TECHNICAL EVALUATION REPORT

WIND AND TORNADO LOADINGS (SEP, III-2)

CONSUMERS POWER COMPANY
BIG ROCK POINT PLANT

NRC DOCKET NO. 50-~~219~~ 155

NRC TAC NO. 41603

NRC CONTRACT NO. NRC-03-79-118

FRC PROJECT C5257

FRC ASSIGNMENT 14

FRC TASK 405

Prepared by

Franklin Research Center
20th and Race Street
Philadelphia, PA 19103

Prepared for

Nuclear Regulatory Commission
Washington, D.C. 20555

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Lead NRC Engineer: D. Persinko

September 21, 1982

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CONTENTS

FOREWORD

This Technical Evaluation Report was prepared by Franklin Research Center under a contract with the U.S. Nuclear Regulatory Commission (Office of Nuclear Reactor Regulation, Division of Operating Reactors) for technical assistance in support of NRC operating reactor licensing actions. The technical evaluation was conducted in accordance with criteria established by the NRC.

1. INTRODUCTION

1.1 PURPOSE OF REVIEW

In the Systematic Evaluation Program (SEP), licensees are required to establish the ability of Class I structures to safely withstand a high wind or tornado strike. After conducting an appropriate investigation, licensees report the conclusions in a safety analysis report (SAR). The purpose of the present review is to provide a technical evaluation of the SAR prepared by the Consumer Power Company (CPCo) for the Big Rock Point Nuclear Power Plant [1].

1.2 GENERIC ISSUE BACKGROUND

Some operating nuclear plants were designed on the basis of local building codes which did not consider the effects of the high wind speeds of tornadoes. Since the construction of these plants, research has led to an understanding of the various phenomena that occur during a tornado strike, and this knowledge has been incorporated into the definition of a design basis tornado (DBT) in Nuclear Regulatory Guide 1.76 [2]. Due to the concern regarding the extent to which older nuclear plants can satisfy DBT licensing criteria, the Nuclear Regulatory Commission (NRC), as part of the SEP, initiated Topic III-2, "Wind and Tornado Loadings," to investigate and assess the structural safety of existing designs against current requirements.

Licensees are required to prepare an SAR addressing the concern of SEP Topic III-2. The SAR should identify the limiting elements of the structural design and specify the loading conditions and threshold wind speeds at which buildings and components fail. As part of Assignment 14, Franklin Research Center (FRC) is assessing the adequacy and accuracy of the SARs. Typical items that are reviewed are the tornado load calculations and combinations, the structural acceptance criteria, and the method of analysis.

1.3 PLANT-SPECIFIC BACKGROUND

The review of the Big Rock Point plant was begun in July 1982. Prior to that time, Consumer's Power Company (CPCo) responded to NRC requests for information by providing architectural-engineering structural drawings. Additional sources of information were a CPCo letter on the SEP structural topics [3] and the plant final hazards summary report [4]. The reactor building, the screen house/discharge structure, the emergency diesel generator room, the 240-ft reinforced concrete stack, the condensate and demineralized water storage tanks, the solid radwaste storage vault, the turbine building, the service building, the turbine building passageway, and the fuel cask loading dock are the safety-related structures that have been evaluated by CPCo and reviewed in the Big Rock Point Plant SAR. The conclusions of this report are summarized in Table 1. The conclusions given in Table 1 for the Big Rock Point Nuclear Plant site are based on a DBT having the following characteristics:

1. Maximum wind speed of 250 mph (combined) rotational and translational speeds)
2. Maximum translational wind speed of 55 mph
3. Maximum pressure change of 1.35 psi.

The original design criteria for the Big Rock Point plant did not include tornado-induced loads. The wind loading criteria of the Big Rock Point structural systems are contained in the American Standard Association Code A58.1-1955 [5]. The graded wind loads of 25 psf at an elevation less than 30 ft, 30 psf at elevation between 30 ft and 49 ft, and 40 psf at elevation between 50 ft and 99 ft, and 45 psf at elevation between 100 ft and 499 ft, were used as lateral design loads at the Big Rock Point plant. The following shape factors were used for application of wind loads:

a. Flat vertical projection: 1.3 shape factor for projected area normal to wind
b. Cylindrical surfaces: 0.6 shape factor used
c. Spherical surfaces: 0.45 shape factor used.

In accordance with standard practice, seismic loads were not considered in a loading combination with wind loads. Allowable stresses, including soil pressures, were increased 33-1/3% when operational and dead loads are combined with wind loads. Also, crane loads were not combined with wind loads.

Table 1. Summary of Conclusions from Big Rock Point Topic III-2 SAR[1]

Structure	Element	Maximum Pressure (psi)	Maximum Wind Velocity (mph)
Reactor building	Steel spherical shell[3]	>1.35	>250
Screen house/ discharge structure	Roof decking	0.41	182
	Concrete walls	>1.35	152
Emergency diesel generator room	Roof decking	0.46	193
	Concrete walls	>1.35	212
240-ft stack	Concrete stack	NA	175
	Foundation	NA	>175
Condensate water storage tank	Tank	>1.35	>250
Demineralized water storage tank	Tank	>1.35	>250[2]
Solid radwaste storage vaults	Superstructure	0.17	100
	Original "low level" vault	1.04	>250
	Original "high level" vault	>1.35	>250
	New vault	>1.35	>250
Turbine building	South wall intermediate columns	0.17	110
	Crane columns and roof truss	0.21	121
	North and south wall bracing	NA	121
	Wall intermediate columns	0.22	125
	Metal siding	0.24	138
	Roof bracing	NA	140
	East and west wall bracing	NA	148
	Roof decking	0.49	198
	Roof purlins	0.82	>250

1. Provides nuts on anchor bolts are tightened.
2. The above analysis is based on a DBT with a maximum velocity of 250 mph and a maximum pressure change of 1.35 psi.
3. The steel spherical shell was originally designed for the pressure force resulting from a 100 mph wind and also for an internal pressure of 27 psig.

Table 1 (Cont.)

Structure	Element	Maximum Pressure (psi)	Maximum Wind Velocity (mph)
Service building	Safety-related block walls	0.03-0.16	NA
	Wall bracing column J	NA	123
	Exterior column	0.23	126
	Metal siding	0.24	138
	Girts	0.28	140
	Control room south wall	0.57	NA
	Roof decking	0.81	233
	Boiler stack	NA	>250
Turbine building passageway	Metal siding	0.24	138
	East and west wall column	0.36	159
	"Blowout" panel	0.50	NA
Fuel cask loading dock/core spray equipment room	Superstructure	NA	>250
	Block wall	0.03	NA

In the Big Rock Point SAR, the Licensee has provided a brief summary of calculations in the tornado structural analysis of the steel spherical shell, the screen house/discharge structure, the 240 ft reinforced concrete stack, and the sandwich wall siding. The Licensee has not provided calculations to support all of the conclusions reported in Table 1 of this report. Relevant parts of the Big Rock Point Nuclear Plant SAR are attached to this report in Appendix I.

Originally, FRC was charged only with auditing the design calculations supporting the conclusions of the Big Rock Point SAR. Since the calculations provided by the Licensee are not adequate, it was decided that, within the original budget and schedule constraints of Assignment 14 and subject to the limited amount of available information, FRC was to perform an independent tornado analysis for a limited sample of the Big Rock Point Class I structures and components. This analysis seeks to estimate the level of structural strength through approximate but conservative structural models (design review assumptions are stated in Sections 2 and 3 of this report). The results of this additional analysis are to be used to assess the conclusions reported in the SAR.

2. REVIEW CRITERIA

The intent of code regulations is to ensure the safety of systems vital to the safe shutdown of a reactor. The General Design Criteria (GDC) of 10CFR50, Appendix A [6] regulate the designs of these safety systems; in particular, GDC 2 requires that structures housing safety-related equipment be able to withstand the effects of natural phenomena such as tornadoes. The design basis must consider the most severe postulated tornado as well as the combined effects of tornado, normal, and accident conditions.

Regulatory Guide 1.76 defines a DBT in terms of the parameters of maximum wind speed, maximum differential pressure, rate of pressure drop, and core radius, given with respect to geographical location. The specified magnitudes of these regional parameters are the acceptable regulation levels, but additional analysis may be performed where appropriate to justify the selection of a less conservative DBT. In Reference 7, the NRC established the tornado parameters to be used in the SEP study of the Big Rock Point plant.

Regulatory Guide 1.117 [8] assists in the identification of structures and systems that should be protected from the effects of a DBT. This regulatory position is elaborated in the Standard Review Plan (SRP), Section 3.3.2 (NUREG-0800) [9]. The analysis presented in this report is of a representative sample of safety-related structural systems at the Big Rock Point plant.

With the dynamic pressure and air flow assumptions from the SRP, Section 3.3.2, and with the aid of Reference 10, a velocity-pressure distribution model can be constructed from the DBT characteristics. The actual forces acting on a structure can be calculated from this model augmented by the experimental data reported in References 11 and 12. These forces arise from wind-induced positive and negative pressures as well as from differential pressures.

An additional tornado load is the impact of wind-borne missiles against structures. The potential missiles are identified in the missile spectrum of the SRP, Section 3.5.1.4 [13], while the particular missiles to be included in this study were identified by the NRC as part of the SEP assignment [7].

References 14 and 15 assist in the determination of the structural effects of missile impact, while the guidelines of the SRP, Section 3.3.2 indicate acceptable combinations of impact effects with the loads resulting from wind and differential pressures.

Since the DBT is considered an extreme environmental event, tornado-induced loads are part of the loading combinations to be used in extreme environmental design (see SRP Section 3.8.4 [17]). The structural effects of these loading combinations are determined by analysis; stresses are calculated either by a working stress or ultimate strength method, whichever is appropriate for the structure under consideration. The ASME Code specifications for an extreme environmental event permit the application of reserve strength factors to allowable working stress design limits, and also permit local strength capacities to be exceeded by missile loadings (concentrated loads) provided that this causes no loss of function in any safety-related systems.

The sources of criteria described above and other source documents used in the evaluation are listed below:

NRC Regulatory Guide 1.76, "Design Basis Tornado for Nuclear Power Plants" [2]

NRC Regulatory Guide, 1.117, "Tornado Design Classification" [8]

NUREG-0800, Standard Review Plan

Section 3.3.2, "Tornado Loadings" [9]
Section 3.5.1.4, "Missiles Generated by Natural Phenomena" [13]
Section 3.5.3, "Barrier Design Procedures" [18]
Section 3.8.2, "Steel Containment" [19]
Section 3.8.4, "Other Seismic Category I Structures" [17]
Section 3.8.5, "Foundations" [20]

AISC Specification for Design, Fabrication and Erection of Structural Steel for Buildings [21]

ACI-318-77, "Building Code Requirements for Reinforced Concrete" [22]

ASME Boiler and Pressure Vessel Code, Section III, Division 1, Subsection NE, "Class MC Components," American Society of Mechanical Engineers [16]

NRC/SEB, "Criteria for Safety-Related Masonry Wall Evaluation," Structural Engineering Branch (1981) [23]

ACI-307-79, "Specification for the Design and Construction of Reinforced Concrete Chimneys" [24].

3. TECHNICAL EVALUATION

3.1 GENERAL INFORMATION

The structures included in this review are the spherical containment, the emergency generator room, the ventilation stack, and other structures reported in the CPCo SAR. These structures are classified seismically as Category I Nuclear Safety Related. The plan of the building arrangement of the Big Rock Point site is shown in Figure 1.

The DBT characteristics taken as a basis for analysis are as follows (unit abbreviations are from the SRP, Section 3.3.2):

Maximum wind speed	360 mph
Maximum pressure drop	3.0 psi
Rate of pressure drop	2.0 psi/sec
Core radius	150 ft.

These characteristics yield a dynamic pressure of 332 psf. For application of this pressure to external flat surfaces of structures, the shape coefficients are 0.80 for windward walls (positive pressure), 0.50 for leeward walls (suction), and 0.70 for roofs (suction). The shape coefficient for the cylindrical ventilation stack is 0.70. Gust factors for tornado loadings are taken as unity.

The design basis missiles are C and F from the Standard Review Plan, Section 3.5.1.4 missile spectrum.

Missile C: Steel rod: 1-in diameter, 3-ft length, 8-lb weight, 220 ft/sec velocity; strikes at all elevations.

Missile F: Utility pole: 13.5-in diameter, 35-ft length, 1490-lb weight, 147 ft/sec velocity; strikes in a zone limited to 30 ft above grade.

The full effects of a tornado are experienced by the main structural members only if the skin of the building (walls, panels, roof decks, etc.) can properly transmit the associated loadings. For the purpose of analysis, the most conservative circumstances of integrity or failure of these elements are assumed. For instance, a steel roof deck may fail when subjected to the DBT differential pressure. However, even though the roof deck failure provides

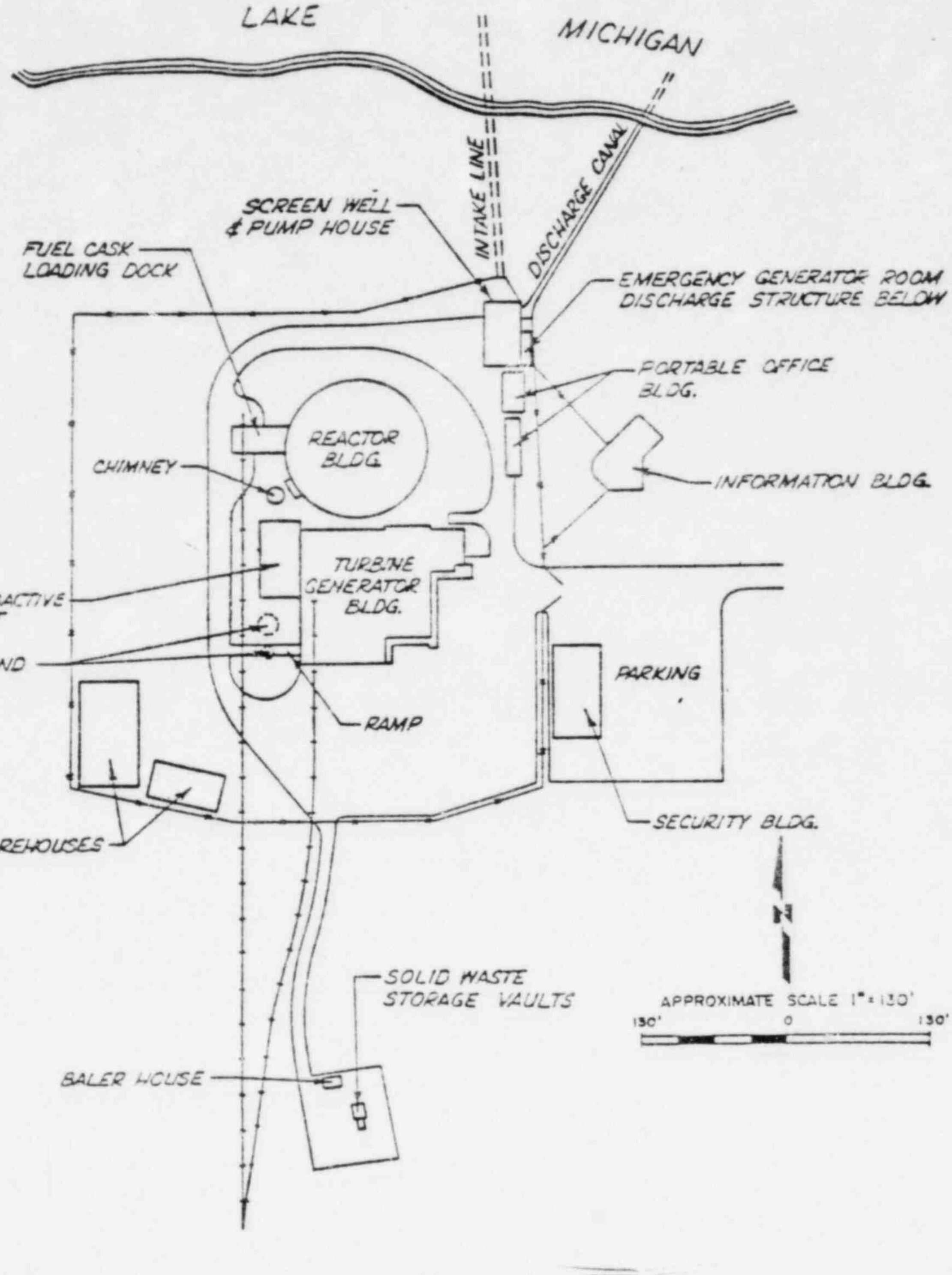

Figure 1. Site Plot Plan

venting, the tornado loads are still assumed to exist so that the strength of other, stronger structural elements can be analyzed.

For most structures, a wind flow field acting at an angle to the surfaces of a building is not as demanding as a frontal attack because the elements resisting lateral forces are oriented and framed so that the effects of adjacent wall loadings are uncoupled. Likewise, the action of windward face pressure and leeward face suction are uncoupled when their actions are resisted by separate structural elements. The most conservative loading cases are chosen accordingly.

The goal of analysis is to identify a structure's weakest members and to establish the threshold wind speed at which these members fail the structural acceptance criteria [17]. This wind speed limit rating depends on the postulated loading conditions. Once a limiting member is identified, the loading conditions used to determine subsequent limiting members are in some cases modified to account for failure of the weaker member. Therefore, conclusions about the strength of structural components are based on a supposition of sequential failure.

The following are typical assumptions for the structural modeling in this report:

1. No snow load exists during a tornado strike.
2. Thickened floor slabs can be used to transmit lateral loads.
3. Connections are designed in accordance with good engineering practice.
4. Unless noted otherwise, steel roof decking is assumed to remain intact.

Additional assumptions are identified on the calculation sheets (see appendices).

3.2 SPHERICAL CONTAINMENT VESSEL

3.2.1 Evaluation

The reactor building is housed in a 130-ft diameter steel sphere with a skin thickness varying from 0.702 in to 0.875 in. The sphere is embedded in and supported below grade by a concrete foundation. The reactor building is a concrete structure supporting the reactor and serving as a radiation shield.

This internal structure is self-supportive, resting on the foundation and separated from the containment vessel by a 6-in clearance. At grade level, the steel shell rests on a sand cushion. The cushion extends 8 ft below grade and is supported by the concrete foundation. The cushion serves to alleviate secondary stresses and the restraint of the foundation. The top of the containment vessel is at elevation 695 ft 6 in, while the adjacent grade is at elevation 592 ft 6 in.

During construction, the sphere was supported by 14 steel columns connected at the sphere equator (elevation 630 ft 6 in). These columns are no longer attached to the sphere.

The containment vessel is geometrically an axisymetric shell structure (radius-to-thickness ratio of 1000) with the axis of symmetry in the vertical direction. The wind pressure distribution on a totally exposed sphere varies from domes of pressure on the upstream and downstream faces centered on the shell equator to a torus of suction passing through the sphere poles and centered on a meridian perpendicular to the air flow. For an embedded sphere, the same pressure distribution is assumed to exist over the exposed surface, with this distribution being rotationally symmetric about an axis parallel to the air flow and grade line. For the geometry and pressure distribution described, the stresses are symmetric about a vertical plane centered on a meridian parallel to the air flow. The shear stresses and normal displacements along this plane are zero.

To estimate the level of shell stresses during a tornado strike, the shell is modeled as a structure resisting all loadings by in-plane forces only. Various integrations of the surface pressure are made to determine the lateral and uplift forces and the overturning moments. Meridional and hoop stresses are calculated through procedures based on membrane theory.

3.2.2 Conclusion

To examine the effects of high wind speeds on the stress levels in the containment sphere, the CPCo SAR modified the stresses based on a 100 mph wind by an overload factor. The results of a membrane analysis of the sphere based on the assumed pressure model indicate that the stresses predicted by the factor method for a 250-mph windspeed are conservative throughout the highly stressed portions of the shell.

