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TECHNICAL EVALUATION REPORT

WIND AND TORNADO LOADINGS (SEP, III-2)

YANKEE ATOMIC ELECTRIC COMPANY
YANKEE ROWE NUCLEAR POWER STATION

NRC DOCKET NO. 50-029

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FRC PROJECT C5257

FRC ASSIGNMENT 14

FRC TASK 407

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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 this review is to provide a technical evaluation of the SAR prepared by the Yankee Atomic Electric Company (YAEC) for the Yankee Rowe Nuclear Power Station [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 concerns 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, the 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 Yankee Rowe SAR was begun in April 1982. Prior to that time, Yankee Atomic Electric Company (YAEC) responded to NRC requests for information by providing architectural-engineering structural drawings. Additional sources of information were a YAEC letter on the SEP structural topics [3] and the plant final safety analysis report [4]. The steel vapor container, the turbine building, the primary auxiliary building, and the diesel generator building are the safety-related structures that have been evaluated by YAEC and reviewed in the Yankee Rowe SAR. (The conclusions of this report are summarized in Table 1.)

The original wind loading criteria of the Yankee Rowe structural systems were the structural load provisions of the American Standard Association code A58.1-1955 [5]. These provisions called for a graded wind load of 20 psf at an elevation less than 30 ft; 25 psf at an elevation between 50 ft and 99 ft; and 40 psf at an elevation between 100 ft and 499 ft. According to attachment B of Reference 3, the vapor container had been designed to withstand loads corresponding to windspeeds of 100 mph from any direction. The criteria for the review in this TER are stated in Section 2 of this report.

FRC was originally charged with auditing the design calculations supporting the conclusions of the Yankee Rowe SAR. However, these calculations were not provided by YAEC. Under a change in work scope for Assignment 14 but within the original budget and schedule constraints, FRC is to perform an independent tornado analysis for a limited sample of the Yankee Rowe Class I structures and components. The FRC 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 and in the Appendices). The results of this additional analysis are to be used to assess the conclusions reported in the SAR.

Table 1. Summary of Conclusions from Yankee Rowe Topic III-2 SAR*

Safety-Related	System	Wind Velocity (mph)
Steel Vapor Container	Structural	252
	Skin	>252
Turbine Building	Structural	158
	Skin	64
	Roof	161
Primary Auxiliary Building	Structural	192
	Skin	40
	Roof	165
Diesel Generator Building	Roof	190
	Skin	46

*Notes:

1. The effect of atmospheric pressure changes and tornado-borne missiles are not considered.
2. Steady air flow is assumed.
3. Specific details on the metal siding and decking are not available; capacities are based on strength of supporting members.

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 Yankee Rowe 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 Yankee Rowe 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 Article CC-3000 in the ASME Boiler and Pressure Vessel Code [16] and the 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 3.8.1, "Concrete 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 2 (ACI-359), "Standard Code for Concrete Reactor Vessels and Containments" [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 primary auxiliary building, the diesel generator building, the control room and the steel vapor container supports. These structures are classified seismically as Category I Nuclear Safety Related. The plan of the building arrangement at the Yankee Rowe site is shown in Figure 1.

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

Maximum wind speed	300 mph
Maximum pressure drop	2.25 psi
Rate of pressure drop	1.2 psi/sec
Core radius	150 ft.

These characteristics yield a dynamic pressure of 230 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). 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 venting, the tornado loads are still assumed to exist so that the strength of other, stronger structural elements can be analyzed.

