 4-3 labeled NBSIR represent an enveloping of these maximum missile speed data points. The basis for extrapolating the NBSIR data in Figure 3-1 is given in Section 3.1 of Reference 4-3.

Results of an additional analysis are presented here in Figure 4-1. From Figure 3 of Reference 4-1, maximum horizontal missile speeds as a function of C_DA/m for six levels of tornado windspeeds are available. Maximum horizontal missile speeds for the utility pole and steel rod were scaled from the figure. A least square curve fit program was used which fits the data to an equation of the form:

$$Y = a_0 + a_1X + a_2X^2$$

The least square polynomial curves along with the data points are shown in Figure 4-1. The coefficient of determination, R^2, for the curves are 0.96 and 0.97 for the utility pole and steel rod, respectively. These high R^2's indicate a good fit to the data. Extrapolation of missile speeds below the 240 mph tornado from the Figure 4-1 curves confirms the Reference 4-3 conclusions that neither the steel rod nor the utility pole would be potential airborne tornado missiles for tornado wind speeds associated with the 10^{-4} and 10^{-5} upper 95 percent confidence interval.

Question 2

Indicate whether the effects of the initial conditions were considered in the estimate of the missile speeds. Provide discussion on this subject.

Response

The missile speeds are based on information from Table 8 and Figure 3 of NBSIR 76-1050 (Reference 4-1). The authors of that report state that the assumptions used to estimate the speeds of Table 8 and Figure 3 are sufficiently conservative for purposes of nuclear power plant design. As discussed in NBSIR 76-1050, some initial conditions other than those used in deriving Table 8 and Figure 3 could conceivably produce higher missile speeds, but it is the author's judgement that the probabilities of occurrence of such higher speeds for any given tornado strike are low.

It is concluded that the initial conditions used in the development of the maximum missile speeds of Table 8 and Figure 3 are sufficiently conservative.

Question 3

Provide discussion regarding the effects of C_DA/m on the missile speed and explain how they were considered in your evaluation.

Response

As C_DA/m increases, missile speed also increases. This is clearly reasonable in that for low values of C_DA/m, the mass dominates and these relatively heavy, dense missiles do not attain appreciably high speeds. Examples of these missiles would be the steel rod and the utility pole. At higher values of C_DA/m, missiles with lower unit weight and relatively high area to mass ratios attain higher speeds; an example would be a wood plank.

Figure 3 in NBSIR 76-1050 shows the variation of maximum horizontal missile speed as a function of C_DA/m for various tornado wind levels. From Figure 3, the following observations can be made:

- For a given missile, (i.e., constant C_DA/m, missile speed decreases as tornado windspeed decreases.

- As the curves progress to lower tornado windspeeds, the curves increase in slope. This implies that as the tornado windspeeds decrease, the limiting C_DA/m value below which missiles are not potentially airborne increases. Also, the steepening of the curves is a result of a missile threshold effect. This threshold effect is shown by rapid change in missile speed over a small range of C_DA/m. Based on the trend shown in the figure, it is concluded that at the tornado speeds of interest, 85 and 165 mph, the steep portion of the curves would be shifted to the right of a C_DA/m value for a utility pole. Therefore, slight variation in the utility pole C_DA/m parameter would not change the conclusion given in the response to Question 1.

- The relatively low missile speeds to the left of the steep portion of the curves is a result of the initial elevation of the missile, 131 feet, and the imparting of some horizontal force on the missile as it falls under gravity to the ground.

REFERENCES

4-1 Tornado-Borne Missile Speeds, NBSIR 76-1050, National Bureau of Standards, April 1976
4-2 Letter, McKenna (NRC) to Papanic (YAEC), "Yankee Nuclear Power Station - NUREG-0825, Sections 4.5 Wind and Tornado Loadings, 4.8 Tornado Missiles and 4.11 Seismic Design Considerations; Enclosure 4 - Evaluation of Yankee Atomic Electric Company (YAEC) Report for Tornado Missiles," dated June 6, 1986
4-3 Cost-Benefit Evaluation for SEP Topic III-2, Wind and Tornado Loadings and SEP Topic III-4A, Tornado Missiles for the Yankee Nuclear Power Station, Revision 1, December 1984