3.3 EMERGENCY GENERATOR ROOM

3.3.1 Evaluation

The emergency generator room is located in the screen house/discharge structure. The walls and floors of the emergency generator room consist of reinforced concrete. The roof is metal-decking supported by structural steel beams. The steel beams are supported on reinforced concrete walls. The top of the roof steel is at elevation 603 ft with adjacent grade at elevation 588 ft 6 in.

The Licensee has submitted a brief summary of the calculations for the capacity of the roof deck, the reinforced concrete wall, and the structural steel roof framing of this structure. The roof deck was found to withstand an uplift pressure of 84.4 psf (corresponding to a tornado wind velocity of 182 mph), a more limiting component as compared to the capacity of the plug welds. The Licensee analyzed the structural steel roof framing and found that it can withstand pressure at least 1.5 times the failure pressure of the roof deck. In forming this conclusion, the Licensee has used an allowable bending stress of 0.90 of the yield stress.

The Licensee found that the west side reinforced concrete wall of the screen house/discharge structure governs the limiting wind velocity. The height-to-width ratio of 10-in-thick reinforced concrete west side wall is 0.23, so that for tornado dynamic pressure load, the Licensee analyzed the wall as a cantilever beam section supported at the base. But for the pressure drop case, the Licensee has assumed the walls supported by the roof deck and modeled it as simply supported at both ends. Moreover, the Licensee's analysis assumes that 1.3 times the pressure load (summation 0.8 times the pressure on windward side and 0.5 times the pressure of leeward side) can be resisted by the combined capacity of the windward and leeward walls. The Licensee has reported that the concrete wall off the emergency generator room can withstand a tornado dynamic pressure corresponding to a wind velocity of 212 mph and a pressure drop of 1.35 psi.

For this review, the allowable pressure that the roof decking can withstand is converted to wind velocities corresponding to the tornado dynamic pressure, differential pressure, and high wind dynamic pressure, as shown on page II-2 of Appendix II. The roof steel was analyzed for uplift pressures, assuming that the roof deck remains intact.

Similarly, for this review, only the windward side reinforced concrete wall is assumed to resist a bending moment due to 0.8 times the tornado dynamic pressure and the full differential pressure is resisted only by one cantilevered wall. The bending capacity of the reinforced concrete wall is converted to the allowble pressures and corresponding wind speeds on page II-1 of Appendix II.

3.3.2 Conclusion

The Licensee found the roof deck and the reinforced concrete wall to be the limiting members of this structure. In this review, it was found that the roof deck has a limit rating of 0.41 psi (108 mph) for differential pressure, 84.4 psf (182 mph) for tornado dynamic pressure, and 61 psf (156 mph) for high wind dynamic pressure (since roof deck details were unavailable, this report assumes no strength reduction due to buckling); the concrete wall has a limit rating of 0.27 psi (87 mph) for differential pressure, 48.2 psf (137 mph) for tornado dynamic pressure, and 28.4 psf (104 mph) for high wind dynamic pressure. The limiting member of the examined roof steel has limit ratings of 0.94 psi (162 mph) for differential pressure and 193 psf (274 mph) for tornado dynamic pressure. The conclusions reached in this review are compared with the Licensee's conclusions in Table 2. In that table, the difference in the strength ratings for the concrete wall are based on the structural modeling and not on the section capacity. (This review agrees with the Licensee's conclusion on the section capacity.)

3.4 REINFORCED CONCRETE STACK

3.4.1 Evaluation

The stack is a tall, slender, reinforced concrete chimney which is used to vent the air circulating through the turbine, service, and reactor buildings and the turbine generator off-gas system to the atmosphere. The stack is 240 ft high with adjacent grade at elevation 593 ft. The reinforced concrete chimney has a cylindrical cross section and tapers parabolically with increasing height.

As described in the D'Applonia Report [26], the foundation of the structure is a 4-ft-thick reinforced concrete mat that is octagonal in plan (the distance across the flats being approximately 34 ft). The mat is embedded to a depth of about 13 ft. The embedded portion of the stack immediately above the mat has an octagonal exterior and a circular interior, the internal radius being approximately 7.8 ft. The diameter at the top of the stack is about 3.8 ft.

The Licensee has performed the analysis of the 240 ft stack based on the provisions of ACI 307-69 [28] and ASCE Paper No. 3269 [11] with the following exception: instead of using the allowable stresses specified in the ACI code for normal wind loading, the Licensee has used an allowable maximum concrete stress of 0.8 f'c (3.2 times ACI allowable of 0.25 f'c) and an allowable tensile steel stress of 0.9 fy (2.88 times ACI allowable of 12,500 psi). The Licensee has used a shape factor of 0.55 instead of 0.7 as specified with height to diameter ratio between 7 and 25, but the ASCE Paper No. 3269 [11] allows using 0.55 as the shape factor for chimneys.

For the purpose of this report, an approximate profile of the stack was obtained by calculating inner and outer diameter from the information contained in Table E-1 and Figure E-5 of Reference 26. This can be found on pages II-3 through II-4 of Appendix II. Using the grade elevation as the base, an attempt was made to verify the moments and the e/r ratio reported by the Licensee in the SAR for a 43-psf wind load on the stack. These calculations can also be found on pages II-5 and II-7 of Appendix II.

3.4.2 Conclusion

The Licensee's conclusions stated in the SAR indicate that the stack can withstand tornado wind loads of up to 175 mph with its base being the most critical section. Based on the allowable soil pressures and the overturning moment provided in the SAR, the foundation will be adequate to withstand overturning under this load (overstress factor need not be applied to allowable soil pressure). The design procedures supporting the above conclusions are acceptable, as Maugh and Rumman [27] also suggest that, for the maximum stress condition, a tensile stress near the yield point of the steel can be permitted, and a compressive stress in the concrete of approximately 0.8 f'c, can be permitted provided the working stress design bending moments are multiplied by appropriate load factors. But since the tornado event is an extreme environmental condition, the bending moments for tornado loads without any load factors should be acceptable as design moments for the stack.

The actual moment calculations, thickness of concrete section, and the reference base of the stack should be checked to confirm the findings reported by the Licensee in the SAR. A judgment based on the audit calculations is that the actual strength of the stack is less than that reported in the SAR.

3.5 OTHER STRUCTURES

This section discusses the conclusions given in the SAR for the structures for which no calculations have been submitted by the Licensee in the SAR. It is based on the limited available information on these structures.

3.5.1 Solid Radwaste Storage Vault

The solid radwaste storage vault is located approximately 400 ft to the south of the turbine building. The vaults are primarily below grade with the bottom side of the radwaste vault floor at elevation 577.75 ft. The vaults have covers that vary in material and thickness from 12-in thick concrete for the original "low level waste vault," to 36-in thick for the original "high level waste vault," and 6-in thick steel for the new vault. The vault structures are enclosed in a steel frame building covered by structural aluminum siding. The Licensee has stated in the SAR, the design wind loading for the enclosure structure is 100 mph.

The Licensee evaluation concluded that the 36-in thick concrete cover and 6-in thick steel cover can withstand differential pressure loads greater than 1.35 psi, with maximum wind velocity of greater than 250 mph. But the 12-in thick concrete cover could be lifted and could potentially expose the contents to the environment when differential pressure is about 1.04 psi for a maximum wind velocity of greater than 250 mph.

For the purpose of this review, it was found that if the 6-in thick steel cover is not fastened to the vault, it can withstand an uplift differential pressure of 1.7 psi which corresponds to a wind speed of 219 mph as shown on page II-8 of Appendix II. The 12-in thick concrete cover can withstand uplift differential pressure of 1.04 psi, which corresponds to a wind speed of 171 mph as shown on page II-8 of Appendix II.

3.5.2 Service Building

The service building encompasses the control room, the switchgear/cable spreading room, and support facilities. The majority of the service building exterior walls are a combination of either unreinforced concrete masonry block or aluminum insulated siding. The roof consists of metal decking and built-up roofing supported on a structural steel frame [1].

The control room consists of reinforced concrete walls 4-ft 6-in thick on the north side, 3-ft-thick on the west side, and 1-ft thick on the east side. The south side wall is a partition wall with 1/2-in thick steel plate cover over the wall. The control room roof is a reinforced concrete slab which varies in thickness from 3 ft to 3 ft 6 in. The Licensee has concluded in the SAR that the south wall of the control room can withstand a differential pressure of 0.57 psi.

The service building also has numerous internal unreinforced concrete masonry block walls which have been found to vary in their capacity to withstand differential pressures of 0.03 psi to 0.16 psi. The Licensee also states in the SAR that the limiting element of the structural steel is the steel bracing on the east side, column line J, which has a capacity to withstand 123 mph wind speed.

In this review it was found that the column line J steel bracings on the east elevation can withstand a tornado dynamic pressure of 39.8 psf, which corresponds to tornado winds of 125 mph, and a high wind pressure of 24.9 psf, which corresponds to high wind speeds of 86 mph, as shown on pages II-9 to II-11 of Appendix II. The east side reinforced concrete wall of the control room has a limit rating of 2.6 psi (271 mph) for differential pressure.

3.5.3 Turbine Building Passageway

The turbine building passageway is the link between the reactor building and the turbine/service building. This structure consists of two levels: the ground level at elevation 593 ft and the second level at elevation 616 ft. The steam pipe tunnel located on the ground level consists of a 2 ft thick reinforced concrete roof slabs. A "blowout" panel is also located on the west exterior wall of the steam pipe tunnel. The penetration room, also located at ground level, shares common walls with the steam pipe tunnel on the west side, the electrical equipment room on the south side, and the steel spherical shell on the north side. The east side wall of the penetration room consists of insulated metal siding supported by a structural steel frame.

The second level provides personnel access to the reactor building and is enclosed by a structural steel frame with insulated metal siding and a built-up roof supported by metal decking. The reinforced concrete slab between the second floor and the penetration room is only 4-1/2-in thick.

The Licensee has reported in the SAR that the concrete walls around the steam pipe tunnel and the structural steel framing can safely withstand a design basis tornado with wind speeds of 250 mph and 1.35 psi differential pressure. But the metal siding can only withstand a pressure drop of 0.24 psi or wind velocities of up to 138 mph. The blowout panels in the steam tunnel concrete walls are designed to fail at a differential pressure of 0.5 psi.

In this review, the capacity of the siding was found to be 27.0 psf (103 mph) for tornado dynamic pressure, 0.15 psi (65 mph) for differential pressure, and 16.9 psf (65 mph) for high wind dynamic pressure (shown in pages II-13 to II-15 of Appendix II). The 8W13 roof element, loaded by uplift pressures, has limit ratings of 173 psf (259 mph) for tornado dynamic pressure and 0.84 psi (154 mph) for differential pressure.

4. CONCLUSIONS

In this section, the results obtained in this review are compared with those of the Licensee given in the SAR. Table 2 compares those results for which sufficient information was available.

Table 2. Comparison of Strength Summary of the Structural Components Given in SAR and this Review*

Structure Element	Results from SAR: Maximum Pressure	Results from SAR: Maximum Velocity	Results of this Review: Cause of Failure**	Results of this Review: Pressure	Results of this Review: Wind Speed
1. Spherical Containment Vessel					
a. Steel shell	>1.35 psi	>250 mph	The membrane stress model supports the Licensee's conclusions.		
2. Emergency Generator Room					
a. Roof decking	0.46 psi	193 mph	2 3 1	0.41 psi 60.6 psf 84.4 psf	108 mph 156 mph 182 mph
b. Concrete walls	>1.35 psi	212 mph	2 3 1	0.27 psi 28.4 psf 48.2 psf	87 mph 104 mph 137 mph
c. Roof steel	--	--	2 1	0.94 psi 193 psf	162 mph 274 mph

* The above comparison is based on the limited information available on various structural elements and also subject to assumptions given in Section 3 and Appendix II.

** Key: 1 = Tornado dynamic pressure; 2 = differential pressure; 3 = high wind dynamic pressure. Tangential wind speeds are listed for differential pressure failures.

Structure Element	Results from SAR Maximum Pressure	Maximum Velocity	Results of this Review Cause of Failure**	Pressure	Wind Speed
3. Reinforced Concrete Stack					
a. 240-ft concrete stack	NA	175 mph	0.9 fy stress in steel and 0.8 f'c stress in concrete are acceptable. There are some calculation errors; the value of angle α for given e/r does not seem correct.		
b. Foundation	NA	>175 mph			
4. Solid Radwaste Storage Vault					
a. Original "low level" vault 12-in thick concrete cover	1.04 psi	>250	2	1.04 psi	171 mph
b. New vault 6-in thick steel cover	1.35 psi	>250	2	1.7 psi	219 mph
5. Service Building					
a. Steel bracing along column line J	NA	123	3 1	24.9 psf 39.8 psf	86 mph 125 mph
b. East side reinforced concrete wall	--	--	2	2.6 psi	271 mph
6. Turbine Building Passageway					
a. Metal siding	0.24 psi	138	2 3 1	0.15 psi 16.9 psf 27.0 psf	65 mph 65 mph 103 mph
b. Roof steel	--	--	2 1	0.84 psi 173 psf	154 mph 259 mph

The above comparison shows that the wind speeds reported by the Licensee are not the same as those found in this review for some structures. In all cases, the capacity of the structural elements reported by the Licensee in the SAR was converted to equivalent wind speeds for different modes of failure.

5. REFERENCES

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APPENDIX I

BIG ROCK POINT SAR CALCULATIONS SUMMARY

Franklin Research Center
A Division of The Franklin Institute
The Benjamin Franklin Parkway, Phila., Pa. 19103 (215) 448-1000

APPENDIX A

CALCULATION SUMMARY FOR THE STEEL SPHERICAL SHELL

The reactor building was originally designed for a 100 mph wind and 27 psig of internal pressure.(A-1)

The following stress evaluation is based upon values obtained from the above reference except that wind stresses are proportioned to represent stresses induced by a 250 mph wind.

$(250 \text{ mph}/100 \text{ mph})^2 = 6.25$

therefore,

250 mph wind stresses = 6.25 x 100 mph maximum wind stresses

Stress Location (A-1)	DL* Dead Load Metal T_2	DL* Dead Load Metal T_1	DL* Dead Load Insulation T_2	DL* Dead Load Insulation T_1	W_T** 250 mph Wind T_2	W_T** 250 mph Wind T_1	$DL+LL+W_T$*** T_2	$DL+LL+W_T$*** T_1	Unit Stresses S_2 (psi)	Unit Stresses S_1 (psi)
8-7	-83	-77	-4	-4	+62	+62	-25	-19	-36	-27
7-6	-89	-46	-4	-2	+150	+162	+57	+114	+81	+162
6-5	-110	+28	-5	+1	+212	+275	+97	+304	+137	+429
5-4	-155	+143	-7	+7	+187	+375	+25	+525	+35	+730
4-3	-295	+368	-13	+18	-56	+568	-364	+954	-492	+1,289
Grade	-423	+538	-20	+20	-350	+812	-793	+1,370	-1,025	+1,170
4' below grade	-495	+611	-22	+22	-756	+756	-1,273	+1,389	-1,645	+1,795
8' below grade	-602	+729	-26	+26	-1,031	+1,031	-1,659	+1,786	-2,143	+2,307

(-) compression; (+) tension
*Values from Table II(a) (A-1)
**6.25 x values from Table I(a) (A-1)
***LL = live load is taken as zero (no snow loading)

CHECK BUCKLING

The critical buckling stress for a spherical segment.

$$S_{cr} = 0.35 \frac{\eta E t}{R} \qquad (A-2)$$

$$= \frac{0.35\ (1)(29 \times 10^3)(0.71)}{(65 \times 12)} = 9.24 \text{ ksi}$$

$$= 9,240 \text{ psi}$$

Allowable uniaxial compression stress

$$S_a = 1,800,000\ t^2/R = 1,800,000 \left\{\frac{(0.774)^2}{(65 \times 12)}\right\} = 1,382 \text{ psi} \qquad (A-1)$$

Under severe loading: $1.6 \times S_a = 1.6 \times 1,382 = 2,212$ psi (A-3)

$S_{cr} > 2,212$ psi $> 2,143$ psi [maximum compressive stress from table on previous page]

Subsequently, no buckling due to wind loading alone would be expected.

Tensile stresses are considered negligible by comparison to those calculated based upon the maximum design internal pressure.(A-1)

ADDITION OF TORNADO-GENERATED MISSILE LOADS TO TORNADO WIND LOADS

Approximate membrane stress due to wind load alone:

$$S_m = \left\{\frac{wR}{2t}\right\}$$

$$w = 0.00256(V)^2(C)(1ft^2/144\ in^2)$$

where

- V = Velocity of tornado (mph) = 250 mph
- C = Combination of drag and pressure distribution coefficients = 0.88 (conservative)(A-4, A-5)
- R = 65 ft x 12 in/ft = 780 in
- t = 0.71 in (minimum thickness)

Therefore, $S_m = 519$ psi for wind only

$$\left\{\frac{S_m}{S_{cr}}\right\} \times 100\% = 6\%$$

The reactor building's resistance to tornado-generated missiles is a function of the buckling strength of the steel spherical shell. Therefore, since the wind loading can only be attributed to 6% of the critical buckling load, the contribution of the wind to the evaluation of tornado missile loading is considered negligible.

<u>APPENDIX A</u>

<u>References</u>

A-1 "Structural Design of the 130-foot Diameter Spherical Containment Vessel for the Big Rock Point Nuclear Power Plant" by Chicago Bridge and Iron Company, CB&I Contract 8-0580, dated June 29, 1961

A-2 K.P. Buchert, "Buckling of Shell and Shell-like Structures" 1973

A-3 Standard Review Plan (U.S. NRC) Section 3.8.4, Other Seismic Category I Structures, Rev 1, July 1981

A-4 <u>Tornado and Extreme Wind Design Criteria</u>, Bechtel Topical Report, BC-TOP-3A, Rev 3,

A-5 "Wind Forces on Structures" ASCE Paper 3269, 1961

APPENDIX B

CALCULATION SUMMARY OF SCREEN HOUSE/DISCHARGE STRUCTURE

The screen house/discharge structure was analyzed for the effects of a tornado event. The analysis consisted of calculating the maximum internal and/or external pressure that a particular structural component could carry. These maximum velocity pressures are then expressed as maximum wind velocities.

1. ROOF DECK - UPLIFT

ALLOWABLE UPLIFT PRESSURE: P_u

$P_u = P_{nu} + \text{DL of roof}$

where

DL of roof = 9.0 psf

P_{nu} = net uplift = Lesser value of deck failure pressure or plug weld failure pressure

DECK FAILURE PRESSURE

$$P_{nu} = \left\{ \frac{0.9 \quad F_y \quad S \; (8)(12)(1,000)}{L^2} \right\} = \left\{ \frac{0.9(33.0)(0.1688)(9.6 \times 10^4)}{(8.167 \times 12)^2} \right\}$$

$= 50.1$ psf $= 0.35$ psi; governs

where

F_y = yield stress for ASTM A-446, grade A steel deck - ksi
S = minimum section modulus of 18 ga. Walcon-narrow rib deck - in^3
L = span between supports - in

PLUG WELD FAILURE PRESSURE

$$P_{nu} = \frac{1.6 \text{ (capacity of weld)}}{L \; (1.143)} = \frac{1.6 \; (299)}{8.167 \; (1.143)} = 51.2 \text{ psf} = 0.36 \text{ psi}$$

where

1.6 = increase in allowable load (B-3)
Capacity of Weld = load per plug weld in accordance with AISI Section 4.2.2 - lbs
L = span between supports - feet
1.143 = Reaction coefficient for a uniformly loaded multiple span beam

CORRESPONDING TORNADO WIND VELOCITY: V

$$V = \left[\frac{P_u}{0.00256\ C_p}\right]^{1/2} = \left[\frac{(50.1 + 9.0)}{0.00256\ (0.7)}\right]^{1/2} = 182 \text{ mph}$$

from $P = 0.00256(V)^2\ C_p$ (B-1)

where

C_p = external pressure coefficient = 0.7 for suction of flat roof in accordance with Reference B-1 Section 6.5.3.2.1

2. STRUCTURAL STEEL ROOF FRAMING

In accordance with Reference B-2 Section 3.6.5.2, the structural steel frame was analyzed for 1.5 times the failure pressure of the roof deck, but not to exceed the maximum tornado pressure loading.