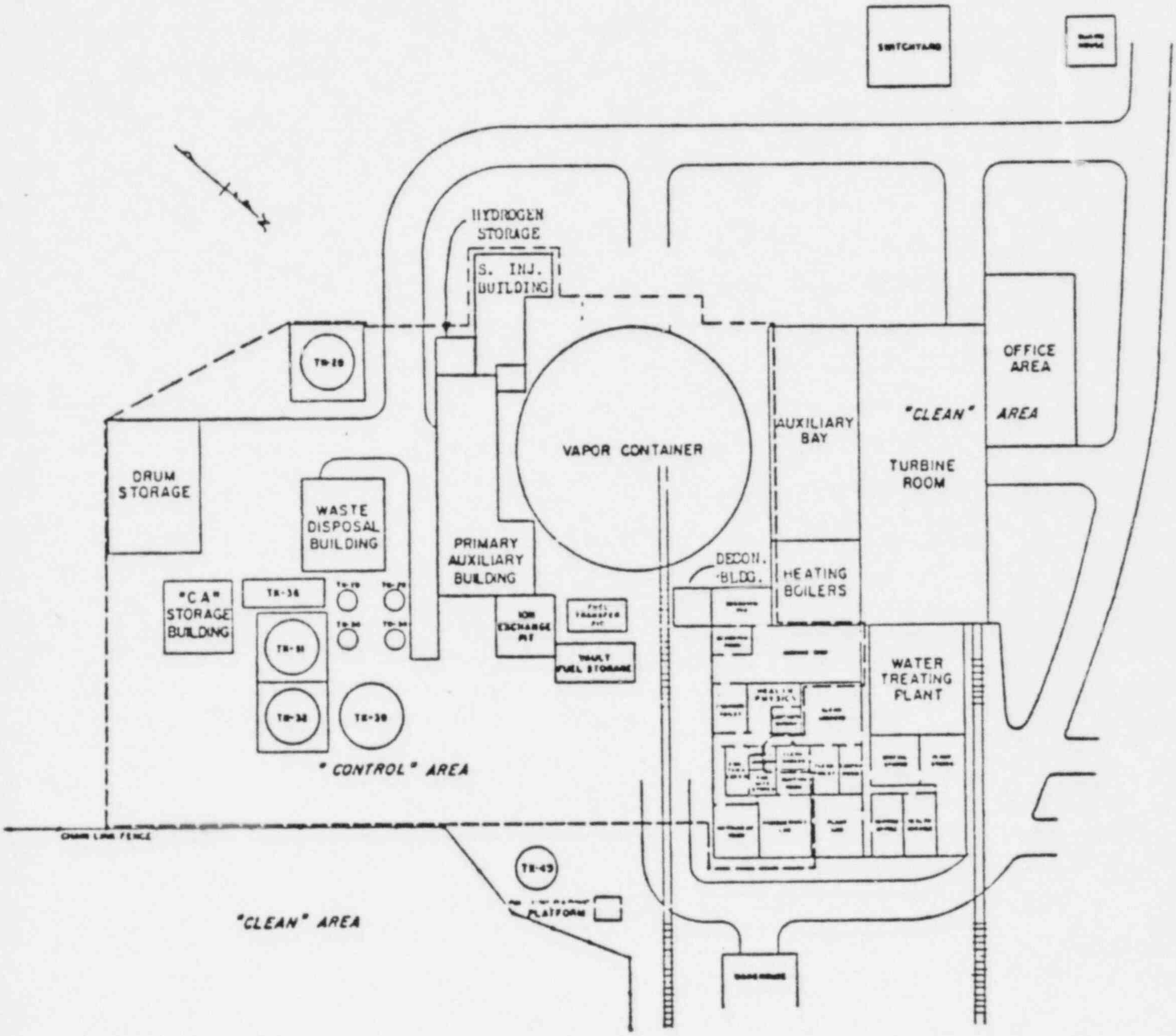

Figure 1. Site Plot Plan

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 VAPOR CONTAINER

3.2.1 Evaluation

The vapor container is a 125-ft diameter steel sphere with a skin thickness varying from 7/8 in to 3 inches. The sphere is suppoprted by 16 hollow steel columns with a diameter of 42 inches and a 7/8-inch wall thickness. The columns are connected at the equator of the sphere 85 feet 6 inches above the grade elevation of 1022 ft. The columns are interconnected and braced by 4-inch diameter steel rods.

The vapor container houses a concrete structure which supports the reactor and acts as a radiation shield. The internal concrete structure is supported by eight concrete columns and is isolated from the vapor container by bellows located where the concrete columns pass through the steel shell.

During a tornado strike, the steel sphere will be subjected to lateral loads which will induce bending moments and axial loads in the supporting steel columns. These columns are modeled as unbraced cantilevered posts which are checked for capacity and stability. The assumptions underlying the distribution of lateral force and the column analysis are given in Appendix A.

3.2.2 Conclusions

The supporting columns of the vapor container can safely withstand the tornado loadings.

3.3 PRIMARY AUXILIARY BUILDING

3.3.1 Evaluation

The primary auxiliary building is located on the south side of the vapor container. The walls and floors of the primary auxiliary building substructure are reinforced concrete. The superstructure is braced steel framing with a high roof at elevation 1065 ft 3 1/2 in and a low roof at elevation 1056 ft 6 in. The roof is 2 1/2 in steel decking, and the walls are 8-in-thick concrete blocks connected to the steel columns by strap anchors. The adjacent grade varies from elevation 1022 ft on the north side to elevation 1035 ft on the south side.

The wind loads are applied to the structure through the concrete block wall-column connections. The columns transmit this load to the roof steel and to the substructure. Bracing in the roof steel plan transmits the lateral forces to vertically braced frames.

The concrete block walls are analyzed as simply supported between floor levels. Since no steel reinforcement has been provided, equilibrium is maintained by tensile forces in the masonry bond. For the extreme environmental loading, strength factors are applied to the allowable stresses.

The steel columns are modeled as simply supported and are subjected to combined bending and axial loads. The roof steel consists of beams running in the north-south direction supported by girders spanning in the east-west direction. The roof deck is assumed to remain intact with the roof beams and to transmit uplift loads to these beams. All design review calculations for this structure can be found in Appendix B.

3.3.2 Conclusion

The limiting members of the primary auxiliary building are the unreinforced concrete block walls which have limit ratings of 0.09 psi (49 mph) for differential pressure, 15.3 psf (77 mph) for tornado dynamic pressure and 12.2 psf (56 mph) for high wind dynamic pressure. The 8W31 steel columns have a limit rating of 0.94 psi (162 mph) for differential pressure and 168 psf (256 mph) for tornado dynamic pressure. The limiting elements of the roof steel are the 12W27 beams which have a limit rating of 1.19 psi (183 mph) for differential pressure; the 10W21 beams which are rated at 1.29 psi (191 mph) for differential pressure; and the 16W36 girder which has a limit rating of 0.60 psi (130 mph) for differential pressure and 123 psf (219 mph) for tornado dynamic pressure.

3.4 DIESEL GENERATOR BUILDING

3.4.1 Evaluation

The diesel generator building is located south-west of the vapor container and is adjacent to the primary auxiliary building. The walls of the diesel generator building are 8-in-thick concrete blocks connected to steel columns by vertical and horizontal standard masonry galvanized iron anchors. The columns support the roof steel and the 1 1/2-inch steel roof deck. The top of the steel of the diesel generator building is at elevation 1037 ft 2 inch, but the accumulator tank room roof is at elevation 1056 ft 3 1/2 inch. The adjacent grade is at elevation 1022 ft.

The roof steel beams, steel columns, and concrete block walls are examined. The concrete walls are analyzed for bending and the roof beams are studied under uplift. The columns are subjected to axial and lateral loads. The design review calculations can be found in Appendix C.

3.4.2 Conclusion

The limiting members of the diesel generator building are the unreinforced concrete block walls which have limit ratings of 0.11 psi (55 mph) for differential pressure, 19.5 psf (87 mph) for tornado dynamic pressure, and 15.5 psf (63 mph) for high wind dynamic pressure. The limiting 8W17 steel column has a rating of 0.26 psi (85 mph) for differential pressure, 46.2 psf (134 mph) for tornado dynamic pressure, and 15.1 psf (80 mph) for high wind dynamic pressure. The limiting elements of the roof steel are the 8W17 beams which have a limit rating of 1.22 psi (186 mph) for differential pressure and 220 psf (313 mph) for tornado dynamic pressure; the 10W21 beams which have ratings of 0.88 psi (157 mph) for differential pressure and 181 psf (266 mph) for tornado dynamic pressure; and the 18W50 girder which has a limit rating of 0.33 psi (96 mph) for differential pressure, 67.9 psf (163 mph) for dynamic pressure, and 46.6 psf (133 mph) for high wind dynamic pressure.