46 0410

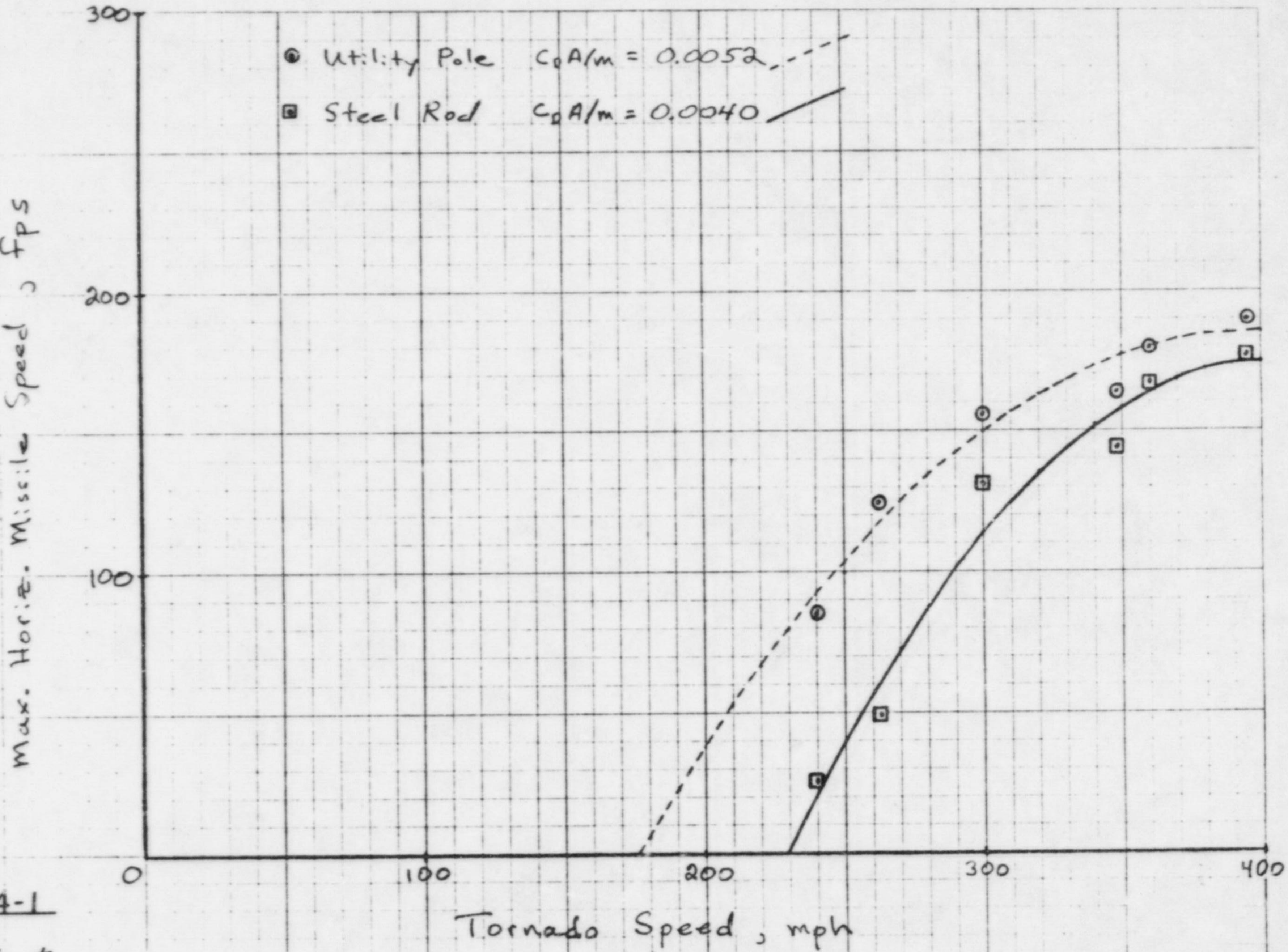


FIGURE 4-1

Yankee Plant
Tornado Missiles, 5/30/86
gal