Check $F_b \geq f_b = \left\{\frac{wL^2\ (12)}{(8)S}\right\}$

Where

w = 1.5 (50.1/1,000) x (Decking span) - k/ft
f_b = applied bending stress - ksi
F_b = 1.6 times allowable bending stress in accordance with Reference B-3 Section 1.5.1.4 - ksi

Note: Deflections were neglected in the analysis.

CHECK MEMBER, W10 x 21

F_b = 20.6 ksi
f_b = 10.2 ksi < 20.6 ksi

CHECK MEMBER, W18 x 50

F_b = 24.8 ksi

f_b = 20.1 ksi* < 24.8 ksi

*Bending stress due to loads imposed by W10 x 21

CHECK MEMBER, W8 x 17

F_b = 19.8 ksi
f_b = 11.5 ksi < 19.8 ksi

The structural steel roof frame is capable of supporting 1.5 times the ultimate load which the steel roof deck can carry.

3. ROOF HATCHES

As observed during a field walkdown, the roof hatches are held down by their deadweight only. (No hold down bolts in place)

$$\text{Uplift pressure} = \frac{\text{Hatch Weight}}{\text{Area of Hatch}}$$

UPLIFT ON 12'-3-1/2" x 6'-6-1/2" HATCH:

$$\text{Allowable Uplift} = \left\{\frac{1,507 \text{ lb}}{(12.29)(6.29)}\right\} = 19.49 \text{ psf} = 0.14 \text{ psi}$$

$$\text{Corresponding Velocity} = \left[\frac{19.49}{0.00256\ (0.7)}\right]^{1/2} = 104 \text{ mph}$$

UPLIFT ON 5'-9-1/2" x 5'-9-1/2" HATCH:

$$\text{Allowable Uplift} = \left\{\frac{694 \text{ lb}}{(5.79)^2}\right\} = 20.7 \text{ psf} = 0.14 \text{ psi}$$

$$\text{Corresponding Velocity} = \left[\frac{20.7}{0.00256\ (0.7)}\right]^{1/2} = 107 \text{ mph}$$

Hatches will remain in place during a 100-year reocurring wind, but not during a tornado event.

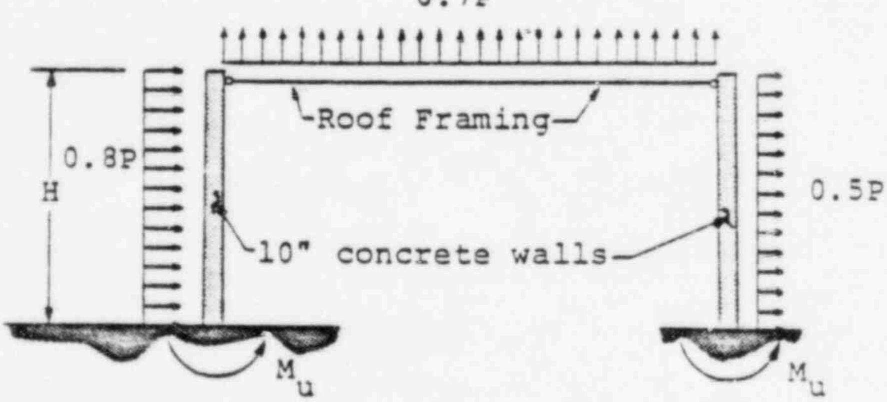
4. ANALYSIS OF EXTERIOR REINFORCED CONCRETE WALLS

The strength of the west wall of the screen house/diesel generator/discharge structure governs the limiting wind velocity.

The west wall has a height-to-width ratio of 0.23; therefore, the wall would not be expected to act as a plate. For analysis, a 1-foot wide vertical strip was checked.

CHECK ALLOWABLE WIND VELOCITY

Check the wall assuming the roof decking is still in place. In that case, the roof decking and roof cross bracing will distribute the horizontal load between the east and west walls (see sketch below).



P = wind pressure = $0.00256(V)^2$
M_u = ultimate moment capacity of a 1-foot wide section at the base of the wall.

From sketch → $2M_u = \{0.8P + 0.5P\}\{H^2/2\}$

Calculate M_u → Check 1-foot wide section
$F_y = 40.0$ ksi
$f'_c = 2.5$ ksi

Area steel (#4 at 15" c/c) = 0.16 in^2/ft = A_s
Wall thickness = 10"
Assume d = 8"

$$a = \left\{\frac{A_s F_y}{0.85 f'_c b}\right\} = \left\{\frac{0.16\ (40.0)}{0.85\ (2.5)(12)}\right\} = 0.25 \text{ in}$$

$$M_u = 0.9 A_s F_y [d-a/2] = 0.9(0.16)(40.0)\ 8-(0.25/2) = 45.4 \text{ in-k} = 3.78 \text{ ft-k}$$

CALCULATE PRESSURE = P

$$2 M_u = [0.8P + 0.5P][H^2/2]$$

$$P = \left\{\frac{2 M_u\ (2)}{H^2(1.3)}\right\} = \left\{\frac{2\ (3.78)(2)}{(14)^2(1.3)}\right\} = 0.0593 \text{ ksf} = 59.3 \text{ psf}$$

CALCULATE CORRESPONDING VELOCITY

$$\text{Velocity} = \left[\frac{P}{0.00256}\right]^{1/2} = \left[\frac{59.3}{0.00256}\right]^{1/2} = 152 \text{ mph}$$

The failure velocity of the wall is 152 mph, which is less than the failure velocity of the roof deck (182 mph). Therefore, the roof deck will distribute the load between the east and west walls as originally assumed.

CHECK ALLOWABLE INTERNAL PRESSURE

Assume the roof deck serves as a support for the top of the wall.

NOTE: Check 1-foot-wide strip; therefore, pressure (P) equals uniform load (w).

$$M_u = \left\{\frac{wL^2}{8}\right\}$$

Solving for P:

$$P = w = \left\{\frac{M_u\ (8)}{(L)^2}\right\} = \left\{\frac{3.78\ (8)}{(14)^2}\right\} = 0.154 \text{ ksf} = 1.07 \text{ psi}$$

NOTE: 1.07 psi is greater than the failure pressure of the roof deck and the roof hatches. Therefore, the building will be fully vented before the walls experience the full differential atmospheric bursting pressure.

APPENDIX B

References

B-1 American National Standard Building Code Requirements for Minimum Design Loads in Buildings and Other Structures, ANSI A58.1 1972, American National Standards Institute

B-2 Tornado and Extreme Wind Design Criteria, Bechtel Topical Report, BC-TOP-3-A, Rev 3

B-3 Standard Review Plant (U.S. NRC) Section 3.8.4, Other Seismic Category I Structures, Rev 1, July 1981

APPENDIX C

EVALUATION OF THE 240-FOOT-HIGH STACK

The analysis of the 240-foot-high stack was performed using ACI 307-69 and ASCE paper 3269 with the following exception: in lieu of using the allowable stresses identified in the ACI code for normal wind loading, the values for the extreme tornado wind loading were:

Maximum concrete stress = $0.8f'_c$ = 2,800 psi
Maximum reinforcing steel stress = $0.9f_y$ = 36,000 psi

These values are consistent with the values used by D'Appolonia in analyzing the 240-foot stack under extreme earthquake loading.(C-1)

The critical sections of the stack were determined by reviewing the original calculation and were found to be 56'-4" above the base for maximum reinforcing steel stresses and at the base for maximum concrete stresses.

The following outlines the method used in analyzing the stack when subjected to a 175 mph wind:

$$P = qC_D = .00256\ (175)^2\ (.55)$$

$$= 43\ \text{lbs/ft}^2$$

In determining the stresses at 56'-4" above the base, the following information was used:

Moment at 56'-4" above the base = 5,011 ft-kips
W = weight = 364 kips
n = 8
β = 0°
ρ = .0057
ρn = .0456
α = 55° (based upon e/r = 2.11)

Stress at mean diameter of chimney shell:

$$f'_{cw} = \frac{W(1-\cos\alpha)}{2rt\left[(1-\rho)(\sin\alpha-\alpha\cos\alpha)-\rho n\pi\cos\alpha\right]}$$

$$= 119.6\ \text{k/ft}^2 = 830\ \text{psi}$$

Stress at outside diameter of chimney shell:

$$f_{cw} = f'_{cw}\left[1 + \frac{t}{2r\ (1-\cos\alpha)}\right]$$

$$= 911\ \text{psi}$$

Maximum stress in vertical reinforcement

$$f_{sw} = nf'_{cw}\left[\frac{1+\cos\alpha}{1-\cos\alpha}\right]$$

$= 24,500$ psi

In determining the stresses at the base, the following information was used:

Moment = 10,242 ft-kips
W = weight = 597.2 kips
$n = 8$
$\beta = 19°$
$\rho = .0067$
$\rho n = .0536$
$\alpha = 63°$ (based upon $e/r = 2.06$)

Stress at mean diameter of chimney shell:

$$f_{cw}' = \frac{W(\cos\beta - \cos\alpha)}{2rt\left[(1-\rho)(\sin\alpha - \alpha\cos\alpha) - (1-\rho+\rho n)(\sin\beta - \beta\cos\alpha) - \rho n\pi\cos\alpha\right]}$$

$= 202.5\text{k/ft}^2 = 1,406$ psi

Stress at outside diameter of chimney shell:

$$f_{cw} = f_{cw}'\left[1 + \frac{t}{2r\cos\beta(\cos\beta - \cos\alpha)}\right]$$

$= 1,528$ psi

Maximum stress in vertical reinforcement:

$$f_{sw} = nf'_{cw}\left[\frac{1+\cos\alpha}{\cos\beta - \cos\alpha}\right]$$

$= 33,300$ psi

When the stack is subjected to the 180 mph wind, the following stress result at the base:

$f'_{cw} = 1,528$ psi
$f_{cw} = 1,665$ psi
$f_{sw} = 37,770$ psi - exceeds allowable stress

The foundation was checked for 175 mph wind as follows:

Total load on the soil (including stack, foundation, soil, and buoyan
= 1,970 kips

Moment at the bottom of the foundation = 11,887 ft-kips

Since $\frac{M}{W} = \frac{11,887}{1,970} = 6.03$ ft is larger than 1/6 of the base, dimension (5.72 ft); full surface contact does not exist.

Maximum soil pressure is determined by: (C-2)

$$p = \left\{\frac{2W}{3a\ (b/2-e)}\right\} = 4.876\ k/ft^2$$

Where

p = soil pressure
W = weight = 1,970 kips
a = width
b = length
e = M/W

The maximum allowable soil pressure for loadings including wind or seismic, in accordance with the original design criteria (C-3) is $5\ k/ft^2 \times 1.33 = 6.65\ k/ft^2$, which is greater than the calculated maximum soil pressure.

APPENDIX C

References

C-1 D'Appolonia Report Volume IV Appendix E "Seismic Safety Margin Evaluation, Reinforced Concrete Stack"

C-2 F.S. Merritt, Standard Handbook for Civil Engineers, 2nd edition

C-3 Design Criteria, Big Rock Point Plant - Consumers Power Company, Job 3159 (Civil, Structural, Architectural)

APPENDIX D

CALCULATION OF ALLOWABLE PRESSURES ON SANDWICH WALL SIDING

The turbine building, service building, and electrical penetration room are all sided with double-panel sandwich-wall insulated panels. The following calculation was used to determine the allowable pressure due to wind loading as well as differential atmospheric pressures. Corresponding wind velocities were also defined.

SANDWICH WALL SIDING CONSISTS OF:

0.032-inch thick x 2.67-inch x 7/8-inch deep corrugated aluminum interior panel, 1-inch thick PF615 Owens-Corning Fiberglass Insulation, and 0.032-inch thick ALCOA 4-inch ribbed aluminum exterior panel.

NOTE: Subgirts not used in wall construction

The following are the four possible loadings on the walls and how they affect the panels:

1. Positive exterior wind pressure: 100% of applied pressure taken by exterior panel since insulation between panels does not transfer load (windward side).

2. Negative exterior wind pressure: Exterior sheet resists 100% of applied pressure (leeward).

3. Interior bursting pressure: Pressure dependent on fasteners.

4. Negative exterior side wall pressure: Same as 2.

NOTES:

1. The allowable pressures were determined using strength of material methods.

2. Use 0.9 x (yield stress) for allowable stress.

CALCULATION OF ALLOWABLE PRESSURE ON INDIVIDUAL PANELS

4 INCH RIB - EXTERIOR PANEL $S_{min} = 0.160 in^3$

$$F_b = \left\{\frac{wL^2\ (12)}{(8)\ S}\right\}$$

therefore,

$$w = \left\{\frac{F_b\ (8)\ S}{(12)\ (L)^2}\right\} \text{ For 1-foot-wide strip}$$

$$= \left\{\frac{(25.2\text{ ksi})\ (8)\ (0.160\text{in}^3)}{(12)\ (7.5833')^2}\right\} = 0.047\text{k/ft/ft}$$

$$= 47\text{ lb/ft}^2 \text{ (maximum pressure)}$$

where

$F_b = 0.9\ (F_{cy}) = 0.9\ (28) = 25.2$ ksi
F_{cy} = Compressive yield stress - ksi

CORRUGATED SIDING - INTERIOR PANEL $S_{min} = 0.0936\ \text{in}^3$

$$w = \frac{(25.2)\ (8)\ (0.0936)}{(12)\ (7.5833')^2} = 0.027\text{k/ft/ft}$$

$$= 27\text{ lb/ft}^2 \text{ (maximum pressure)}$$

CALCULATION OF ALLOWABLE PRESSURE FOR CASE 1:

Exterior panel resists 100% of load.

Maximum allowable pressure = 47 psf (from above)

Corresponding Wind Velocity $= \left[\frac{47}{0.00256\ (0.8)}\right]^{1/2} = 151$ mph

CALCULATION OF ALLOWABLE PRESSURE FOR CASES 2 AND 4:

100% of load on exterior panel.

The allowable leeward or sidewall velocity pressure = 47 lb/ft^2
= 0.33 psi

Corresponding leeward wind velocity $= \left[\frac{47}{0.00256\ (0.5)}\right]^{1/2} = 191$ mph

and

Corresponding sidewall wind velocity $= \left[\frac{47}{0.00256\ (0.7)}\right]^{1/2} = 162$ mph

CHECK FASTENER LOADING

FASTENER = Self-tapping screw: No. 14 x 3-1/2 inch long, hex head Type B, Stainless Steel Type 305, with 0.050" x 0.625" O.D. aluminum and neoprene washer.

Allowable load/fastener = $0.17\ t\ F_{ty}$ Reference D-1 Section 5.3.3

$t = 0.032$ in

$F_{ty} = 30.0$ ksi = yield stress of siding Table 7.22. (D-2)

NOTE: Fastener capacity is governed by "pull-over" failure.

Allowable load/fastener = 0.17 (0.032) (30.0) = 163 lb

$$\text{Allowable pressure} = \frac{[\text{allowable load (lb)}]\ (1.33)\ (12)}{[\text{fastener spacing (in)}]\ (1.25)\ [\text{span (ft)}]}$$

$$= \frac{(163)\ (1.33)\ (12)}{(8")\ (1.25)\ (7.5833')} = 34.3 \text{ psf} = 0.24 \text{ psi}$$

where

1.33 = load increase factor for wind loading
1.25 = reaction coefficient

$$\text{Corresponding leeward velocity} = \left[\frac{34.3}{0.00256\ (0.5)}\right]^{1/2} = 164 \text{ mph}$$

$$\text{Corresponding side wall velocity} = \left[\frac{34.3}{0.00256\ (0.7)}\right]^{1/2} = 138 \text{ mph; governs}$$

SUMMARY OF SIDING ALLOWABLE LOADINGS

The following table summarizes the allowable loads for the sandwich wall siding.

Load Type	Allowable Pressure psf	Allowable Pressure psi	Allowable Velocity mph	Failure Mode*
Windward velocity pressure (positive)	47.0	0.33	151	S
Leeward velocity pressure (negative)	34.3	0.24	164	F
Sidewall velocity pressure (negative)	34.3	0.24	138	F
Bursting pressure (negative)	34.3	0.24	-	F

*S = Siding failure
F = Fastener failure (pullover)

APPENDIX D

References

D-1 Specifications for Aluminum Structures, Construction Manual Section 1; 3rd Edition

D-2 Aluminum Standards and Data; 1979

APPENDIX II

DESIGN REVIEW CALCULATIONS

Franklin Research Center
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Title: Concrete Wall in Screen House / Diesel Gen / Discharge Structure.

M_u = Capacity of 10" thick reinforced concrete wall in bending, calculated by Licensee, reported in the SAR

$M_u = 3.78$ k-ft   Cantilever wall section

$M_u = 0.8P \cdot \frac{H^2}{2}$ $H = 14$ ft

$\therefore P = \frac{3.78 \times 2 \times 1000}{0.8(14)^2} = 48.2143$ psf

a) Allowable tornado dynamic pressure = 48.2143 psf

$\Rightarrow$ corresponding wind speed = $\sqrt{\frac{48.2143}{0.00256}}$ = $\underline{137.24 \text{ mph}}$

b) Allowable differential pressure = 48.2143×0.8

= 38.57 psf (0.27 psi)

$\Rightarrow$ Corresponding wind speed = $\sqrt{\frac{38.57}{0.00571}}$ = $\underline{86.88 \text{ mph}}$

c) Allowable high wind dynamic pressure = $\frac{48.2143}{1.7}$

[Load factor = 1.7 for Concrete as per SRP 3.8.4, Sec. 3b]   = 28.3614 psf

$\Rightarrow$ corresponding wind speed = $\underline{103.94 \text{ mph}}$

(ANSI A58.1, Table 5,C)

Franklin Research Center
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Project	Page	By	Date	Ch'k'd	Date	Rev.	Date
02G - C5257 -01	II - 2	RA	AUG '82	GTB	9-82		

Title: ROOF DECKING IN SCREEN HOUSE/DIESEL GEN./DISCHARGE STR.

Deck failure pressure when 0.9 Fy allowable bending stress used = 50.1 psf

∴ Deck failure pressure when 0.6 Fy allowable bending stress used, will be = $\frac{50.1(0.6F_y)}{0.9F_y}$ = 33.4 psf

D.L. of roof = 9.0 psf.

a) Allowable differential pressure will be = 50.1 + 9.0
= 59.1 psf (0.41 psi)

⇒ corresponding wind velocity will be = $\sqrt{\frac{59.1}{0.00511}}$ = 107.54 mph

b) Allowable tornado dynamic pressure = $\frac{50.1+9.0}{0.7}$ = 84.43 psf
(Use 0.7 shape factor)

⇒ corresponding wind speed will be = $\sqrt{\frac{84.43}{0.00256}}$ = 181.6 mph

c) Allowable high wind dynamic pressure = $\frac{33.4+9}{0.7}$
= 60.57 psf

(ANSI, A58.1-1972, Table 5.C)
⇒ Corresponding wind speed will be = 156 mph

FORM CS-FRC-81

Franklin Research Center A Division of The Franklin Institute The Benjamin Franklin Parkway, Phila., Pa. 19103	Project 02G-C5257-01		Page II-3
	By RA Date AUG 82	Ch'k'd GTB Date 9-82	Rev. Date

Title REINFORCED CONCRETE STACK - REVIEW

Based on Table E-1* and Figure E-5*, the following value of section diameter/contour were found.