3.5 CONTROL ROOM

3.5.1 Evaluation

The control room is located inside the auxiliary building, which is north of the vapor container. The control room floor slab is at elevation 1052 ft 8 in, while the roof slab is at elevation 1070 ft 2 in. The adjacent grade elevation is 1021 ft. The south wall of the auxiliary building is 3 ft thick reinforced concrete up to elevation 1052 ft 5 inches, where, at the control room level, the wall thickness increases to 4 ft. The east and west walls are 4 ft thick reinforced concrete between elevations 1050 ft 8 in and 1066 ft. The west and south walls are exposed to the atmosphere. The east wall is common with the heating boiler room, while the north wall is common with the turbine building.

The east and west control room walls are supported on reinforced concrete piers. The area bounded by the piers is enclosed by concrete block. For the loadings considered in this review, the concrete block walls are structurally independent of the piers and the reinforced concrete walls. The reinforced concrete roof of the control room is a series of 3-ft-deep precast concrete beams supporting a roof slab.

The west side wall transfers lateral loads to the concrete piers and is examined in bending. The piers are subjected to bending by the lateral tornado loads and are modeled as cantilever beams fixed at grade level. The review calculations and analysis can be found on pages D-1 to D-11 of Appendix D.

3.5.2 Conclusion

The west side reinforced concrete wall can withstand all of the tornado loadings. The limiting elements are the concrete piers which have a rating of 0.51 psi (120 mph) for differential pressure and 91.6 psf (189 mph) for tornado dynamic pressure.

4. CONCLUSIONS

The results of the tornado structural analysis for the vapor container supports, the primary auxiliary building, the diesel generator building, and the control room are summarized in Table 2.

Table 2. Strength Summary of the Structural Components Analyzed[a]

Structure	Element[b]	Cause of Failure[c]	Wind Speed (mph)
Vapor Container	Supporting Steel Columns	--	--
Primary Auxiliary Building	Concrete Block Wall[d]	2	49
		3	56
		1	77
	8W31 Columns	2	162
		1	256
	16W36 Roof Girders	2	130
		1	219
	12W27 Roof Beams	2	183
	10W21 Roof Beams	2	191

a. The ratings of some structural components are not definitive but are estimates based on approximate modeling.

b. Note that this table does not imply that all inadequate elements have been identified or that the most limiting element of the design has been found. Structural details not included in this review are windows, doors, and roof decks.

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

d. The concrete block wall ratings are given for tension stress normal to bed joint in unreinforced block walls.

Table 2 (Cont.)

Structure	Element[b]	Cause of Failure[c]	Wind Speed (mph)
Diesel Generator Building	Concrete Block Wall (d)	2	55
		3	63
		1	87
	8W17 Columns	3	80
		2	85
		1	134
	18W50 Roof Girder	2	96
		3	133
		1	163
	10W21 Roof Beam	2	157
		1	266
	8W17 Roof Beams	2	186
		1	313
Control Room	Reinforced Concrete Piers	2	120
		1	189
	Reinforced Concrete Wall Between Elevations 1052 ft and 1066 ft	--	--

While not specifically reviewed, additional areas of concern are the waste disposal and fuel storage buildings. The north wall of the control room, while not exposed to the atmosphere, is masonry block construction which will be subjected to differential pressure loadings with failure of the siding and the roof deck of the turbine building. And as a final note, if the primary ventilation stack were to fail and fall, it would have the potential to impact safety-related structures.

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ACI 307-79

APPENDIX A

STEEL VAPOR CONTAINER

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- Weight of VAPOR CONTAINER Sphere = 11,500 KIPS (MAX. WEIGHT)
- No. of STEEL Columns = 16

Axial Weight on each Column = $\frac{11500}{16}$

$P = 718.75$ KIPS

OUTER RADIUS of Column = 21" $\quad\therefore\quad r_o = 21"$

WALL Thickness = 7/8" $\quad\quad r_i = 20.125"$

$$\text{Area} = A = \pi (r_o^2 - r_i^2)$$

$$= \pi ((21)^2 - (20.125)^2)$$

$$\underline{A = 113.0483 \text{ in}^2}$$

$\therefore$ Axial STRESS, f_a,

$$= \frac{P}{A} = \frac{718.75}{113.0483} = \underline{6.358 \text{ Ksi}}$$

TAKE UNBRACED Length AS DISTANCE Between W.P. TO W.P. = 70' 11"

Hollow Circle,

$$I = \pi/4 \, (r_o^4 - r_i^4)$$

RADIUS of GYRATION, R,

$$= \sqrt{\frac{I}{A}}$$

$$R = \frac{\sqrt{r_o^2 + r_i^2}}{2} = \frac{\sqrt{(21)^2 + (20.125)^2}}{2}$$

$\underline{R = 14.543 \text{ IN.}}$

TAKE $k = 1.0$

$$\frac{kl}{R} = \frac{(1.0)[(70)(12) + 11]}{14.543} = \underline{58.516}$$

$$C_c = \sqrt{\frac{2\pi^2 E}{F_y}} = 133.75$$

where $F_y = 32$ ksi

$$\left(\frac{kl}{R} / C_c\right) = \underline{.438}$$

$$F_a = \frac{\left[1 - \frac{1}{2}\left(\frac{kl}{R}/C_c\right)^2\right] F_y}{\frac{5}{3} + \frac{3}{8}\left(\frac{kl}{R}/C_c\right) - \frac{1}{8}\left(\frac{kl}{R}/C_c\right)^3}$$

$$\therefore F_a = \frac{28.93}{1.747} = 16.56^{\text{KSI}}$$

$$\therefore \frac{f_a}{F_a} = \frac{6.358}{16.56} = \underline{.384}$$

ASSUME $F_b = 0.6 \times 32 = 19.2$

$$\frac{f_a}{F_a} + \frac{f_b}{(1 - f_a/F_e')F_b} \leq 1.0$$

$$F_e' = \frac{12\pi^2 E}{23(kl/R)^2} = \underline{43.611}$$

$$f_b = \left(1 - \frac{f_a}{F_a}\right)\left(1 - \frac{f_a}{F_e'}\right) F_b$$

$$= (.616)(.854)(19.2)$$

$$\underline{f_b = 70.10 \text{ ksi}}$$

For EACH Column,

$$I = \pi/64 \, (d_o^4 - d_i^4)$$ where $d_o = 42''$

$$= \pi/64 \, [(42)^4 - (40.25)^4]$$ $d_i = 40.25''$

$$= \pi/64 \, [3111696.0 - 2624602.5)$$

$$= \pi/64 \, [487093.5]$$

$$\underline{I = 23910.146 \text{ in}^4}$$

BENDING CAPACITY OF COLUMN,

$$M = \frac{f_b I}{y}$$

$$= \frac{(10.10)(23910.146)}{21}$$

$M = 11500.13$ K-in.