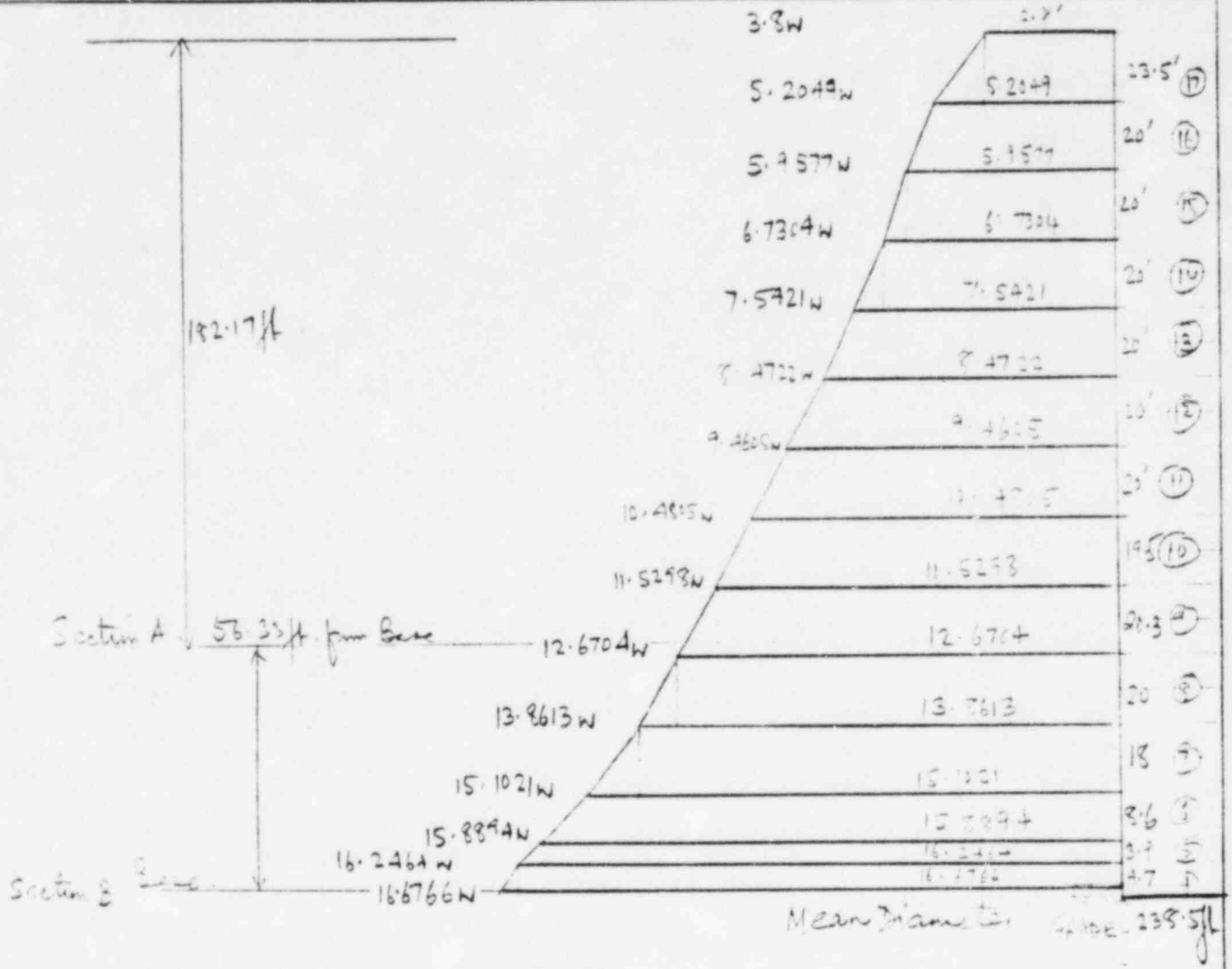
Section	Axial Area	Moment of Inertia	Do (ft.)	Di (ft.)	Mean Dia D (ft.)
4	47.1 ft²	EXTRAPOLATED			16.6766
5	61.8 ft	EXTRAPOLATED			16.2464
6	43.91	1320.0	16.763997	15.[illegible]	15.33[illegible]4
7	32.24	921.0	15.781654	14.42[illegible]	15.1[illegible]
8	27.85	670.3	14.500884	13.221797	13.8613
9	23.47	472.0	13.260004	12.08076	12.6704
10	19.91	331.6	12.079515	10.980186	11.5293
11	17.11	235.5	11.000164	9.9608674	10.[illegible]
12	14.85	166.6	9.9601528	8.9608614	9.4605
13	13.03	117.3	8.9617842	7.9826753	8.4722
14	11.36	81.1	8.0215133	7.0626252	7.5421
15	9.93	56.5	7.2000134	6.260[illegible]	6.7304
16	8.98	40.1	6.43744[illegible]3	5.4779[illegible]	5.9577
17	8.02	27.4	5.6953807	4.714[illegible]	5.2049

* From D'Appolonia Report, Reference # 26

FORM CS-FRC-81

Franklin Research Center A Division of The Franklin Institute The Benjamin Franklin Parkway, Phila., Pa. 19103	Project 02G-C5257-01		Page II-4
	By RA Date AUG 82	Ch'k'd STB Date 9-82	Rev. Date

Title REINFORCED CONCRETE STACK - REVIEW



$\therefore M_A = 110803.3\,w$ lbs-ft

If $w = 43$ lbs-ft

$\therefore M_A = 4764.5419$ k-ft

$\therefore M_B = 215576.28w$ lbs-ft

$\therefore M_B = 9269.78$ k-ft

FORM CS-FRC-81

Moment at Section A

$$M_A = \Big[3.8\,(23.5)(170.42) + 1.4049\left(\frac{23.5}{2}\right)(166.503333) +$$
$$5.2049\,(20)(148.67) + 0.7528\left(\frac{20}{2}\right)(145.33667) +$$
$$5.9577(20)(128.67) + 0.7727\left(\frac{20}{2}\right)(125.33667) +$$
$$6.7304\,(20)(108.67) + 0.8117\left(\frac{20}{2}\right)(105.33667) +$$
$$7.5421\,(20)(88.67) + 0.9301\left(\frac{20}{2}\right)(85.33667) +$$
$$8.4722\,(20)(68.67) + 0.9883\left(\frac{20}{2}\right)(65.33667) +$$
$$9.4605\,(20)(48.67) + 1.02\left(\frac{20}{2}\right)(45.33667) +$$
$$10.4805(19.5)(28.92) + 1.0493\left(\frac{19.5}{2}\right)(25.67) +$$
$$11.5298\,(19.17)(9.585) + 1.1406\left(\frac{19.17}{2}\right)(6.39)\Big]w$$

$$\therefore M_A = w\Big[15218.506 + 2748.5662 + 15476.25 + 1094.0945 +$$
$$15331.545 + 968.47645 + 14627.851 + 855.01775 +$$
$$13375.16 + 793.71637 + 11635.719 + 645.72231 +$$
$$9208.8507 + 462.43403 + 5910.3732 + 262.62143 +$$
$$2118.5368 + 69.85964\Big]$$

$$M_A = 110803.3\,w \quad \text{lbs-ft}$$

Moment at Base Section B.

$$
\begin{aligned}
M_B = W\Big[\, & 3.8(23.5)(226.75) + 1.4049\left(\tfrac{23.5}{2}\right)(222.83663) + \\
& 5.2049(20)(205.0) + 0.7528\left(\tfrac{20}{2}\right)(201.66667) + \\
& 5.9577(20)(185.0) + 0.7727\left(\tfrac{20}{2}\right)(181.66667) + \\
& 6.7304(20)(165.0) + 0.8117\left(\tfrac{20}{2}\right)(161.66667) + \\
& 7.5721(20)(145.0) + 0.9301\left(\tfrac{20}{2}\right)(141.6667) + \\
& 8.4722(20)(125.0) + 0.9383\left(\tfrac{20}{2}\right)(121.6667) + \\
& 9.4605(20)(105.0) + 1.02\left(\tfrac{20}{2}\right)(101.6667) + \\
& 10.4805(19.5)(85.25) + 1.0493\left(\tfrac{19.5}{2}\right)(82.0) + \\
& 11.5298(20.3)(65.35) + 1.1406\left(\tfrac{20.3}{2}\right)(61.9667) + \\
& 12.6704(20)(45.2) + 1.1909\left(\tfrac{20}{2}\right)(41.86667) + \\
& 13.8613(18)(26.2) + 1.2408\left(\tfrac{18}{2}\right)(23.2) + \\
& 15.1021(8.6)(12.9) + 0.7873\left(\tfrac{8.6}{2}\right)(14.333) + \\
& 15.8894(3.9)(6.65) + 0.357\left(\tfrac{3.9}{2}\right)(6.0) + \\
& 16.2464(4.7)\left(\tfrac{4.7}{2}\right) + 0.4302\left(\tfrac{4.7}{2}\right)\left(\tfrac{4.7}{3}\right)\Big]
\end{aligned}
$$

$$
\begin{aligned}
= \; & 20248.775 + 3678.4924 + 21340.09 + 1518.1467 + 22043.49 + 1403.7384 \\
& 22210.32 + 1312.2484 + 21872.09 + 1317.6416 + 21180.5 + 1202.4317 \\
& 19867.05 + 1037.0 + 17422.52 + 838.91535 + 15295.49 + 717.39366 \\
& 11454.042 + 498.59017 + 6536.9891 + 259.07904 + 1675.427 + 48.523922 \\
& 412.09159 + 4.1769 + 179.44149 + 1.583853\,] = 215576.28W \text{ lbs-ft.}
\end{aligned}
$$

From ACI 307-79, for sections with no openings

$$\frac{e}{r} = \frac{(1-\beta)(\alpha - \sin\alpha\cos\alpha) + np\pi}{2\left[(1-\beta)(\sin\alpha - \alpha\cos\alpha) - np\pi\cos\alpha\right]}$$

For section 56 ft 4 in above base

$\frac{e}{r} = 2.11$ and $\alpha = 55°$ $= 0.959931$ rad

$n = 8$

$\beta = 0$

$p = 0.0057$

$np = 0.0456$

$$\therefore \frac{e}{r} = \frac{(1-0.0057)(0.959931 - \sin 55°\cos 55°) + 8\pi \times 0.0057}{2\left[(1-0.0057)(\sin 55° - 0.959931\cos 55°) - 8(0.0057)\pi\cos 55°\right]}$$

$$= \frac{0.6305478}{0.3697176} = 1.7055 \neq 2.11$$

$\frac{e}{r} = 2.11$ given is not the same as $\frac{e}{r}$ from formula

The method of stack analysis is acceptable but not the numerical numbers. There is some numerical error.

6" thick Steel Cover

Approximate Density of steel = 490 lb/ft^3

∴ Dead wt of steel cover = 0.5×490
= 245 psf

If no fasteners are provided
∴ The allowable uplift pressure = 245 psf (1.7014 psi)

⟹ corresponding wind speed = $\sqrt{\dfrac{245}{0.00511}}$ = 218.96 mph

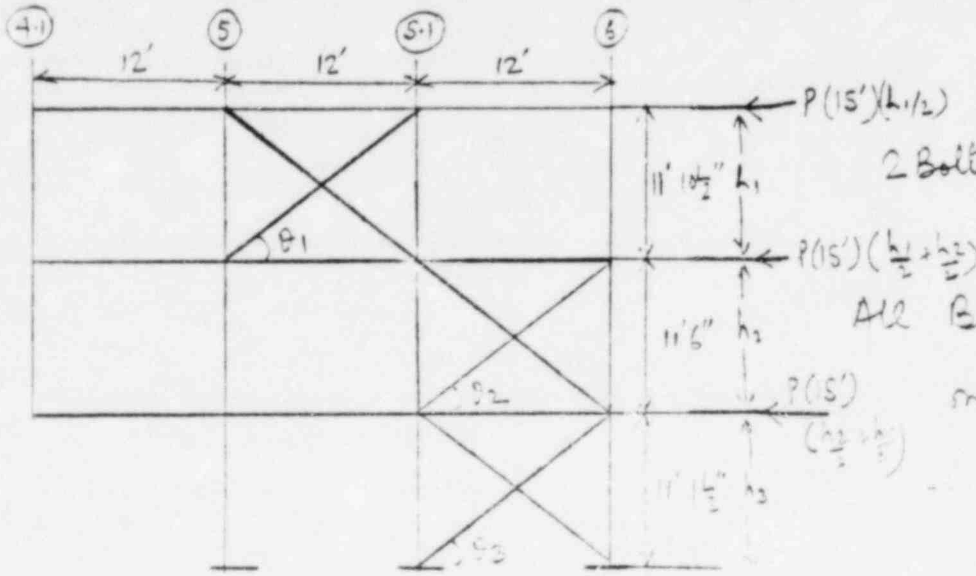
12" thick Concrete cover

Allowable uplift pressure = 1.04 psi

∴ the corresponding wind speed = $\sqrt{\dfrac{1.04 \times 144}{0.00511}}$

= 171.194 mph

Title: STEEL BRACINGS ALONG EAST ELEVATION — COLUMN LINE J



2 Bolts of 3/4" φ

All Bracings are one angle ∠ 3×2½×1/4

Gross area of angle = 1.31 in²

Bolts area $= \dfrac{\pi d^2}{4}$ $\qquad d = \frac{7}{8}$" holes

$= \dfrac{\pi}{4}\left(\dfrac{7}{8}\right)^2 = 0.6013204$

Assume the bolts are longitudinal, subtract one bolt area

$\therefore$ net area = 1.31 − 0.6013204

$\therefore A_n = 0.7086795 \text{ in}^2$

For Tornado loadings increase allowable stress by 1.6

$\therefore$ Allowable Tensile stress = $1.6 \times 0.6 F_y$

$F_y = 36$ ksi $\qquad = 34.56$ ksi

$\therefore$ Tensile capacity of each bracing $= 34.56 A_n$

$= 34.56 \times 0.7086795$

$\therefore T = 24.491964$ kips

Franklin Research Center A Division of The Franklin Institute The Benjamin Franklin Parkway, Phila., Pa. 19103	Project 02G - C5257 - 01		Page II - 10
	By RA Date AUG 82	Ch'k'd GTS Date 3/2/	Rev. Date

Title STEEL BRACING ALONG EAST ELEVATION — COLUMN LINE J.

For Compressive strength of each bracing

Assuming connection at middle X also

$$\text{Unbraced Length} = \frac{1}{2}\left(\sqrt{(12)^2 + (11.875)^2}\right) = 8.4412029'$$

$$\therefore Kl/r_{min} = \frac{8.4412029 \times 12}{0.753} = 134.52116 > C_c = 126.1$$

$K = 1.0$ for $F_y = 36$ ksi

$$F_a = \frac{12\pi^2 E}{23(Kl/r)^2} = \frac{3434622.3}{23(134.52116)^2}$$

$\therefore F_a = 8.2522036$ ksi

For Tornado increase allowable stress by 1.6

$\therefore$ allowable Compressive stress $= 8.2522036 \times 1.6 = 13.203526$ ksi

$\therefore$ Compressive capacity of each bracing $= 13.203526 \times 1.31$

$C = 17.296619$ kips

Total resistance to wind force will be

$(T+C)\cos\theta_1 = (24.491964 + 17.296619)\,0.5052354 = 21.113074$

$(T+C)\cos\theta_2 = (24.491964 + 17.296619)\,0.5212669 = 21.783006$

$(T+C)\cos\theta_3 = (24.491964 + 17.296619)\,0.5377837 = 22.473219$

All load goes into the bottom most bracing section

Total capacity for lateral wind = 22.473219 kips

lateral supported width = 15 ft.

windward side load = 0.8P, leeward side load = 0.5P

Franklin Research Center A Division of The Franklin Institute The Benjamin Franklin Parkway, Phila., Pa. 19103	Project 02G – C5257 – 01		Page II – 11
	By RA	Date AUG '82	Ch'k'd Date	Rev. Date

Title: STEEL BRACING ALONG EAST ELEV. — COLUMN LINE J.

Allowable pressure bracing can resist $= \dfrac{22.473219 \times 1000}{15 \times 1.3(h_1 + h_2 + \frac{h_3}{2})}$

(With allowable increase of 1.6) $\therefore p = \dfrac{22473.219}{15 \times 1.3(28.9375)} = 39.826273$ psf

If no increase of 1.6 allowed

then pressure allowable $= \dfrac{39.83}{1.6} = 24.891421$ psf

a) Allowable tornado dynamic pressure $= 39.826273$ psf

$\Rightarrow$ corresponding wind speed $= \sqrt{\dfrac{39.826273}{0.00256}}$

$= \underline{124.73 \text{ mph}}$

b) Allowable high wind dynamic pressure 24.8914 psf

$\Rightarrow$ corresponding wind speed : $\underline{86.334 \text{ mph}}$

(ANSI A58.1-1972 Table 5C)

FORM CS-FRC-81

Title EXTERIOR METAL SIDING — CAPACITY AND WIND SPEEDS.

The Licensee states in the SAR that:

a) Windward face siding can withstand tornado dynamic pressure of 47psf (0.33psi) which corresponds to 151mph wind speeds when 0.8 shape factor is used ∴ Allowable tornado dynamic pressure $= \frac{47}{0.8} = 58.75$ psf

b) But for differential pressure case, fasteners were to be limiting, with allowable pressure of 34.3psf (0.24psi) which corresponds to 138mph wind speeds for side walls with 0.7 shape factor and 164mph for leeward walls with 0.5 shape factor

For Limiting case

∴ Allowable differential pressure = 34.3 = 0.24 psi

$\Rightarrow$ corresponding wind velocity $= \sqrt{\frac{34.3}{0.00511}} = \underline{81.93 \text{ mph}}$

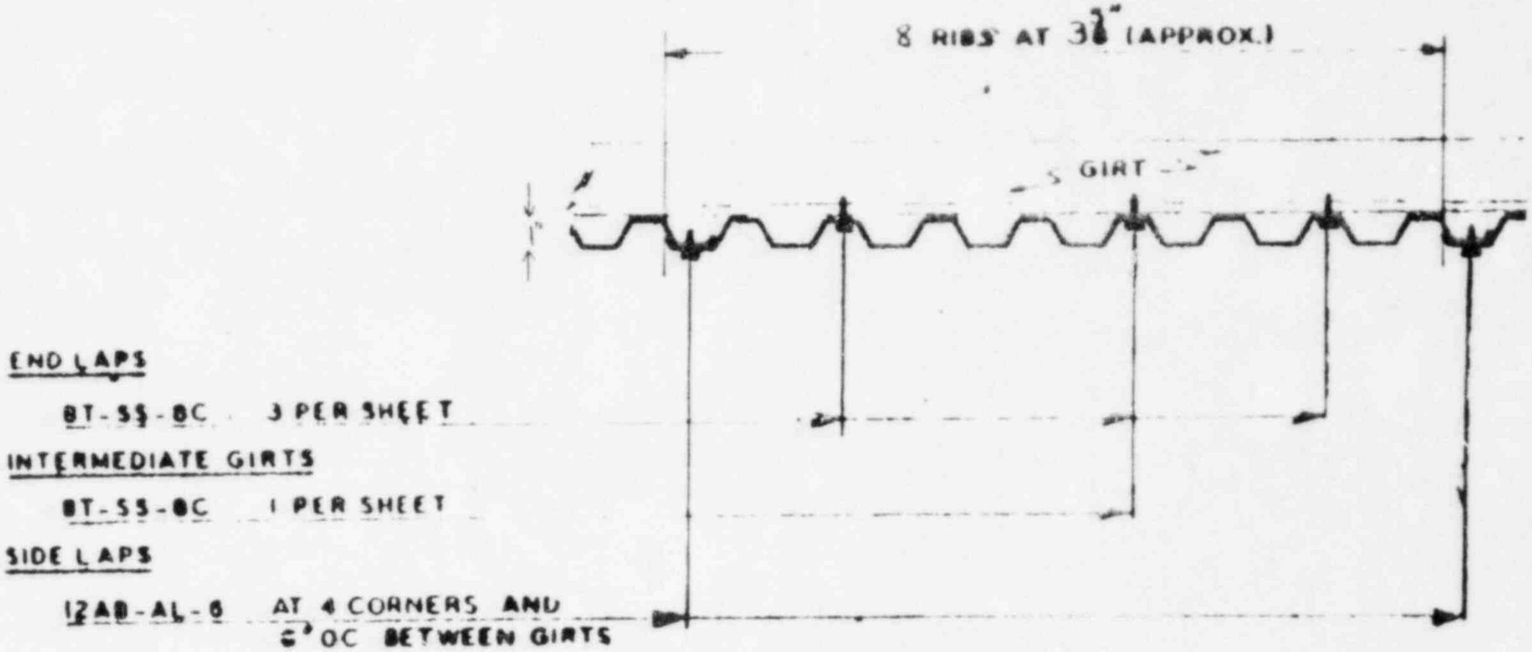
From allowable tornado dynamic pressure, we find the allowable high wind dynamic pressure $= \frac{58.75}{1.6} = 36.72$ psf

(Divide by 1.6, as the Licensee has used 0.9Fy as allowable stress)

(Assume height above grade 30ft)

$\Rightarrow$ corresponding wind speed = $\underline{105.32 \text{ mph}}$

(ANSI A58.1-1972, Table 5c)



SQUARE RIB SIDING – FASTENING SCHEDULE

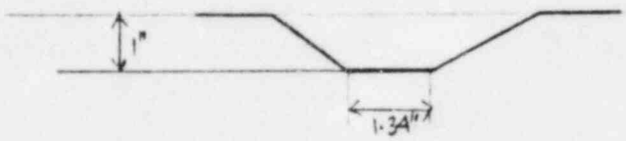
[FROM DWG No. 1-08
Contract No. 5312 T-60-GO

ELWIN G. SMITH & COMPANY, INC.
PITTSBURGH, PA.]

From the ...
SAR Appendix ...
Siding panel is
0.032-inch thick
ALCOA 4-inch Ribbed
Aluminum

Title EXTERIOR SIDING — ALLOWABLE STRESS CHECK BUCKLING

These dimensions shown below are approximate from the drawing



From Licencee SAR the allowable stress is 25.2 ksi, but buckling criteria was not checked.
thickness = 0.032" and $S = 0.160\ in^3$
formula used to calculate pressure

$$w = \frac{F_b(8)S}{12L^2}$$

According to 2 of the book 'Formulas for Stress and strain' by Roark & Young (McGraw Hill) page 426

Allowable $\sigma = 15{,}000 - 123K\frac{b}{t}$ $\qquad K\frac{b}{t} < 81$

$\sigma = \frac{33\times10^6}{(K\frac{b}{t})^2}$ $\qquad K\frac{b}{t} > 81$

Take $K = 3$

$b = \frac{1.34}{2} = 0.67$ $\qquad K\frac{b}{t} = \frac{3\times0.67}{0.032} = 62.8125$

$t = 0.032"$

$\therefore \sigma = 15{,}000 - 123\times62.8125$

$= 7274.0625\ psi$

Allowable stress $\therefore \sigma = 7.2740625\ ksi$

For tornado increase allowable by 1.6 then $\sigma = 7.2741\times1.6 = 11.6385\ ksi$

∴ Allowable wind pressure

$$w = \frac{11.6385 \times 8\ (0.160)}{12 \times (7.5833)^2}$$

$$= 0.0215876\ \text{Ksf}$$

$$\therefore w = 21.587654\ \text{psf}$$

a) Allowable differential pressure = 0.15 psi

$\Rightarrow$ corresponding wind velocity $= \sqrt{\frac{21.5876}{0.00511}} = 65$ mph

b) Allowable tornado dynamic pressure $= \frac{21.587654}{0.8} = 26.985$ psf
(Use 0.8 shape factor)

$\Rightarrow$ corresponding wind velocity $= \sqrt{\frac{26.985}{0.00256}} = 102.67$ mph

c) If the allowable high wind dynamic pressure = 16.875 psf

Wind speed from Table 6C of ANSI A58.1-1972
will be dependent on elevation above grade

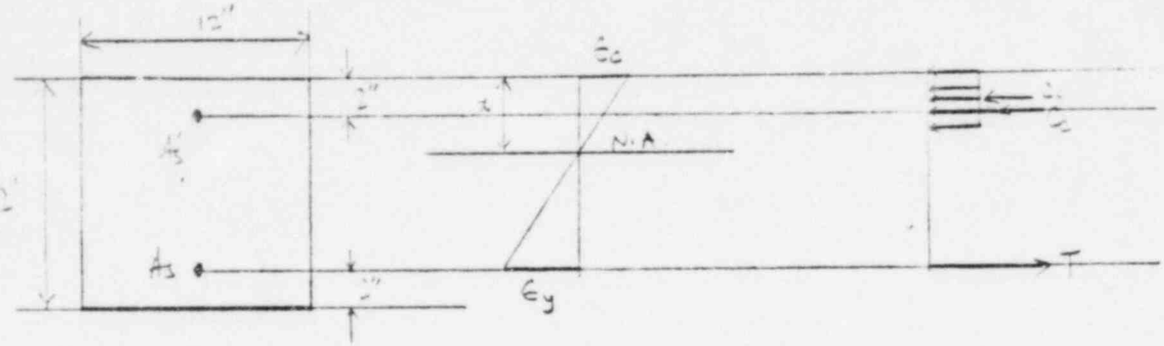
For Penetration rooms height above grade is 34 ft 6 inch
Corresponding wind speed = 65.1 mph

Franklin Research Center A Division of The Franklin Institute The Benjamin Franklin Parkway, Phila., Pa. 19103	Project 02G-C5257-01		Page II-16
	By RA, Date SEP'82	Ch'k'd GTB, Date 9/82	Rev. Date

Title: EAST SIDE REINFORCED CONCRETE WALL — CONTROL ROOM

This wall is 12" thick #4 bars at 12" Each face each way. Roof load transferred through steel frame. Design this as a beam section.

Consider 1ft wide section for analysis



Area of steel $A_s = A_s'$ = Area of #4 bar = 0.20 in²

Steel $f_y = 40$ ksi

Concrete $f_c' = 2.5$ ksi

Compression $C_c = 0.85 f_c' a b$

$a = 0.85x$

$\therefore C_c = 0.85(2.5)(0.85x)12 = 21.675x$ kips

$C_s = A_s'(f_y - 0.85 f_c')$

$= 0.2(40 - 0.85 \times 2.5) = 7.575$ kips

Tension $T = A_s f_y$

$= 0.2 \times 40 = 8$ kips

FORM CS-FRC-81

Franklin Research Center
A Division of The Franklin Institute
The Benjamin Franklin Parkway, Phila., Pa. 19103

Project 02G – C5257 – 01		Page II-17
By RA Date SEP'82	Ch'k'd GTS Date 9/82	Rev. Date

Title: EAST SIDE REINFORCED CONCRETE WALL — CONTROL ROOM

For equilibrium

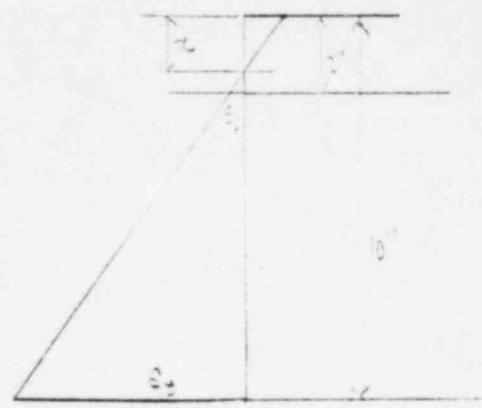
$$C_c + C_s = T$$

$$\therefore 21.675x + 7.575 = 8$$

$$\Rightarrow x = 0.0196 \text{ inch}$$

This implies upper steel also in tension

From Strain Geometry



$$\frac{\epsilon_s}{(2-x)} = \frac{\epsilon_y}{(10-x)}$$

$$\epsilon_s = \epsilon_y \frac{(2-x)}{(10-x)}$$

$$f_s = f_y \frac{(2-x)}{(10-x)}$$

$$\therefore T' = A_s' f_s$$

$$= 0.2 f_y \frac{(2-x)}{(10-x)} = 0.2 \times 40 \frac{(2-x)}{(10-x)}$$

$$\therefore T' = 8 \frac{(2-x)}{(10-x)}$$

For equilibrium

$$T + T' = C_s$$

$$8 + 8\frac{(2-x)}{(10-x)} = 21.675x$$

$$\Rightarrow 21.675x^2 - 216.75x + 96 - 16x = 0$$

FORM CS-FRC-81

Franklin Research Center A Division of The Franklin Institute The Benjamin Franklin Parkway, Phila., Pa. 19103	Project 02G – C5257 – 01		Page II-18	
	By RA	Date SEP'82	Ch'k'd [illegible] Date 9/82	Rev. Date

Title: EAST SIDE REINFORCED CONCRETE WALL — CONTROL ROOM

$$\Rightarrow \quad 21.675x^2 - 232.75x + 96 = 0$$

$$\therefore \quad x = \frac{+232.75 \pm \sqrt{(232.75)^2 - 4\times 96\times 21.675}}{2\times 21.675}$$

$$x = 0.4296506 \text{ inch}$$

$$\therefore \quad C = 21.675x = 21.675\times 0.4296506$$

$$= 9.3126788 \text{ kips}$$

$$\therefore \quad T' = \frac{8(2-x)}{(10-x)} = 1.3126788 \text{ kips}$$

Now the moment capacity of the wall

$$M = T\left(d - \frac{a}{2}\right) + T'\left(2 - \frac{a}{2}\right)$$

$a = 0.85x$
$= 0.365203$

$$= 8(10 - 0.1826015) + 1.3126788\,(2 - 0.1826015)$$

$$M = 80.924545 \text{ k-inch}$$

$$\therefore M = 6.7437374 \text{ k-ft}$$

The vertical span of wall = 12 ft. For one foot section allowable pressure is p. Then $M = \frac{pl^2}{8}$

$$\therefore \quad p = \frac{8\times 6.7437374\times 1000}{12\times 12} = 374.65208 \text{ psf}$$

a) Allowable differential pressure = 2.602 psi

$\Rightarrow$ corresponding wind speed $= \sqrt{\frac{374.65208}{0.00256}} = \underline{\underline{370.77 \text{ mph}}}$

b) Allowable tornado dynamic pressure $= \frac{374.65208}{0.8} = 468.32$ psf
(Use 0.8 shapefactor)

$\Rightarrow$ corresponding wind speed $= \sqrt{\frac{468.32}{0.00256}} = \underline{\underline{427.71 \text{ mph}}}$

FORM CS-FRC-81

Franklin Research Center
A Division of The Franklin Institute
The Benjamin Franklin Parkway, Phila., Pa. 19103

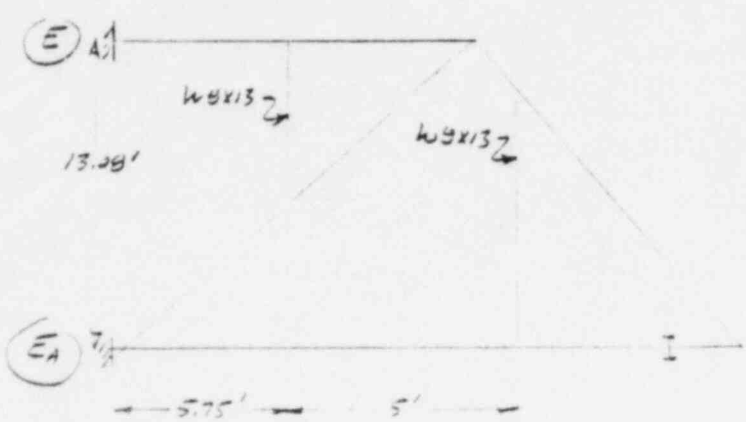
Project	Page
026 – C5257-01	II – 17

By	Date	Ch'k'd	Date	Rev.	Date
DJB	9/17/82	RA	9/82		

Title: PASSAGEWAY, ROOF STEEL, BIG ROCK POINT

ROOF STEEL

W8x13
$A = 3.83\ IN^2$
$S_{xx} = 9.90\ IN^3$
$R_T = 1.02\ ^{IN}$
$d/A_F = 7.87$



FOR $l = 13.08'$

$l/R_T = 13.08 \times 12'' / 1.02^{IN} = 153.9 > 119$

$F_B = 170 \times 10^3 / (153.9)^2 = 7.18^{KSI}$

$F_B = 12 \times 10^3 / (13.08 \times 12^{IN})(7.87) = 9.71^{KSI}$

$\Rightarrow M_{ALLOW} = (9.90\ IN^3)(9.71\ KSI)$
$= 96.1\ ^{K-IN}$
$= 8.01\ ^{K-FT}$

SUPPORT AREA $= \frac{1}{2}(5.75 + 5')(11') = 5.38\ ^{\square'}/_{FT}$

FOR UPLIFT PRESSURE $(M = \frac{1}{8} w l^2)$

$8.01^{K \cdot FT} = \frac{1}{8}(5.38\,P)(13.08)^2$

$P = 69.62\,psf$

FORM CS-FRC-81

Franklin Research Center A Division of The Franklin Institute The Benjamin Franklin Parkway, Phila., Pa. 19103	Project 026-C5357-01				Page II-20	
	By DJB	Date 9/17/82	Ch'k'd RA	Date 9/82	Rev.	Date

Title: PASSAGEWAY, ROOF STEEL, BIG ROCK POINT

DEAD WEIGHT OF BEAM: 13#/' ⟹ 13/5.33 = 2.41 PSF

DEAD WEIGHT OF ROOF DECK: ASSUME 7 PSF

∴ TORNADO UPLIFT CAPACITY = (1.6)(69.62) + 2.41 + 7 = 120.7996 psf

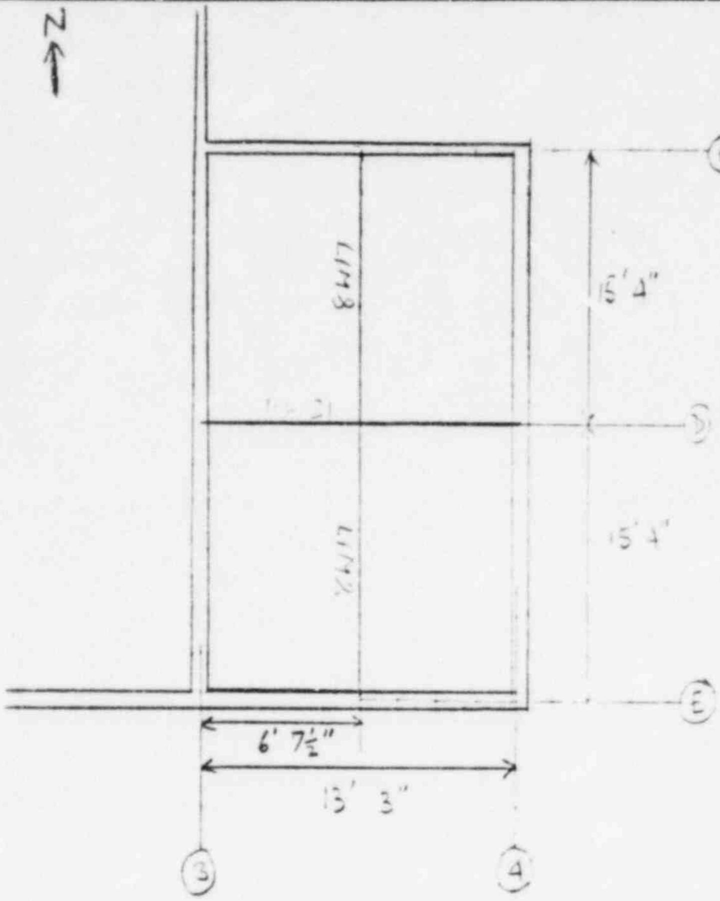
DIFFERENTIAL PRESSURE $\frac{120.8}{144} = .8389$ psi $V = \sqrt{\frac{120.7996}{.00511}} = 153.75$ mph

TORNADO DYNAMIC PRESSURE $\frac{120.7996}{0.7} = 172.571$ $V = \sqrt{\frac{172.571}{.00256}} = 259$ mph

Franklin Research Center
A Division of The Franklin Institute
The Benjamin Franklin Parkway, Phila., Pa. 19103

Project 02G – C5257 – 01			Page II-21
By RA	Date SEP '82	Ch'k'd GTB Date 9/82	Rev. Date

Title: ROOF STEEL – EMERGENCY DIESEL GENERATOR ROOM



Roof Beam

8W17

Unbraced Length

$l = 15'4"$

From AISC handbook

For $F_y = 36$ ksi

Allowable Moment

$M = 15$ k-ft

Area of Roof deck supported by 8W17 = 6'7½" x 15'4"

For 8W17 $I_{xx} = 56.6 \text{ in}^4 \quad d = 8 \text{ in}$

$\therefore$ allowable stress $f_b = \frac{M}{I} y$

$= \frac{15 \times 12}{56.6} \times \frac{8}{2}$

$\therefore f_b = 12.721$ ksi

For tornado f_b can be increased by 1.6 times

FORM CS-FRC-81

(8W17 Cont'd)

Convert allowable moment to allowable pressure

$$M = \frac{w l^2}{8}$$

$$\therefore \text{load } w = \frac{8 \times 1000 \times 15}{(15.333)^2} = 510.397 \text{ lbs/ft}$$

$$\therefore \text{allowable pressure} = \frac{510.397}{6.625}$$

$$p = 77.0411 \text{ psf}$$

For tornado increase allowable by 1.6

$$\therefore \text{Allowable tornado pressure} = 1.6 \times 77.0411$$

$$= 123.2657 \text{ psf}$$

For uplift add wt. of roof deck

From the SAR wt. of decking = 9 psf

$$\text{Self wt of 8W17} = \frac{17}{6.625} = 2.566 \text{ psf}$$

$$\therefore \text{Allowable uplift pressure} = 123.2657 + 9 + 2.566$$

$$= 134.8317 \text{ psf}$$

a) Allowable uplift pressure 8W17 can withstand = 134.8317 psf which is equivalent to differential pressure of 0.9363 psi corresponding to a wind speed of

$$= \sqrt{\frac{134.8317}{0.00511}} = \underline{162.44 \text{ mph}}$$

b) Allowable tornado dynamic pressure (0.7 shapefactor) $= \frac{134.8317}{0.7}$

$$= 192.6167 \text{ psf}$$

Franklin Research Center A Division of The Franklin Institute The Benjamin Franklin Parkway, Phila., Pa. 19103	Project 02G - C5257 - 01			Page II-23
	By RA	Date SEP '82	Ch'k'd GTB Date 9/82	Rev. Date

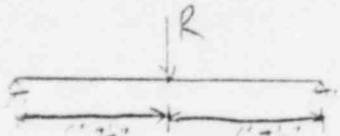
Title: ROOF STEEL — EMERGENCY DIESEL GENERATOR ROOM

For allowable tornado dynamic pressure of [illegible] psf

corresponding wind speed $= \sqrt{\dfrac{192.6167}{0.00256}} = \underline{274.301 \text{ mph}}$

10W21

The 10W21 supports reaction from 8W17



R is the reaction from 8W17

Unbraced length for uplift load = 6'7½"

Using the AISC Manual beam diagrams

allowable moment in beam $M = 39.4$ k-ft

Max. moment due to R at center $M = \dfrac{Rl}{4}$

$l = 13'3"$ $\qquad M = 39.4$ k-ft

$\therefore R = \dfrac{4 \times 39.4 \times 1000}{13.25}$

$\therefore R = 11894.34$ lbs.