$M = 958.34$ K-ft.

CALCULATION OF LOADING ON EACH COLUMN,

— OUTER DIAMETER OF SPHERE = 125 ft.

$$\text{PROJECTED AREA} = \frac{\pi d^2}{4}$$

$$= 12271.846 \text{ ft}^2$$

SUPPOSE SUPPORTS ARE GOOD for w psf,

$R = (6380)(20)(22)(125)$

$= 3.509 \times 10^8$

Force = 12271.846 w lB.

Drag Coeff. = 0.2

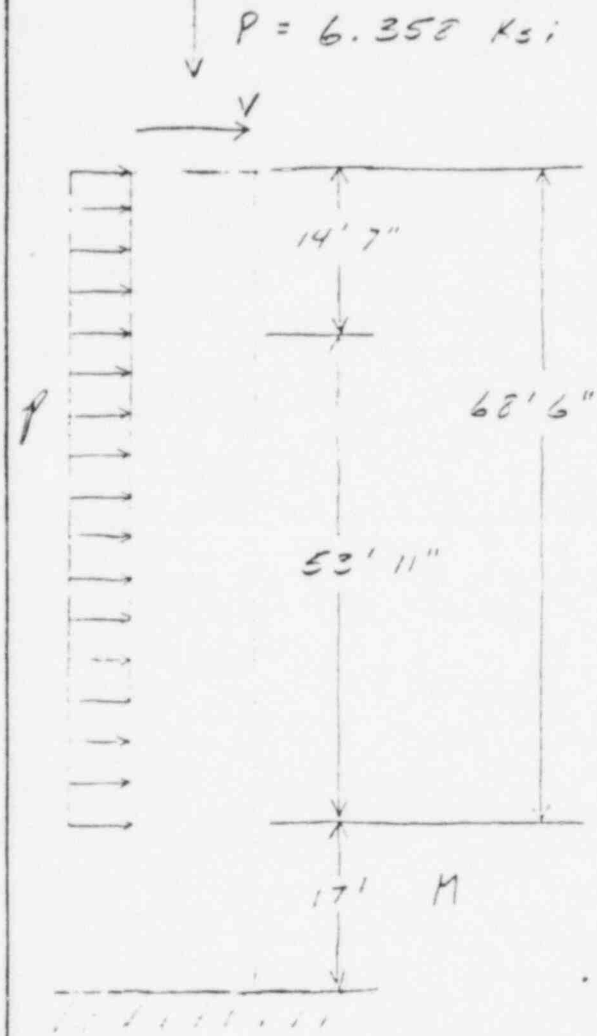
$F = C_D \cdot q \cdot A.$

-Shear on each Column,

$$V = \frac{Force}{16} = \frac{12271.846\, w}{16}$$

$\therefore V = 766.99w$ lbs

$V = 0.76699\, w$ Kips



$V = 0.76699\,w$ Kips

$p = D_o \cdot w$

$= 3.5w$ lbs/ft

$= 0.0035w$ Kips/ft

$$M = \frac{p l^2}{2} + V \cdot l$$

$$= \frac{0.0035w(68.5)^2}{2} + 0.76699w(68.5)$$

$$= 8.211w + 52.539w$$

$$\therefore M = 60.75w$$

$$M_{Allow} = 958.34$$

$$\therefore w = \frac{958.34}{60.75}$$

$w = 15.775$ ksf

APPENDIX B

PRIMARY AUXILIARY BUILDING DESIGN REVIEW CALCULATIONS

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Franklin Research Center A Division of The Franklin Institute The Benjamin Franklin Parkway, Phila., Pa. 19103	Project 02G-C5257-01				Page B-1	
	By RA	Date JUNE '82	Ch'k'd MLP	Date 6/82	Rev.	Date

Title PRIMARY AUXILIARY BUILDING — YANKEE ROWE

THE PRIMARY AUXILIARY BUILDING IS BASICALLY A STRUCTURAL STEEL STRUCTURE WITH CONCRETE BLOCK WALLS. IN THIS ANALYSIS ROOF DECKING, ROOF BEAM, STEEL COLUMNS AND CONCRETE BLOCK WALLS HAVE BEEN EXAMINED.

WE ASSUME THAT CONCRETE BLOCK WALL TRANSFERS ALL THE LOAD IN THE LATERAL DIRECTION ON TO STEEL COLUMNS BEFORE IT FAILS. NO DETAILS & PROPERTIES ARE AVAILABLE FOR CONC. BLOCK WALL. ASSUMED SECTION PROPERTIES HAVE BEEN USED IN THE ANALYSIS FOR CONC. BLOCK WALL AND STEEL ROOF DECKING ALSO.

ROOF STEEL BEAM & STEEL DECKING ANALYZED FOR UPLIFT PRESSURE CASE. EVEN AFTER FAILURE, DECKING ASSUMED TO BE TRANSFERRING LOAD TO BEAMS.

Franklin Research Center A Division of The Franklin Institute The Benjamin Franklin Parkway, Phila., Pa. 19103	Project 02G-C5257-01				Page B-2	
	By RA	Date APRIL '82	Ch'k'd MLD	Date 6/82	Rev.	Date

Title STEEL ROOFING — PRIMARY AUXILIARY BUILDING. YANKEE ROWE

THE ROOF PLAN BETWEEN COL. LINES 3-5 AND COL. LINES 6-8, CONSISTS OF STEEL DECK SUPPORTED BY STEEL BEAMS. THE STEEL DECKING IS 2½" THICK. WE ASSUME STEEL DECK WILL REMAIN PARTIALLY INTACT TO THE BEAM AND TRANSFER LOADS.

BEAMS

12 WF 27

SUPPORTS AREA OF 17.5' X 8.625'

UNBRACED LENGTH FOR UPLIFT CASE IS 17.5'

FOR $F_y = 36$ ksi FROM AISC MANUAL BEAM ALLOWABLE MOMENT GRAPHS.

MOMENT M = 35.5 k-ft.