Convert this to pressure $p = \dfrac{11896.34}{(15.2[illegible]) \, [illegible]}$

$\therefore p = 117.08943$ psf

FORM CS-FRC-81

Franklin Research Center A Division of The Franklin Institute The Benjamin Franklin Parkway, Phila., Pa. 19103	Project 02G-C5257-01		Page II-24
	By RA Date SEP '82	Ch'k'd GTB Date 9/82	Rev. Date

Title: ROOF STEEL - EMERGENCY DIESEL GENERATOR ROOM

For tornado loading increase allowables by 1.6

$\therefore 1.6p = 187.34317$ psf

$\therefore$ Allowable uplift pressure = 187.34317 + 9 ([illegible])

= 196.34317 psf

a) Allowable differential pressure = 196.34317 psf = 1.3635 psi

$\Rightarrow$ corresponding wind speed = $\sqrt{\frac{196.34317}{0.00511}}$ = 196.02 mph

b) Allowable tornado dynamic pressure = $\frac{196.34317}{0.7}$

(Use 0.7 shape factor)

= 280.49024 psf

$\Rightarrow$ corresponding wind speed = $\sqrt{\frac{280.49024}{0.00256}}$

= 331.008 mph

FORM CS-FRC-81

APPENDIX III

SPHERICAL CONTAINMENT SHELL DESIGN REVIEW CALCULATIONS

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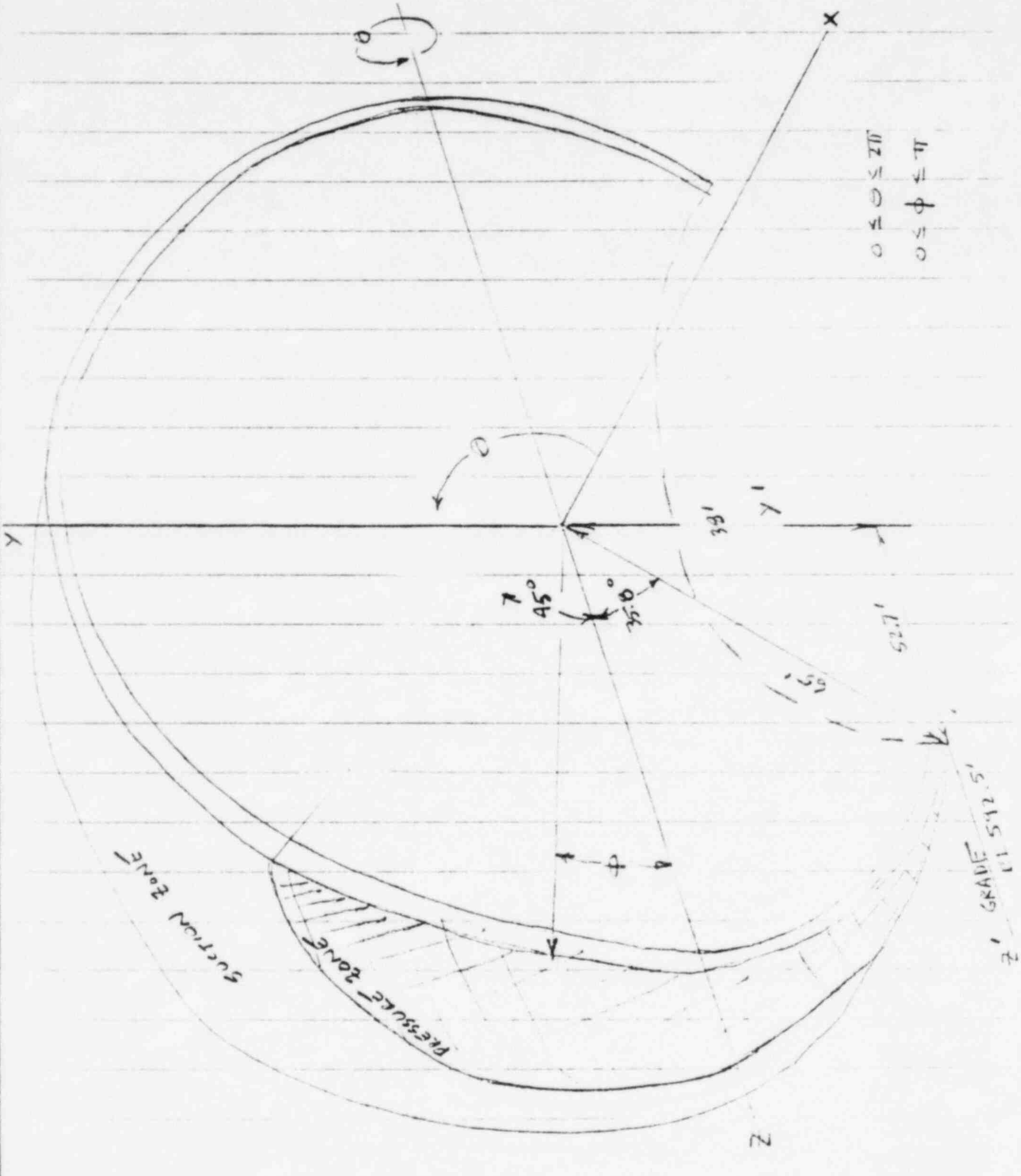
Franklin Research Center A Division of The Franklin Institute The Benjamin Franklin Parkway, Phila., Pa. 19103	Project 026-C5257-01				Page III-1	
	By DJB	Date 3/10/82	Ch'k'd RA	Date 8/82	Rev.	Date

Title: CONTAINMENT VESSEL, BIG ROCK POINT



FORM 207-5M-4-80-CP

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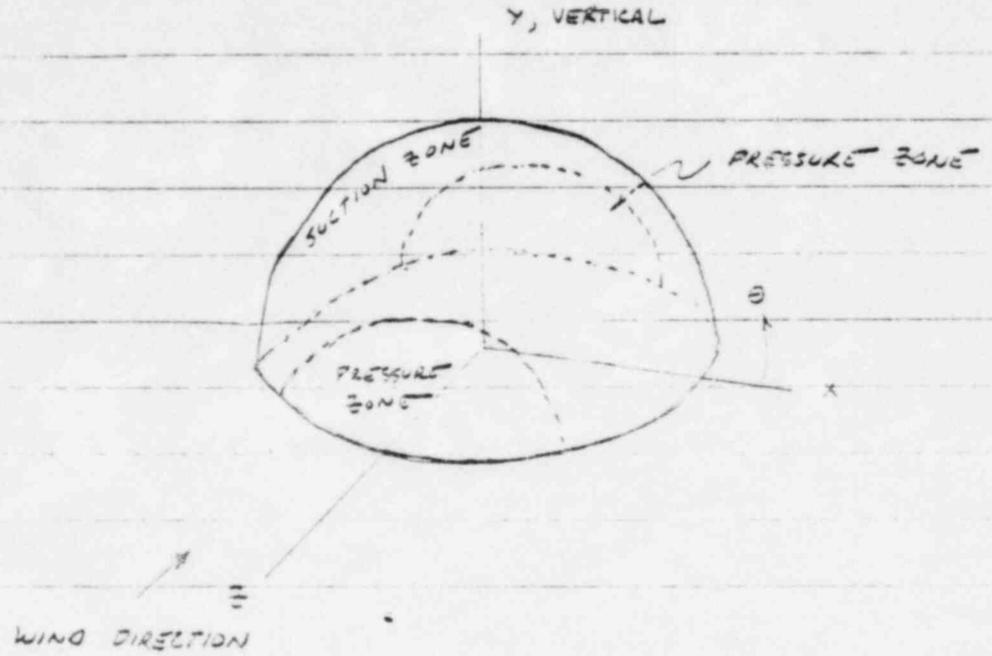
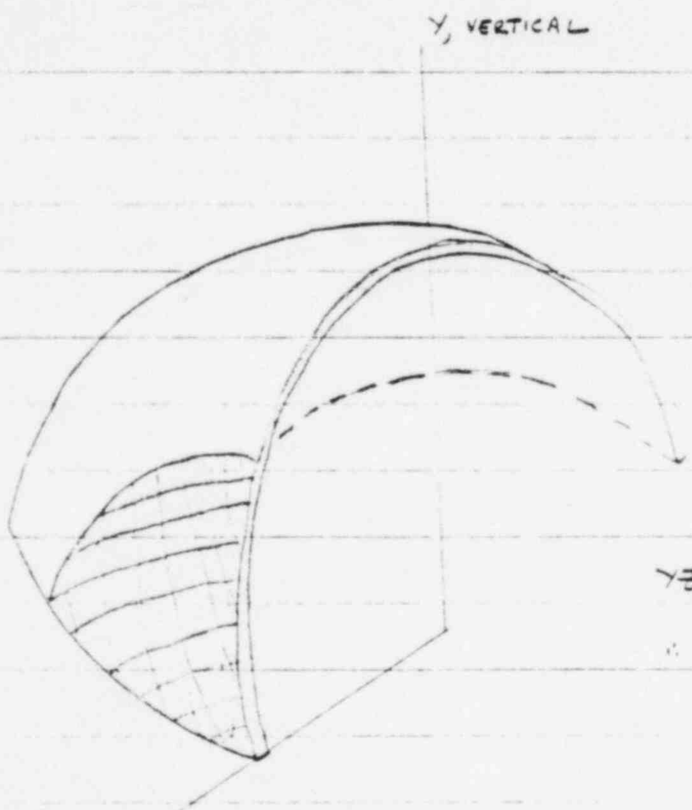
Project				Page
02B-C5257-01				III-2
By	Date	Ch'k'd	Date	Rev. / Date
DJS	8/16/82	RA	8/82	

Title
CONTAINMENT VESSEL, B's ROCK POINT





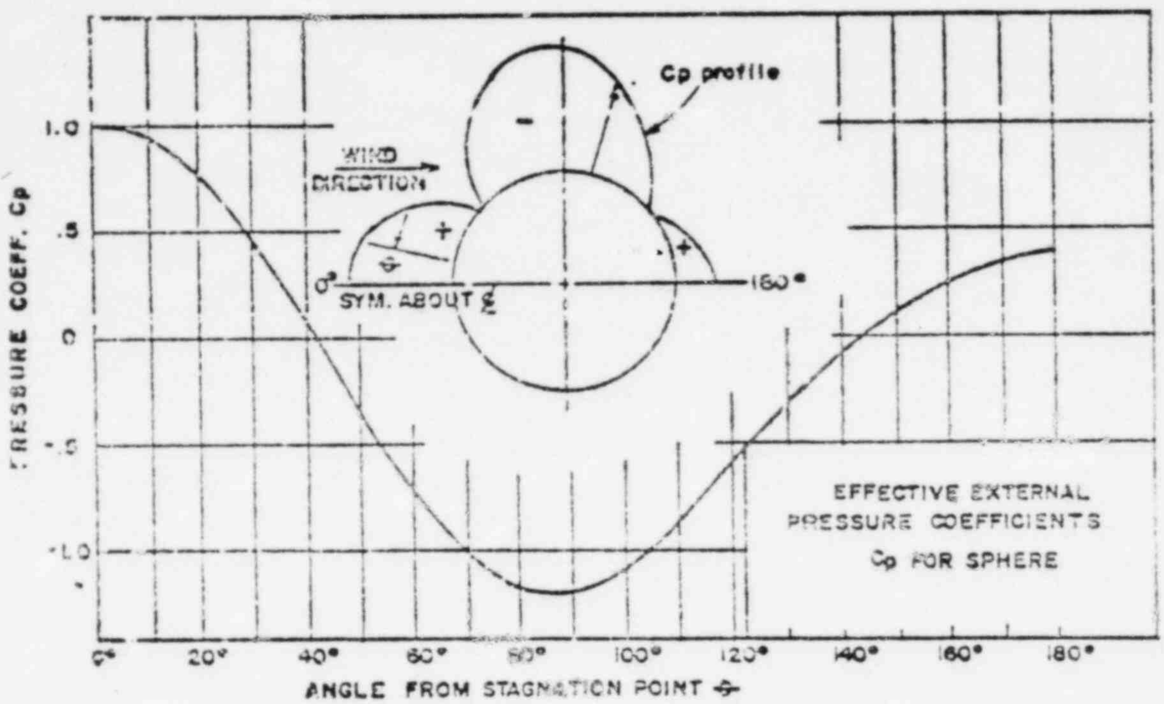
YZ PLANE OF SYMMETRY

∴ SHEAR STRESSES ON CUT AND

NORMAL DISPLACEMENTS ARE ZERO

Franklin Research Center A Division of The Franklin Institute The Benjamin Franklin Parkway, Phila., Pa. 19103	Project 026-C5357-01				Page III-3	
	By DJB	Date 8/16/82	Ch'k'd RA	Date 8/82	Rev.	Date

Title: CONTAINMENT VESSEL, PRESSURE DISTRIBUTION ON SPHERES, BIG ROCK POINT



Velocity Pressure Distribution

WIND FORCES ON STRUCTURES

ASCE PAPER # 3269

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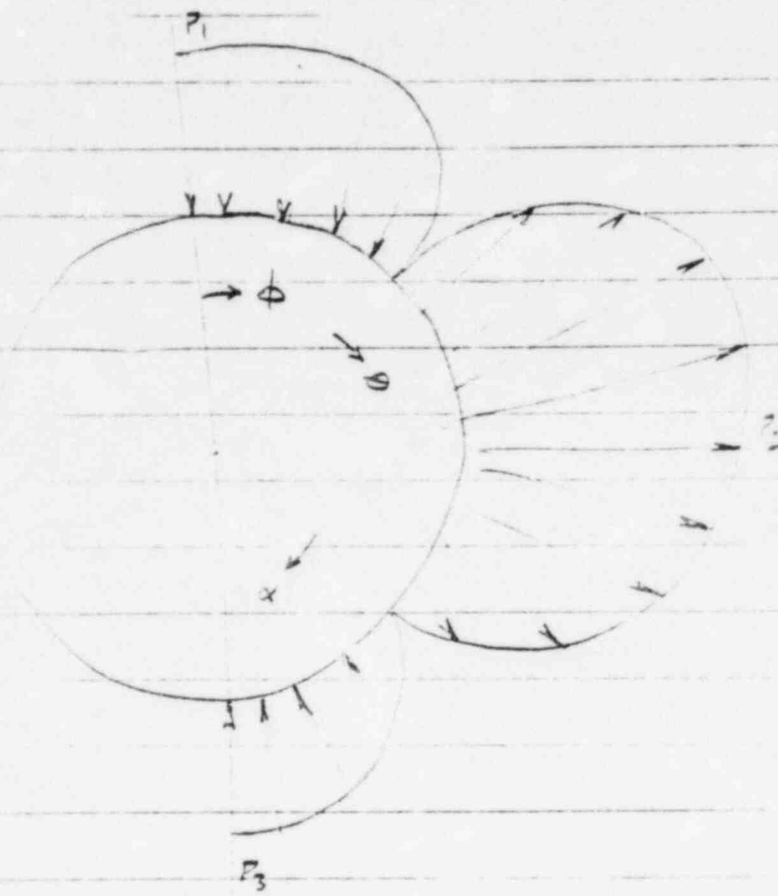
Project		Page	
026-C5257-01		III - 4	
By: DJB	Date: 8/16/82	Ch'k'd: RA	Date: 8/82

Rev. Date

Title: CONTAINMENT VESSEL, APPROXIMATE PRESSURE DISTRIBUTION, BIG ROCK POINT



$P_1 = -1.0\ P$

$P_2 = 1.2\ P$

$P_3 = -.40\ P$

$P(\phi) = P_1 \cos 2\phi \qquad 0 \le \phi \le \pi/4$

$P(\theta) = P_2 \sin 2\theta \qquad \pi/4 \le \phi \le 3\pi/4 \qquad \theta = \phi - \pi/4$

$P(\alpha) = P_3 \sin 2\alpha \qquad 3\pi/4 \le \phi \le \pi \qquad \alpha = \phi - 3\pi/4$

$\therefore \quad P(\phi) = P_1 \cos 2\phi \qquad 0 \le \phi \le \pi/4$

$= P_2 \sin(2\phi - \pi/2) \qquad \pi/4 \le \phi \le 3\pi/4$

$= P_3 \sin(2\phi - 3\pi/2) \qquad 3\pi/4 \le \phi \le \pi$

FORM 207-5M-4-80-CP

$$\sin(2\phi - \pi/2) = \sin 2\phi \cos(\pi/2) - \cos 2\phi \sin(\pi/2)$$

$$= -\cos 2\phi$$

$$\sin(2\phi - 3\pi/2) = \sin 2\phi \cos(3\pi/2) - \cos 2\phi \sin(3\pi/2)$$

$$= \cos 2\phi$$

$$P(\phi) = P_1 \cos 2\phi \qquad 0 \le \phi \le \pi/4$$

$$= -P_2 \cos 2\phi \qquad \pi/4 \le \phi \le 3\pi/4$$

$$= P_3 \cos 2\phi \qquad 3\pi/4 \le \phi \le \pi$$

$P_1 = -1.0\,P$

$P_2 = 1.2\,P$

$P_3 = -.40\,P$

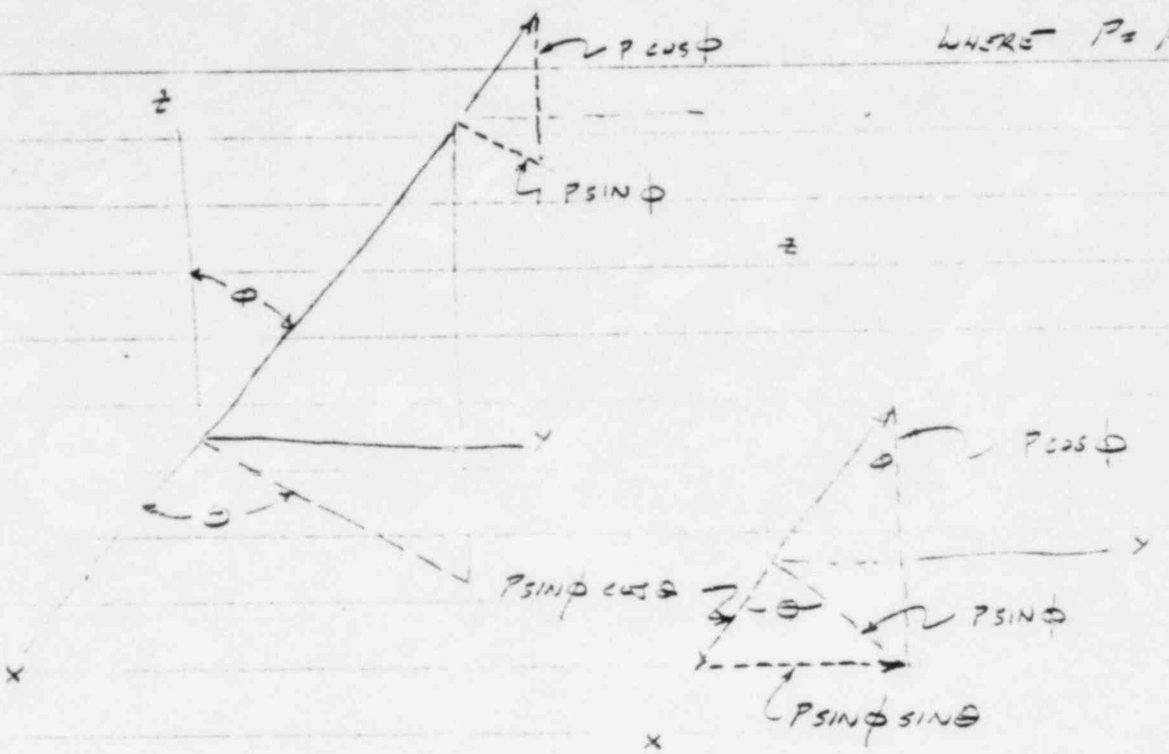
WHERE $P = .00256\,V^2$

V : WINDSPEED (mph)

P : DYNAMIC PRESSURE (psf)