∴ ALLOWABLE LOAD/UNIT LENGHT

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

$$\therefore w = \frac{8 \times 35.5}{(17.5)^2}$$

$= 0.9273469$ kips/ft

$= 927.3469$ lbs/ft

THIS SUPPORT LATERAL LENTH = 8.625 ft

∴ PRESSURE $p = 107.52$ psf

Increase steel allowable by 1.6 for Tornado

∴ Allowable pressure = 172.032 psf

FORM CS-FRC-81

10 WF 21

SUPPORTS AREA OF 15' X 7.5'

UNBRACED LENGHT FOR UPLIFT CASE IS 15'

FOR $F_y = 36$ ksi FROM AISC MANUAL, BEAM ALLOWABLE MOMENT GRAPHS, WE GET

ALLOWABLE MOMENT = 24.5 k-ft.

∴ ALLOWABLE LOAD/ UNIT LENGHT $w = \frac{8M}{l^2}$

$$\therefore w = \frac{8 \times 24.5}{(15)^2}$$

$$= 0.87111 \text{ k/ft}$$

$$= 871.111 \text{ lbs/ft}$$

THE LATERAL SUPPORT LENGHT HERE IS 7.5'

∴ PRESSURE $p = 116.1482$ psf.

Increase steel allowable stress by 1.6 for Tornado.

∴ Allowable pressure = 1.6×116.1482

$= 185.837$ psf

$W_{16 \times 36}$ $S_{xx} = 56.5^{IN^3}$, $R_T = 1.91^{IN}$, $d/A_F = 5.30\ IN^{-1}$, $l = 21.75'$

1.5-6b $F_B = \frac{170 \times 10^3}{(21.75 \times 12/1.91)^2} = 9.19^{KSI}$

1.5-7 $F_B = \frac{12 \times 10^3}{(21.75 \times 12)(5.30)} = 8.69^{KSI}$

∴ ALLOWABLE MOMENT = $8.68 \times 56.5 = 490.4$ K·IN
= 40.9 K·FT

AND WITH LOADS AT 1/3 POINTS $R = 5.40^K$
PLUS $1/8\,(.040)(21.75)^2 = 2.4$ K·FT

SUPPORT AREA = $7.59 \times 15 = 113.79'$

∴ ALLOW $P = 47.5^{PSF} + 10^{PSF}$ D.W.

Franklin Research Center A Division of The Franklin Institute The Benjamin Franklin Parkway, Phila., Pa. 19103	Project 02G-C5257-01				Page B-4	
	By RA	Date JUNE '82	Ch'k'd MLP	Date 6/82	Rev.	Date

Title STEEL ROOFING — PRIMARY AUXILIARY BUILDING

Calculation of allowable wind speeds for roof beams.

Roof beam 12WF27

Allowable pressure with stress increase = 172.032 psf.

a) ∴ Allowable Differential pressure = 1.195 psi

Differential pressure equivalent wind speed = $\sqrt{\frac{172.032}{0.00511}}$

$= 183.48$ mph

b) Tornado dynamic pressure = $\frac{172.032}{0.7}$ psf.
(Use 0.7 shape factor)

= 245.76 psf

Equivalent wind speed = $\sqrt{\frac{245.76}{0.00256}}$

= 309.84 mph.

Roof Beam 10WF21

Allowable pressure with stress increase = 185.837 psf.

a) ∴ Allowable Differential pressure = 1.291 psi

Differential pressure equivalent wind speed = $\sqrt{\frac{185.837}{0.00511}}$

= 190.70 mph.

b) Tornado dynamic pressure = $\frac{185.837}{0.7}$
(Use 0.7 shape factor)

= 265.4816 psf

Equivalent wind speed = $\sqrt{\frac{265.4816}{0.00256}}$

= 322 mph

FORM CS-FRC-81

Column 8 WF 31

Unbraced length = 1056.5 − 1039.5 = 17 ft.

Section properties:

Area = 9.12 in^2
d = 8 in.
I_{xx} = 110 in^4 r_{xx} = 3.47 in.
I_{yy} = 37 in^4 r_{yy} = 1.61 in.
r_T = 2.21 in

$$\text{minimum } \frac{kl}{r} = \frac{(17 \times 12)}{1.61}$$

$$= 126.7 > C_c \quad \text{for } F_y = 36 \text{ ksi}$$

$$\frac{l}{r_T} = \frac{(17 \times 12)}{2.21} = 92.3 < 119\sqrt{C_b} \quad \text{WHEN } C_b = 1.0$$

USE $F_b = \dfrac{12 \times 10^3 C_b}{ld/A_f}$ AS COMP. FLANGE AREA = TENSION FLANGE AREA.

$\therefore F_b = 25.4647 \text{ ksi} > 0.6F_y$

$\therefore$ Use $F_b = 0.6 \times 36$
$= 21.6$ ksi

Axial load on column 3E$_b$ =

wt. of steel deck + wt. of steel beams + live load

Wt. of deck = (16 × 8.625) × 4.77 = 658.26 lbs.
Self wt. of beam = 20 × 2 + (30/2) = 69 lbs.
Live load = (16 × 8.625) × 40 = 5520 lbs.

TOTAL = 6247.26 lbs = 6.25 KIPS

$$f_a = \frac{6.25}{9.12} = .685$$

$$F_a = \frac{12\pi^2 E}{23\left(\frac{Kl}{r}\right)^2} = \frac{12\pi^2(29\times10^3)}{23\,(126.7)^2} = 9.3 \text{ ksi}$$

$$\frac{f_a}{F_a} = \frac{.685}{9.3} = .074$$

$$\frac{f_a}{F_a} + \frac{f_b}{F_b} = 1.0$$

$$\frac{f_b}{F_b} = 1.0 - \frac{f_a}{F_a} = 1.0 - .074 = .926$$

$$f_b = F_b(.926) = (21.6)(.926) = 20.0016 \text{ ksi}$$

$$f_b = \frac{M}{I}\cdot y \qquad y = d/2$$

$$M = \frac{f_b \cdot I}{d/2} = \frac{(20.0016)(110)}{(8/2)} = 550.044 \text{ k-in.}$$
$$= 45.837 \text{ k-ft.}$$

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

$$w = \frac{8M}{l^2} = \frac{8\times45.837}{17^2} = 1.2688 \text{ k/ft}$$
$$= 1268.84 \text{ lbs/ft}$$

pressure $\frac{1268.8}{16} = 79.303$ psf.

Increase steel allowables by 1.6 for Tornado

∴ Allowable pressure = $1.6\times79.303 = 126.885$ psf

Calculation of corresponding wind speeds for Steel columns with live loads

Column (3E3) is 8WF31

WITH LIVE LOADS

Allowable pressure = 126.885 psf

a) ∴ Allowable differential pressure = 0.881 psi

For this differential pressure corresponding

$$\text{wind speed} = \sqrt{\frac{126.885}{0.00511}} = \underline{157.58\ \text{mph}}$$

b) ∴ Tornado dynamic pressure allowed $= \frac{126.885}{0.8}$
(Use 0.8 shape factor)
$= 158.606$ psf

For this tornado dynamic pressure corresponding

$$\text{wind speed} = \sqrt{\frac{158.606}{0.00256}}$$

$$= \underline{248.91\ \text{mph}}$$

For Column 8WF31 with no live load

approximate axial load = 1.5 kips

$f_a = \frac{1.5}{9.12} = 0.1645\,ksi \Rightarrow \therefore \frac{f_a}{F_a} = \frac{0.1645}{9.3} = 0.01768$

$F_b = 21.6\,ksi$

Use $\frac{f_a}{F_a} + \frac{f_b}{F_b} \leq 1.0 \Rightarrow f_b = \left(1 - \frac{f_a}{F_a}\right) F_b$

$= 21.22\,ksi$

$f_b = \frac{M}{I} \cdot y \Rightarrow M = \frac{21.22 \times 110}{8/2} = 583.495\,k\text{-}in$

$\therefore M = 48.625\,k\text{-}ft.$

Use $M = \frac{wl^2}{8} \Rightarrow$ actual allowable load $w = \frac{8 \times 48.625}{17^2} \times 1000$

$= 1346.009\,lbs/ft.$

For a lateral width of 16 ft. pressure $= \frac{1346.009}{16} = 84.1255\,psf$

Increase steel allowables by 1.6 for tornado

$\therefore$ allowable pressure $= 1.6 \times 84.1255$

$= 134.6\,psf.$

WITH NO LIVE LOADS

a) $\therefore$ allowable differential pressure = 0.935 psi

For this differential press. corresponding wind speed $= \sqrt{\frac{134.6}{0.00511}} = \underline{162.3\,mph}$

b) $\therefore$ allowable tornado dynamic pressure $= \frac{134.6}{0.8} = 168.251\,psf.$

(Use 0.8 shape factor)

$\Rightarrow$ corresponding wind speed $= \sqrt{\frac{168.251}{0.00256}} = \underline{256.4\,mph}$

CONCRETE BLOCK WALL: CAPACITY IN VERTICAL DIRECTION

16'8" high 8" Concrete Block Wall

Wt. of 8" thick Concrete Block = 55 psf.

For 1 ft. width, compressive stress = $\frac{16.67 \times 55}{8 \times 12}$

$= 9.55$ psi

Area Inertia $= \frac{1}{12} bh^3 = \frac{1}{12}(12)(8)^3 = 512 \text{ in}^4$

Let W = WIND LOAD PER UNIT LENGTH

Moment $= \frac{1}{8} w l^2$

$M = \frac{1}{8} w (16.67)^2$

$M = 34.72\, w \times 12 = 416.7\, w$

Assume the masonry block is Grouted.
Allowable Tensile stress normal to Bed Jt. = 27 psi

$$27 = \frac{416.7\,(w)\,(8/2)}{512} - \frac{9.55}{2}$$

$31.77 = 3.26\, w$

$\Rightarrow w = 9.76$ psf

Increase allowables by 1.3 for Tornado conditions in block walls.

Use a shapefactor for 0.8

$27 \times 1.3 = 3.26W - 4.78 \Rightarrow W = 12.2$ PSF

Project 02G-C5257-01				Page B-10	
By RA	Date JUNE '82	Ch'k'd MLP	Date 6/82	Rev.	Date

Title PRIMARY AUXILIARY BUILDING – CONCRETE BLOCK WALL

Calculation of corresponding wind speeds for allowable pressure load on concrete block walls

Allowable pressure = 12.2 PSF

a) ∴ Allowable differential pressure = 0.085 psi

For this differential pressure corresponding

$$\text{wind speed} = \sqrt{\frac{12.2}{0.00511}} = \underline{48.9 \text{ mph}}$$

b) ∴ Allowable tornado dynamic pressure = $\frac{12.2}{0.8}$

= 15.3 psf.

For this tornado dynamic pressure corresponding

$$\text{wind speed} = \sqrt{\frac{15.3}{0.00256}} = \underline{77.3 \text{ mph}}.$$

c) ∴ Allowable high wind dynamic pressure = $\frac{9.76}{0.8}$ = 12.2 psf

⇒ corresponding wind speed = $\underline{55.5 \text{ mph}}$

APPENDIX C

DIESEL GENERATOR BUILDING DESIGN REVIEW CALCULATIONS

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Project 02G - C5257-01				Page C-1	
By RA	Date JUNE '82	Ch'k'd	Date	Rev.	Date

Title DIESEL GENERATOR BUILDING.

THE DIESEL GENERATOR BUILDING IS BASICALLY A STRUCTURAL STEEL STRUCTURE WITH CONCRETE BLOCK WALLS. IN THIS ANALYSIS ROOF DECKING, ROOF BEAM, STEEL COLUMNS AND CONCRETE BLOCK WALLS HAVE BEEN EXAMINED.

WE ASSUME THAT CONCRETE BLOCK WALL TRANSFERS ALL THE LOAD IN THE LATERAL DIRECTION ONTO STEEL COLUMNS BEFORE IT FAILS. NO DETAILS & PROPERTIES ARE AVAILABLE FOR CONC BLOCK WALL & ROOF DECKING. ASSUMED SECTION PROPERTIES HAVE BEEN USED IN THE ANALYSIS FOR CONC. BLOCK WALL AND STEEL ROOF DECKING ALSO.

ROOF STEEL BEAM & STEEL DECKING HAVE BEEN ANALYZED FOR UPLIFT PRESSURE CASE. EVEN AFTER FAILURE, DECKING ASSUMED TO BE PARTIALLY INTACT TO THE BEAM AND TRANSFERS LOADS.