Title: CONTAINMENT VESSEL, LATERAL AND UPLIFT FORCE INTEGRALS. BIG ROCK POINT

WHERE $P = P(\phi)$



DIFFERENTIAL AREA = $dS = a^2 \sin\phi \, d\phi \, d\theta$

$$F_x = \int_{AREA} P(\phi) \sin^2\phi \cos\theta \, a^2 \, d\phi \, d\theta$$

$$F_y = \int_{AREA} P(\phi) \sin^2\phi \sin\theta \, a^2 \, d\phi \, d\theta$$

$$F_z = \int_{AREA} P(\phi) \cos\phi \sin\phi \, a^2 \, d\phi \, d\theta$$

Franklin Research Center A Division of The Franklin Institute The Benjamin Franklin Parkway, Phila., Pa. 19103	Project 026-C5257-01	Page III-7
	By JJB Date 3/13/82 Ch'k'd RA Date 3/82	Rev. Date

Title: Containment Vessel, Force F_3, Big Rock Point

Total Force in Wind Direction

$$F_3 = \int_{AREA} P(\phi)\cos\phi\sin\phi\, a^2\, d\phi\, d\theta$$

$$\approx \int_0^{2\pi}\int_0^{\pi} P(\phi)\cos\phi\sin\phi\, a^2\, d\phi\, d\theta - \int_{1.2\pi}^{1.8\pi}\int_{.2\pi}^{.8\pi} P(\phi)\cos\phi\sin\phi\, a^2\, d\phi\, d\theta$$

$$H_1(\phi) = \cos 2\phi \cos\phi \sin\phi$$

$$H_2(\phi) = -\cos 2\phi \cos\phi \sin\phi$$

$$H_3(\phi) = \cos 2\phi \cos\phi \sin\phi$$

$$= 2\pi a^2 \left[\int_0^{.25\pi} P_1 H_1\, d\phi + \int_{.25\pi}^{.75\pi} P_2 H_2\, d\phi + \int_{.75\pi}^{\pi} P_3 H_3\, d\phi\right]$$

$$- (1.8\pi - 1.2\pi)\, a^2 \left[\int_{.20\pi}^{.25\pi} P_1 H_1\, d\phi + \int_{.25\pi}^{.75\pi} P_2 H_2\, d\phi + \int_{.75\pi}^{.80\pi} P_3 H_3\, d\phi\right]$$

$$= 2\pi a^2 (.125 P_1 + 0.0 P_2 - .125 P_3) - .6\pi a^2 (.0119 P_1 + 0.0 P_2 - .0119 P_3)$$

$$= 2\pi a^2 ((.125)(-1.0) - (.125)(-.40)) - .6\pi a^2 ((.0119)(-1.0) - (.0119)(-.40))\, P$$

$$= \pi a^2 P\, ((2)(-.075) - (.6)(-.00714))$$

$$= -.146\, \pi a^2 P$$

$$= -.000373\, \pi a^2 V^2$$

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TOTAL UPLIFT FORCE

$$F_y = \int_{AREA} P(\phi) \sin^2\phi \sin\theta \, a^2 \, d\phi \, d\theta$$

$$= \int_0^{2\pi}\int_0^{\pi} P(\phi)\sin^2\phi\sin\theta\, a^2 d\phi\, d\theta - \int_{1.2\pi}^{1.8\pi}\int_{.2\pi}^{.8\pi} P(\phi)\sin^2\phi\sin\theta\, a^2 d\phi\, d\theta$$

$$F_1(\phi) = \cos 2\phi \sin^2\phi$$

$$F_2(\phi) = -\cos 2\phi \sin^2\phi$$

$$F_3(\phi) = \cos 2\phi \sin^2\phi$$

$$= a^2\left(-\cos\theta\Big|_0^{2\pi}\right)\left[\int_0^{.25\pi} P_1F_1\,d\phi + \int_{.25\pi}^{.75\pi} P_2F_2\,d\phi + \int_{.75\pi}^{\pi} P_3F_3\,d\phi\right]$$

$$- a^2\left(-\cos\theta\Big|_{1.2\pi}^{1.8\pi}\right)\left[\int_{.2\pi}^{.25\pi} P_1F_1\,d\phi + \int_{.25\pi}^{.75\pi} P_2F_2\,d\phi + \int_{.75\pi}^{.80\pi} P_3F_3\,d\phi\right]$$

$$= 0 - a^2(-.8090 - .8090)(.0097 P_1 + .393 P_2 + .0097 P_3)$$

$$= 1.62a^2\left((.0097)(4.0) + (.393)(1.2) + (.0097)(-.40)\right)P$$

$$= 1.71\,a^2 P$$

$$= .00439\,a^2 V^2$$

Franklin Research Center A Division of The Franklin Institute The Benjamin Franklin Parkway, Phila., Pa. 19103	Project 026-C5357-01				Page III-9	
	By DJB	Date 9/10/82	Ch'k'd RA	Date 8/82	Rev.	Date

Title: CONTAINMENT VESSEL, FORCE F_X ON HEMISPHERE, BIG ROCK POINT

<u>TOTAL TRANSVERSE FORCE</u> (ACTS ON EACH HEMISPHERE)

$$F_x = \int_{AREA} P(\phi) \sin^2\phi \cos\theta \, a^2 \, d\phi \, d\theta$$

$$= \int_{-\pi/2}^{\pi/2} \int_0^{\pi} P(\phi) \sin^2\phi \cos\theta \, a^2 \, d\phi \, d\theta - \int_{-\pi/2}^{-.2\pi} \int_{.2\pi}^{.8\pi} P(\phi) \sin^2\phi \cos\theta \, a^2 \, d\phi \, d\theta$$

$$= a^2 \left(\sin\theta \Big|_{-\pi/2}^{\pi/2}\right) \left[\int_0^{.25\pi} P_1 F_1 \, d\phi + \int_{.25\pi}^{.75\pi} P_2 F_2 \, d\phi + \int_{.75\pi}^{\pi} P_3 F_3 \, d\phi \right]$$

$$- a^2 \left(\sin\theta \Big|_{-\pi/2}^{-.2\pi}\right) \left[\int_{.2\pi}^{.25\pi} P_1 F_1 \, d\phi + \int_{.25\pi}^{.75\pi} P_2 F_2 \, d\phi + \int_{.75\pi}^{.8\pi} P_3 F_3 \, d\phi \right]$$

$$= 2a^2 (.0537 P_1 + .393 P_2 + .0537 P_3) - a^2(-.5878+1)(.0097 P_1 + .393 P_2 + .0097 P_3)$$

$$= 2a^2 P \left((.0537)(-1.0) + (.393)(1.2) + (.0537)(-.40)\right)$$

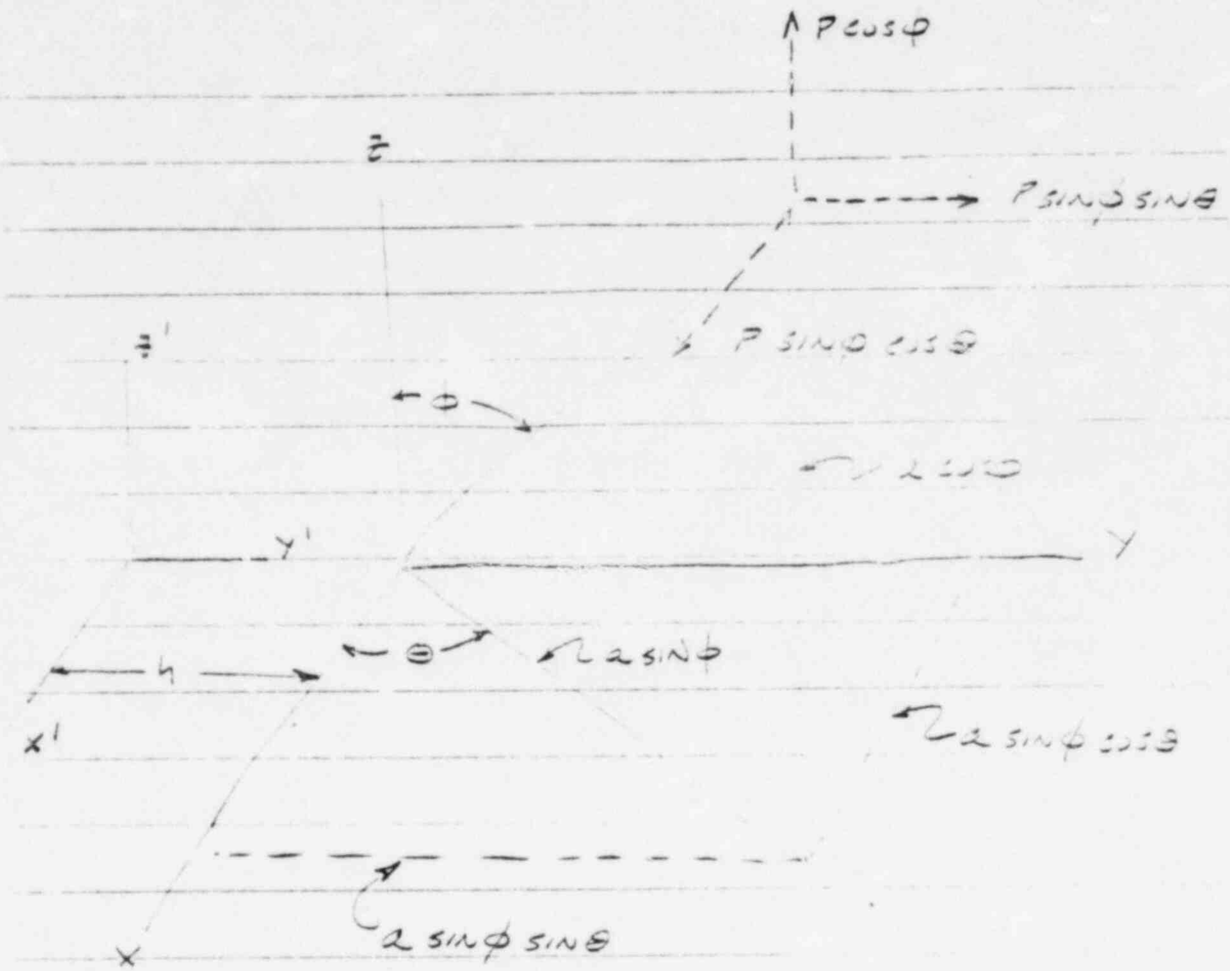
$$- .4122 \, a^2 \left((.0097)(-1.0) + (.393)(1.2) + (.0097)(-.40)\right)$$

$$= 1.99 a^2 P - .436 a^2 P$$

$$= 1.55 a^2 P$$

$$= .00393 a^2 V^2$$

Title: CONTAINMENT VESSEL, MOMENT STRESS RESULTANTS, BIG ROCK POINT



MOMENTS ABOUT EQUATOR (x-z PLANE)

$$M_x = \int_{AREA} (P\cos\phi \, a\sin\phi\sin\theta - P\sin\phi\sin\theta \, a\cos\phi)\, a^2 \sin\phi \, d\phi \, d\theta = 0$$

SAME FOR ALL MOMENTS

$$\bar{M} = \bar{R} \times \bar{P}$$

$$= \bar{R} \times \lambda\bar{R} = 0$$

NOTE: MOMENT = 0 IS INDEPENDENT OF $P(\phi)$ APPROXIMATION.

Franklin Research Center A Division of The Franklin Institute The Benjamin Franklin Parkway, Phila., Pa. 19103	Project: 026 - C5257-01				Page: III - 11	
	By: DJB	Date: 3/19/82	Ch'k'd: RA	Date: 8/82	Rev.	Date

Title: CONTAINMENT VESSEL, MOMENT M_x', BIG ROCK POINT

OVERTURNING MOMENT

$$M_x' = \int_{AREA} \left(P_{(\phi)} \cos\phi \,(a \sin\phi \sin\theta + h) - P_{(\phi)} \sin\phi \sin\theta \, a \cos\phi\right) a^2 \sin\phi \, d\phi \, d\theta$$

$$= \int_{AREA} P_{(\phi)} a^2 h \cos\phi \sin\phi \, d\phi \, d\theta$$

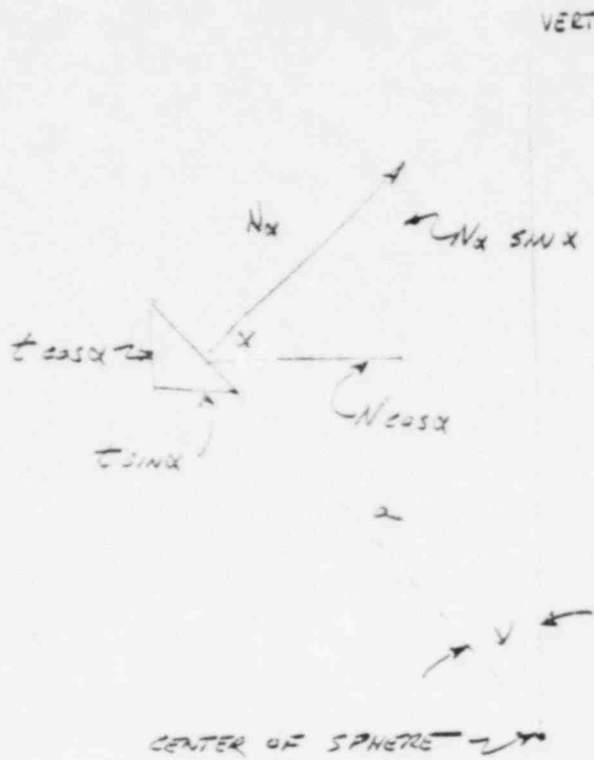
$$\approx \int_0^{2\pi} \int_0^{\pi} P_a^2 h \cos\phi \sin\phi \, d\phi \, d\theta - \int_{1.2\pi}^{1.8\pi} \int_{1.2\pi}^{.8\pi} P_a^2 h \cos\phi \sin\phi \, d\phi \, d\theta$$

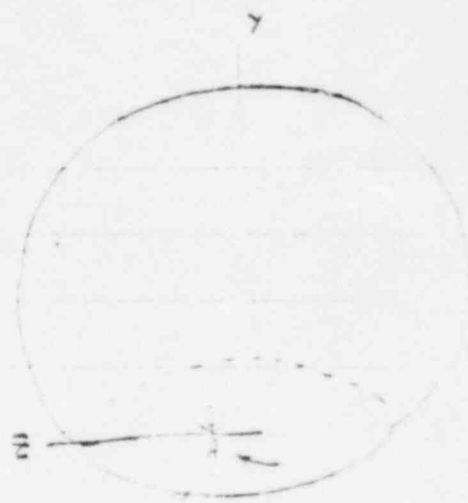
$$= F_z h$$

$$= -.000373 \, \pi h a^2 V^2$$

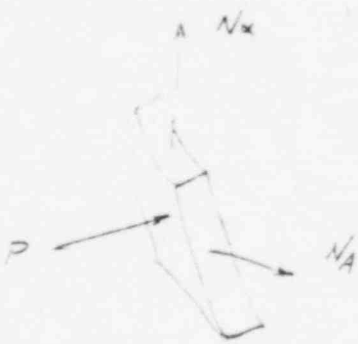
FORM 207 5M-4-80-CP

Title: CONTAINMENT VESSEL, BASE MEMBRANE STRESSES, BIG ROCK POINT





DEFINITION OF ANGLE A



$$\frac{N_\alpha}{a} + \frac{N_A}{a} = P$$

"EQUILIBRIUM EQUATION FOR MEMBRANE"

Franklin Research Center A Division of The Franklin Institute The Benjamin Franklin Parkway, Phila., Pa. 19103	Project: 026-C5257-01				Page: III-13
	By: DJB	Date: 9/20/82	Ch'k'd: GTB	Date: 9-83	Rev. / Date

Title: CONTAINMENT VESSEL, BASE MEMBRANE STRESSES, BIG ROCK POINT

FOR UPLIFT N_x AVERAGE IS CALCULATED

$$(N_x \sin\alpha)(2\pi a \sin\alpha) = F_y \implies N_x = \frac{F_y}{2\pi a \sin^2\alpha}$$

$$= \frac{.00437\, a^2 V^2}{2\pi a \sin^2\alpha}$$

$$= .000694\, a V^2 / \sin^2\alpha$$

FOR OVERTURNING N_x IS CALCULATED

$$\sigma = \frac{(-M)(a\sin\alpha \cos A)}{\pi (a\sin\alpha)^3 (t \sin\alpha)} = \frac{-M \cos A}{\pi a^2 t \sin^3\alpha} \implies N_x = \frac{-M \cos A}{\pi a^2 \sin^3\alpha}$$

$$\sigma = N_x / t$$

$$N_x = \frac{.000373\, \pi h a^2 V^2 \cos A}{\pi a^2 \sin^3\alpha}$$

$$= .000373\, h V^2 \cos A / \sin^3\alpha$$

FROM EQUILIBRIUM

$$N_A = P a - N_x$$

Title: CONTAINMENT VESSEL, BASE MEMBRANE STRESSES, BIG ROCK POINT

A. AT GRADE LEVEL $\alpha = 125.9°$ $\sin\alpha = .911$ $h = 30'$ $t = .774$ IN

UPLIFT $N_x = .000699 \times 65' \times V^2/(.911)^2 = .0691\, V^2$

OVERTURNING $N_x = .000373 \times 35' \times V^2 \cos A/(.911)^2 = .0266\, V^2 \cos A$

SUPERPOSITION $N_x = (.0691 + .0266 \cos A)\, V^2$

UPSTREAM $\phi = 55.8° \Rightarrow a\,P(\phi) = (65')(-1.0 \times .00256\, V^2)(\cos 71.6°) = -.0525\, V^2$

MIDDLE $\phi = 90° \Rightarrow a\,P(\phi) = (65')(1.2 \times .00256\, V^2)(\cos 180°) = -.20\, V^2$

DOWNSTREAM $\phi = 144.2° \Rightarrow a\,P(\phi) = (65')(-.40 \times .00256\, V^2)(\cos 288.4°) = -.0210\, V^2$

A.1 UPSTREAM END $A = 0°$ $\cos A = 1.0$

SUPERPOSITION

$N_\alpha = (.0691 + .0266)\, V^2 = .0957\, V^2$

$N_A = (-.0525\, V^2 - .0957\, V^2) = -.148\, V^2$

OVERTURNING

$N_x = .0266\, V^2$

$N_A = (-.0525 - .0266)\, V^2 = -.0791\, V^2$

STRESSES FOR $V = 250$ MPH

$N_\alpha = 5981$ #/FT OR 498 #/IN $\Rightarrow \sigma = 644$ PSI FROM SUPERPOSITION

$N_A = -9250$ #/FT OR -771 #/IN $\Rightarrow \sigma = -996$ PSI "

Title: CONTAINMENT VESSEL, BASE MEMBRANE STRESSES, BIG ROCK POINT

FOR UPLIFT N_x AVERAGE IS CALCULATED

$$(N_x \sin x)(2\pi a \sin x) = F_y \Rightarrow N_x = \frac{F_y}{2\pi a \sin^2 x}$$

$$= \frac{.00439\, a^2 Y^2}{2\pi a \sin^2 x}$$

$$= .000698\, a\, Y^2 / \sin^2 x$$

FOR OVERTURNING N_x IS CALCULATED

$$\sigma = \frac{(-M)(a \sin x \cos A)}{\pi (a \sin x)^3 (t \sin x)} = \frac{-M \cos A}{\pi a^2 t \sin^3 x}$$

$$\sigma = N_x / t$$

$$\Rightarrow N_x = \frac{-M \cos A}{\pi a^2 \sin^3 x}$$

$$N_x = \frac{.000373\, \pi\, h a^2 Y^2 \cos A}{\pi a^2 \sin^3 x}$$

$$= .000373\, h Y^2 \cos A / \sin^3 x$$

FROM EQUILIBRIUM

$$N_A = P a - N_x$$

Franklin Research Center A Division of The Franklin Institute The Benjamin Franklin Parkway, Phila., Pa. 19103	Project 026-C5257-01		Page III-14
	By DJB, Date 9/23/82	Ch'k'd GTR, Date 9-82	Rev. Date