18 W 50

$S_{xx} = 89.1\ in^3$ $\quad \ell = 32'$ SINCE COMPRESSION FLANGE IS NOW THE BOTTOM FLANGE!

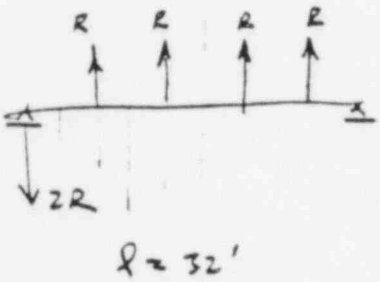
$R_T = 1.96\ in$

$d/A_F = 4.21$

$\ell/R_T = 32 \times 12/1.96 = 196 > 119$

$F_B = \dfrac{12 \times 10^3}{32 \times 12 \times 4.21} = 7.42\ ksi \Rightarrow M = (7.42)(89.1) = 661\ k\text{-}in\ /\ 55.1\ k\text{-}ft$

$F_B = \dfrac{170 \times 10^3}{(196)^2} = 4.43\ ksi$



$MOM = R\left(\frac{1}{10} + \frac{3}{10} - 2 \times \frac{5}{10}\right)\ell$

$= -\frac{6}{10} R\ell \Rightarrow R = 2.87\ kips$

SUPPORT AREA $= \frac{1}{2}(13.5' + 21.67')(6.58') = 115.7\ ft^2$

PRESS $= \dfrac{2.87}{115.7} = 24.8\ psf$

EST D.W. $= 5\ psf + \dfrac{50 \times 1}{17.6'} = 7.84\ psf$

TORNADO $P_A = 24.8 \times 1.6 + 7.84 = 47.5\ psf$

DIFFERENTIAL PRESSURE	.330 PSI	96 MPH
DYNAMIC PRESSURE	67.9 PSF	163 mph
HIGH WIND	46.6 PSF	133 mph

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Project	Page		
02G-C5257-01	C-3		
By RA	Date APRIL '82	Ch'k'd Date	Rev. Date

Title: ROOF STEEL – DIESEL GEN. BLDG.

10 WF 21

supports area of 21'8" × 6'7"

unbraced length = 11'6"

Allowable Moment = 30.5 k·ft (from Beam Diag. of AISC)

$l = 11.5'$ $\quad M = \frac{Wl^2}{8}$

$w = \frac{8M}{l^2} = \frac{8(30.5)}{(21.67)^2} \times 1000 = 519.7633$ psf

pressure $= \frac{w}{6'7"} = 78.9514$ psf.

Increase steel allowables by 1.6 for Tornado ∴ allowable pr = 1.6 × 78.9514 = 126.322 psf

a) Allowable diff pressure = 0.88 psi

⇒ corresponding wind speed = 157.23 mph

b) Allowable tornado dy. pr = $\frac{126.322}{0.7}$ = 180.46 psf ⇒ wind speed = 265.504 mph

8 WF 17

$l = 13'-6"$ lateral length = 6'7"

M = 16.5 k-ft Allowable Moment

$w = \frac{8M}{l^2} = \frac{8(16.5)}{(13.5)^2} = 0.724$ k/ft = 724 lbs/ft

Convert this load to pressure $= \frac{724}{6.583} = 110$ psf

Increase steel allowables by 1.6 for Tornado

∴ Allowable pressure = 1.6 × 110 = 176 psf.

a) ∴ Allowable differential pressure = 1.222 psi ⇒ Corresponding wind speed = 185.56 mph

b) ∴ Allowable Tornado dynamic press. = $\frac{176}{0.7}$ = 251.43 psf ⇒ corresponding wind speed ≅ 313.39 mph
(Use 0.7 shape factor)

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COLUMNS

8 WF 17

[ASSUME ALL LOADS ARE TRANSFERRED TO COLUMNS BY BLOCK WALLS IN LATERAL DIRECTION, BEFORE BLOCK WALLS FAIL]

Unbraced length = 1037'2" - 1022'11" = 14'3"

Section properties:

Area = 5.01 in²
$d = 7.93$ in.
$b_f = 5.25$
$I_{xx} = 56.6 \text{ in}^4$ $\quad r_{xx} = 3.36$ in.
$I_{yy} = 7.44 \text{ in}^4$ $\quad r_{yy} = 1.22$ in.
$r_T = 1.4$ in.

$$\text{minimum } \frac{Kl}{r} = \frac{(14.25')(12)}{1.22} = 140.16 > 126.1 = C_c$$

$$\frac{l}{r_t} = \frac{(14.25)(12)}{1.4} = 122.14$$

$$\therefore F_a = \frac{12\pi^2 E}{23\left(\frac{Kl}{r}\right)^2} = \frac{12\pi^2(29\times10^3)}{23(140.16)^2} = 7.61 \text{ ksi}$$

take $C_b = 1$

$$\therefore F_b = \frac{(170\times10^3)C_b}{(l/r_T)^2} = \frac{170\times10^3}{(122.14)^2}$$

$F_b = 11.39$ ksi

$$\text{But } F_b = \frac{12\times10^3 C_b}{l\,d/A_f} = \frac{12\times10^3}{(14.25\times12)\times4.95} = 14.177 \text{ ksi}$$

USE $F_b = 14.177$ ksi

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Project 02G-C5257-01				Page C-5	
By RA	Date APRIL '82	Ch'k'd	Date	Rev.	Date

Title STEEL COLUMN — DIESEL GEN. BLDG.

Now consider Column 105 W

Dead weight of 16 ft of 18WF50(2) = 16 x 50 x 2 = 1600
11'6" of 10 WF 25 = 11.5 x 25 = 287.5
11'6" of 10WF21(2) = 11.5 x 21 x 2 = 483
16 ft. of 8 B 10 = 16 x 10 = 160
13'6" of 18 WF45 = 13.5 x 45 = 607.5
13'6" of 8 WF 17 (2) = 13.5 x 17 x 2 = 459