Title: CONTAINMENT VESSEL, BASE MEMBRANE STRESSES, BIG ROCK POINT

A. AT GRADE LEVEL $\alpha = 125.9°$ $\sin\alpha = .8111$ $h = 38'$ $t = .774^{IN}$

UPLIFT $N_\alpha = .000699 \times 65' \times V^2/(.8111)^2 = .0691\,V^2$

OVERTURNING $N_\alpha = .000373 \times 38' \times V^2 \cos A/(.8111)^3 = .0266\,V^2 \cos A$

SUPERPOSITION $N_\alpha = (.0691 + .0266 \cos A)\,V^2$

UPSTREAM $\phi = 55.9° \Rightarrow a\,P(\phi) = (65')(-1.0 \times .00256\,V^2)/(\cos 71.6°) = -.0525\,V^2$

MIDDLE $\phi = 90° \Rightarrow a\,P(\phi) = (65')(1.2 \times .00256\,V^2)/(\cos 180°) = .20\,V^2$

DOWNSTREAM $\phi = 144.2° \Rightarrow a\,P(\phi) = (65')(-.40 \times .00256\,V^2)/(\cos 288.4°) = -.0210\,V^2$

A.1 UPSTREAM END $A = 0°$ $\cos A = 1.0$

SUPERPOSITION

$N_\alpha = (.0691 + .0266)\,V^2 = .0957\,V^2$

$N_A = (-.0525\,V^2 - .0957\,V^2) = -.148\,V^2$

OVERTURNING

$N_\alpha = .0266\,V^2$

$N_A = (-.0525 - .0266)\,V^2 = -.0791\,V^2$

STRESSES FOR $V = 250^{MPH}$

$N_\alpha = 5981\ ^{\#}/FT$ OR $498\ ^{\#}/IN \Rightarrow \sigma = 644^{PSI}$ (FROM SUPERPOSITION)

$N_A = -9250\ ^{\#}/FT$ OR $-771\ ^{\#}/IN \Rightarrow \sigma = -996^{PSI}$ "

FORM 207-5M-4-80-CP

Franklin Research Center A Division of The Franklin Institute The Benjamin Franklin Parkway, Phila., Pa. 19103	Project 026-C5257-01				Page III-15	
	By DJS	Date 9/23/82	Ch'k'd GTB	Date 9-82	Rev.	Date

Title CONTAINMENT VESSEL, BASE MEMBRANE STRESSES, BIG ROCK POINT

A.2 MIDDLE $A = 90°$ $\cos A = 0.0$

SUPERPOSITION

$N_x = .0691 V^2$

$N_A = (.120 - .0691) V^2 = .0509 V^2$

OVERTURNING

$N_x = 0$

$N_A = .120 V^2$

STRESSES FOR $V = 250$ MPH

$N_x = 4319$ #/FT OR 360 #/IN ⇒ $\sigma = 465$ PSI FROM SUPERPOSITION

$N_A = 7500$ #/FT OR 625 #/IN ⇒ $\sigma = 807$ PSI FROM OVERTURNING

A.3 DOWNSTREAM $A = 180°$ $\cos A = -1.0$

SUPERPOSITION

$N_x = (.0691 - .0266) V^2 = .0425 V^2$

$N_A = (-.0210 + .0425) V^2 = .0635 V^2$

OVERTURNING

$N_x = -.0266 V^2$

$N_A = (-.0210 + .0266) V^2 = .0056 V^2$

STRESSES FOR 250 MPH

$N_x = 2656$ #/FT OR 221 #/IN ⇒ $\sigma = 286$ PSI

$N_A = 3969$ #/FT OR 331 #/IN ⇒ $\sigma = 427$ PSI

FORM 207-5M-4-80-CP

Title: CONTAINMENT VESSEL, BASE MEMBRANE STRESSES, BIG ROCK POINT

B. AT 4' BELOW GRADE $\alpha = 130.3°$ $\sin\alpha = .7627$ $h = 43'$ $t = .774$ IN

UPLIFT $N_x = .000679 \times 65' \times V^2/(.7627)^2 = .0781\ V^2$

OVERTURNING $N_x = .000373 \times 42' \times V^2 \cos A/(.7627)^3 = .0353\ V^2 \cos A$

SUPERPOSITION $N_x = (.0781 + .0353 \cos A)\ V^2$

B.1 UPSTREAM END $A = 0°$ $\cos A = 1.0$

SUPERPOSITION $N_x = (.0781 + .0353)\ V^2 = .113\ V^2$

$N_A = -.113\ V^2$

OVERTURNING $N_x = .0353\ V^2$

$N_A = -.0353\ V^2$

B.2 MIDDLE $A = 90°$ $\cos A = 0.0$

SUPERPOSITION $N_x = .0781\ V^2$

$N_A = -.0781\ V^2$

OVERTURNING $N_x = 0.0$

$N_A = 0.0$

Franklin Research Center
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Project	Page
026-C5357-01	III-17

By	Date	Ch'k'd	Date	Rev.	Date
DJB	9/23/82	STB	9-82		

Title: CONTAINMENT VESSEL, BASE MEMBRANE STRESSES, BIG ROCK POINT

8.3 DOWNSTREAM $\quad A = 180^\circ \quad \cos A = -1.0$

SUPERPOSITION $\quad N_x = (.0781 - .0353)V^2 = .0428\ V^2$

$N_A = -.0428\ V^2$

OVERTURNING $\quad N_x = -.0353\ V^2$

$N_A = .0353\ V^2$

STRESSES FOR $V = 250$ MPH

$N_x = 7062$ #/FT OR 539 #/IN $\Rightarrow \sigma = 760$ PSI

$N_A = -7062$ #/FT OR -539 #/IN $\Rightarrow \sigma = 760$ PSI

FORM CS-FRC-81

Franklin Research Center A Division of The Franklin Institute The Benjamin Franklin Parkway, Phila., Pa. 19103	Project 026-C5257-01				Page III-13	
	By DJB	Date 9/23/82	Ch'k'd GTB	Date 9-8?	Rev.	Date

Title: CONTAINMENT VESSEL, BASE MEMBRANE STRESSES, BIG ROCK POINT

C. AT 8' BELOW GRADE $\alpha = 135.05°$ $\sin\alpha = .7065$ $h = 96'$ $t = .77$ IN

UPLIFT $N_x = .000699 \times 65 \times V^2/(.7065)^2 = .0910\,V^2$

OVERTURNING $N_x = .000373 \times 96 \times V^2 \cos A/(.7065)^3 = .0497\,V^2 \cos A$

SUPERPOSITION $N_x = (.0910 + .0497 \cos A)\,V^2$

C.1 UPSTREAM END $A = 0°$ $\cos A = 1.0$

SUPERPOSITION $N_x = (.0910 + .0497)\,V^2 = .140\,V^2$

$N_A = -.140\,V^2$

OVERTURNING $N_x = .0497\,V^2$

$N_A = -.0497\,V^2$

C.2 MIDDLE $A = 90°$ $\cos A = 0.0$

SUPERPOSITION $N_x = .0910\,V^2$

$N_A = -.0910\,V^2$

OVERTURNING $N_x = 0.0$

$N_A = 0.0$

FORM CS-FRC-81

C.3 <u>DOWNSTREAM</u> $A = 180°$ $\cos A = -1.0$

<u>SUPERPOSITION</u> $N_x = (1.09 \cdot .0487) V^2 = .0423 V^2$

$N_A = -.0423 V^2$

<u>OVERTURNING</u> $N_x = -.0487 V^2$

$N_A = .0487 V^2$

<u>STRESSES</u> FOR $V = 250$ MPH

$N_x = 8750$ #/FT OR 729 #/IN $\Rightarrow \sigma = 942$ PSI

$N_A = -8750$ #/FT OR -729 #/IN $\Rightarrow \sigma = -942$ PSI

Franklin Research Center A Division of The Franklin Institute The Benjamin Franklin Parkway, Phila., Pa. 19103	Project: 026-C5257-01				Page: III-20	
	By: DJB	Date: 8/23/82	Ch'k'd: GTB	Date: 9-82	Rev.	Date

Title: CONTAINMENT VESSEL, BASE MEMBRANE STRESSES, BIG ROCK POINT

CHECK MAGNITUDE OF N_A AVERAGE (ON CENTER MERIDIAN)

$$F_x = .00399\, a^2 V^2$$

$$\text{AREA} = 1.4\pi\, a$$

$$\Rightarrow \quad N_{A\,AVG} = \frac{.00399}{1.4\pi}\, a V^2 = .0588\, V^2$$

FOR $V = 250$ MPH

$N_A = 3675$ #/FT OR 306 #/IN $\Rightarrow$ $T = 383$ PSI AT BASE

FORM CS-FRC-81

Franklin Research Center A Division of The Franklin Institute The Benjamin Franklin Parkway, Phila., Pa. 19103	Project 026-C5257-01		Page III-21
	By DJB Date 9/14/82	Ch'k'd GTB Date 9/16/82	Rev. Date

Title CONTAINMENT VESSEL, BASE MEMBRANE STRESSES, BIG ROCK POINT

STRESS COEFFICENTS

	WINDWARD		MIDDLE		LEEWARD	
LEVEL	N_x *	N_A *	N_x	N_A	N_x	N_A
GRADE	.0957	-.149	.0691	.0509	.0425	.0635
4' BELOW	.113	-.113	.0781	-.0781	.0423	-.0023
8' BELOW	.140	-.140	.0910	-.0910	.0423	-.0229

* COEFFICENT × V^2 = STRESS #/FT

Title: CONTAINMENT VESSEL, BASE MEMBRANE STRESSES, BIG ROCK POINT

LOADING COMBINATION: WIND PLUS DEAD LOAD

	N_x	N_A	N_x	N_A			
	WINDWARD #/IN		DEAD LOAD (METAL AND INSULATION)		BIAXIAL	UNIAXIAL	BI + UNI *
GRADE	448	-771	-443	558	443/692 = .64	328/1384 = .24	.88
4' BELOW	539	-589	-517	633	517/692 = .75	[illegible]/1384 = .05	.80
8' BELOW	729	-729	-628	755	628/692 = .91	101/1384 = .07	.98
	MIDDLE #/IN **						
GRADE	560	265	-443	558	—	—	—
4' BELOW	407	-407	-517	633	407/692 = .59	110/1384 = .08	.67
8' BELOW	474	-474	-628	755	474/692 = .68	152/1384 = .11	.79
	LEEWARD #/IN						
GRADE	221	331	-443	558	—	—	—
4' BELOW	223	-223	-517	633	223/692 = .32	294/1384 = .21	.53
8' BELOW	220	-220	-628	755	220/692 = .32	408/1384 = .30	.62

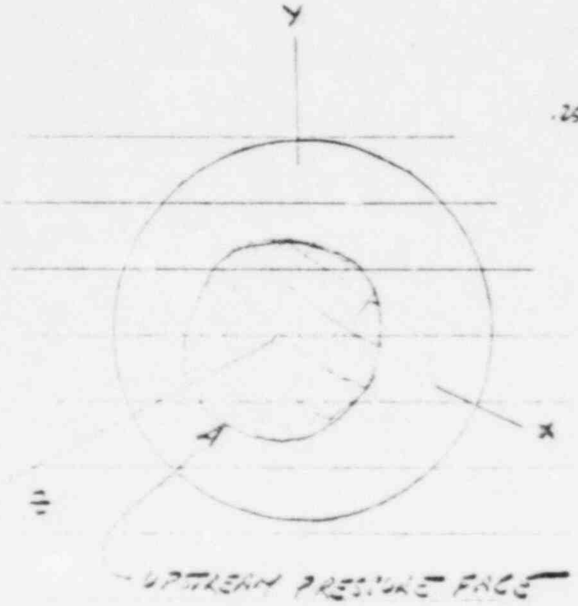
BIAXIAL COMPRESSION CRITERIA $900{,}000\ t^2/R = 692$ #/IN

UNIAXIAL COMPRESSION CRITERIA $1800000\ t^2/R = 1384$ #/IN

* NEGLECTING TENSILE STRESSES AND NOT APPLYING EXTREME ENVIRONMENTAL FACTOR

** ESTIMATES LEVEL OF INTERACTION WITHOUT SHEAR STRESS COMPONENT

Title: CONTAINMENT VESSEL, UPSTREAM FACE STRESSES, BIG ROCK POINT



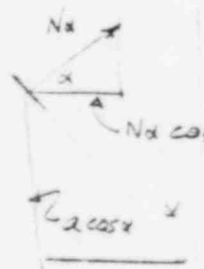
$$.25 * L_z = \int_0^{\pi/2}\int_0^{\pi/4} P(\phi)\cos\phi \sin\phi \, a^2 \, d\phi \, d\theta$$

$$= \tfrac{1}{2} a^2 \pi \int_0^{\pi/4} P \cos 2\phi \cos\phi \sin\phi \, d\phi$$

$$= \tfrac{1}{2} a^2 \pi \, (-.125) P$$

$$= -.0625 \, a^2 \pi P$$

$$\therefore L_z = -.25 a^2 \pi P = -.00064 \, a^2 \pi V^2$$



$$(N\alpha \cos\alpha)(2\pi a \cos\alpha) = L_z$$

$$\therefore N\alpha = \frac{-.00064 \, \pi a^2 V^2}{2\pi a \cos^2\alpha} = -.00032 \, a \, V^2/\cos^2\alpha$$

FOR $\alpha = 45°$ $t = .709^{IN}$

$$N\alpha = -.00032 \times 65 \times V^2/(.7071)^2 = -.0416 \, V^2$$

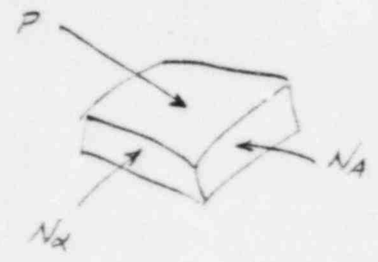
FOR $V = 250^{MPH}$

$N\alpha = -2600$ #/FT OR -217 #/IN $\Rightarrow \sigma = -306^{PSI}$

Franklin Research Center A Division of The Franklin Institute The Benjamin Franklin Parkway, Phila., Pa. 19103	Project 026-C5357-01		Page III-27
	By DJB Date 3/24/82	Ch'k'd ETB Date 9-82	Rev. Date

Title: CONTAINMENT VESSEL, UPSTREAM FACE STRESSES, BIG ROCK POINT

CENTER OF PRESSURE



$$\left.\begin{array}{l} N_\alpha + N_A = P_a \\ \text{AT CENTROID } N_\alpha = N_A \end{array}\right] \Longrightarrow N_\alpha = N_A = \frac{P_a}{2}$$

$$\therefore N_\alpha = N_A = \frac{-.00256 \times 65 \times V^2}{2} = -.0832\,V^2$$

FOR $V = 250^{MPH}$ $\quad \alpha = 90^\circ$ $\quad t = .719^{IN}$

$N_\alpha = -5200^{\#}/FT$ OR $-433^{\#}/IN \Longrightarrow \sigma = -603^{PSI}$

FORM CS-FRC-81

Franklin Research Center A Division of The Franklin Institute The Benjamin Franklin Parkway, Phila., Pa. 19103	Project 026-05257-01		Page III-25
	By DJB Date 9/24/82	Ch'k'd Date 9/28/82	Rev. Date

Title: CONTAINMENT VESSEL, UPLIFT FORCE L_y, BIG ROCK POINT

UPLIFT FORCE ON UPPER HEMISPHERE

$$L_y = \int_0^{\pi}\int_0^{\pi} P(\phi)\sin^2\phi \sin\theta \, a^2 \, d\phi \, d\theta$$

$$= 2a^2 \int_0^{\pi} P(\phi)\sin^2\phi \, d\phi$$

$$= 2a^2 \left[\int_0^{\pi/4} P_1 \cos 2\phi \sin^2\phi \, d\phi + \int_{\pi/4}^{3\pi/4} P_2(-\cos 2\phi)\sin^2\phi \, d\phi + \int_{3\pi/4}^{\pi} P_3 \cos 2\phi \sin^2\phi \, d\phi \right]$$

$$= 2a^2(.0537 P_1 + .393 P_2 + .0537 P_3)$$

$$= 2a^2 P\left((.0537)(-1.0) + (.393)(1.2) + (.0537)(-.40)\right)$$

$$= 1.99 \, a^2 P$$

$$= .00510 \, a^2 l^2$$

FORM CS-FRC-81

Franklin Research Center
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Project				Page
026-C5257-01				III-26
By: DJB	Date: 9/14/82	Ch'k'd: [illegible]	Date: [illegible]	Rev. / Date

Title: CONTAINMENT VESSEL, EQUATOR STRESSES, BIG ROCK POINT

UPLIFT EQUATOR STRESSES (AVERAGE) SECTION #5-#6 $t = .719^{IN}$

$$N_\alpha = \frac{.00510\, a^2 V^2}{2a\pi} = .000812\, a V^2 = .0528\, V^2$$

UPSTREAM $\phi = 0° \Rightarrow a P(\phi) = 165'(-1.0 \times .00256 V^2)(\cos 0°) = -.166 V^2$

MIDDLE $\phi = 90° \Rightarrow a P(\phi) = (165')(1.2 \times .00256 V^2)(\cos 90°) = .20 V^2$

DOWNSTREAM $\phi = 180° \Rightarrow a P(\phi) = (165')(-.40 \times .00256 V^2)(\cos 180°) = -.0676 V^2$

AND $N_A = a P_\phi - N_\alpha$

$\therefore$ WORST CASE $= (-.166 - .0528) V^2 = -.219 V^2$

STRESSES FOR 250 MPH WIND

$N_\alpha = 3300$ #/FT OR 275 #/IN $\Rightarrow \sigma = 382$ PSI

$N_A = -13688$ #/FT OR -1140 #/IN $\Rightarrow \sigma = -1586$ PSI

MODIFICATIONS

DEAD WEIGHT $N_A = 143 + 7 = 150$ #/IN

PRESSURE ZONE $(2a^2 P)(-.0537) = -.000275\, a^2 V^2$ "LOCAL DOWNWARD FORCE"
(LOCAL EFFECTS)
$\therefore N_\alpha = (-.000275\, a^2 V^2)/2a\pi = -.00284\, V^2 \xrightarrow{V = 250\,MPH} 178$ #/FT

$\therefore N_A = -1140$ #/IN $+ 150$ #/IN $+ 14.8$ #/IN $= -975$ #/IN $\Rightarrow \sigma = -1356$ PSI

FORM CS-FRC-81

Franklin Research Center
A Division of The Franklin Institute
The Benjamin Franklin Parkway, Phila., Pa. 19103

Project	Page
026-C5357-01	III-27

By	Date	Ch'k'd	Date	Rev.	Date
DJB	9/14/82	GTB	9/16/82		

Title: CONTAINMENT VESSEL, BASE MEMBRANE STRESSES, BIG ROCK POINT

STRESS COEFFICIENTS

	Nx	NA
WINDWARD	.0528	-.219
MIDDLE	.0528	.20-.0528 = .147
LEEWARD	.0528	-.0666-.0528 = -.119

LOADING COMBINATION: WIND PLUS DEAD LOAD

LEVEL	WIND LOAD		DEAD LOAD	
#5-#8	Nx	NA	Nx	NA
	275	-1140	-162	150
	275	765	-162	150
	275	-620	-162	150

BIAXIAL COMPRESSION CRITERIA 900,000 t²/R = 597 #/IN

UNIAXIAL COMPRESSION CRITERIA 1,800,000 t²/R = 1194 #/IN

WORST CASE $\frac{162}{597} + \frac{978}{1194} = .27 + .82 = 1.09$ OK WITH ALLOWABLE STRESS INCREASES.

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