TOTAL → 3597.0 lbs
⇒ 3.597 KIPS

Live load: 16 x 25 x 40 psf = 16000 = 16 KIPS

TOTAL load = 16 + 3.597 = 19.597 KIPS

$$f_a = \frac{19.597}{5.01} = 3.91$$

$$\frac{f_a}{F_a} = \frac{3.91}{7.61} = .514$$

$$F'_{e_x} = \frac{12\pi^2 E}{23\left(\frac{14.25\times12}{3.36}\right)^2} = 57.65$$

$$\frac{f_b}{\left(1-\frac{f_a}{F'_{ex}}\right)F_b} = 1-\frac{f_a}{F_a}$$

$$f_b = (1-.514)\left[\left(1-\frac{3.91}{57.65}\right)14.177\right]$$

$$f_b = 6.423 \text{ ksi}$$

WITHOUT LIVE LOAD
TAKE AXIAL LOAD = 5 kips

$$\therefore f_a = \frac{5}{5.01} = 1 \text{ ksi}$$

$$\frac{f_a}{F_a} = \frac{1.0}{7.61} = 0.1311 < 0.15$$

$$\therefore \text{Use } \frac{f_a}{F_a} + \frac{f_b}{F_b} \le 1.0$$

$$\therefore f_b = \left(1-\frac{f_a}{F_a}\right)F_b$$

$$= (1-0.1311)\,14.177$$

$$\therefore f_b = 12.32 \text{ ksi}$$

Title STEEL COLUMN - DIESEL GEN. BLDG.

$$\frac{M}{I} \cdot y = f_b \implies M = \frac{f_b I}{y} \qquad y = d/2$$

$$= \frac{(6.423)(56.6)}{(7.93/2)} \qquad = \frac{12.32(56.6)}{(7.93/2)}$$

$$= 91.683 \text{ k-inch} \qquad = 175.8351 \text{ k-inch}$$

$$\therefore M = 7.6402 \text{ k-ft} \qquad = 14.653 \text{ k-ft}$$

Now, convert allowable moment to pressure

$$M = \frac{wl^2}{8}$$

$$\implies w = \frac{8 \times 7.6402 \times 1000}{(14.25)^2} = 301.0 \text{ lbs/ft}$$

For a lateral width of 25 ft. pressure $= \frac{301}{25} = 12.04$ psf

Increase steel allowable by 1.6 for tornado.

$\therefore$ allowable pressure $= 1.6 \times 12.04 = 19.264$ psf

WITH LIVE LOAD

a) $\therefore$ Allowable diff. pressure = 0.134 psi

For this diff. pr., equivalent wind speed $= \sqrt{\frac{19.264}{0.00511}} = \underline{61.4 \text{ mph}}$

b) $\therefore$ Allowable Tornado dynamic pressure $= \frac{19.264}{0.8} = 24.1$ psf.
(Use 0.8 shape factor)

corresponding wind speed $= \sqrt{\frac{24.1}{0.00256}} = \underline{96.985 \text{ mph}}$

c) High wind dynamic pressure $= \frac{12.04}{0.8} = 15.05$ psf.

$\therefore$ Corresponding wind speed = 80.1 mph

Franklin Research Center A Division of The Franklin Institute The Benjamin Franklin Parkway, Phila., Pa. 19103	Project 02G- C5257-01		Page C-7
	By RA Date JUNE 82	Ch'k'd Date	Rev. Date

Title STEEL COLUMNS — DIESEL GEN. BLDG.

WITH NO LIVE LOAD CASE

$M = 14.653$ k-ft

Now, convert allowable moment to pressure

$$M = \frac{wl^2}{8}$$

$$\Rightarrow w = \frac{8 \times 14.652923}{(14.25)^2} \times 1000$$

$w = 577.277$ lbs/ft.

For a lateral width of 25ft. pressure $= \frac{577.277}{25}$

$= 23.091$ psf.

Increase steel allowable by 1.6 for tornado

$\therefore$ allowable pressure $= 1.6 \times 23.091$

$= 36.946$ psf.

WITH NO LIVE LOAD

a) $\therefore$ Allowable diff pressure $= 0.26$ psi

$\Rightarrow$ Corresponding wind speed $= \sqrt{\frac{36.946}{0.00511}}$

$= \underline{85.03 \text{ mph}}$.

b) $\therefore$ Allowable Tornado dynamic pressure $= \frac{36.946}{0.8} = 46.182$ psf

$\Rightarrow$ corresponding wind speed $= \sqrt{\frac{46.182}{0.00256}} = \underline{134.31 \text{ mph}}$

c) High wind dynamic pressure $= \frac{23.091}{0.8} = 28.864$ psf

$\rightarrow$ Corresponding wind speed $= 104.77$ mph.

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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 C-8	
	By RA	Date APRIL '82	Ch'k'd	Date	Rev.	Date

Title: CONCRETE BLOCK WALL — DIESEL GENERATOR BUILDING.

DIESEL GENERATOR BUILDING

8" thick block wall

Along column line Ⓧ, Btw. elev. 1022'-10" & 1037'6"

∴ height = 14'-8"

NORTH SIDE WALL. ASSUME HOLLOW BLOCK WALL

for $l = 14'\text{-}8''$

Wt. of 8" thick concrete block wall = 55 psf.

For 1 ft. width, compressive stress

$$= \frac{(14.67)(55)}{(8)(12)} = 8.4 \text{ psi}$$

$I = 512 \text{ in}^4$

$$M = \frac{1}{8} w l^2$$

$$= \frac{1}{8} w (14.67)^2 \times 12$$

$$= 322.67\, w \text{ k-inch}$$

Allowable Tensile stress normal to Bed Jt. = 27 psi

$$27 = \frac{322.67\, w\, (8/2)}{512} - \frac{8.4}{2}$$

$$31.2 = 2.52\, w$$

$\Rightarrow w = 12.38$ psf is the pressure

$(1.3)(27) = 2.52W - 4.2 \Rightarrow W = 15.6$ PSF

FORM CS-FRC-81

Increase allowable by 1.3 for Tornado.

$\therefore$ allowable pressure = 15.6 PSF

Calculation of wind speeds for this pressure loading.

a) $\therefore$ Allowable diff. pressure = 0.108 psi

$\Rightarrow$ Corresponding wind speed = $\sqrt{\dfrac{15.6}{0.00511}}$ = 55.3 mph

b) Allowable tornado dynamic pr. = $\dfrac{15.6}{0.8}$
(Use 0.8 shape factor)

= 19.5 psf

$\rightarrow$ corresponding wind speed = $\sqrt{\dfrac{19.5}{0.00256}}$ = 87.3 mph

c) $\therefore$ Allowable high wind dynamic pr = $\dfrac{12.38}{0.8}$

= 15.475 psf

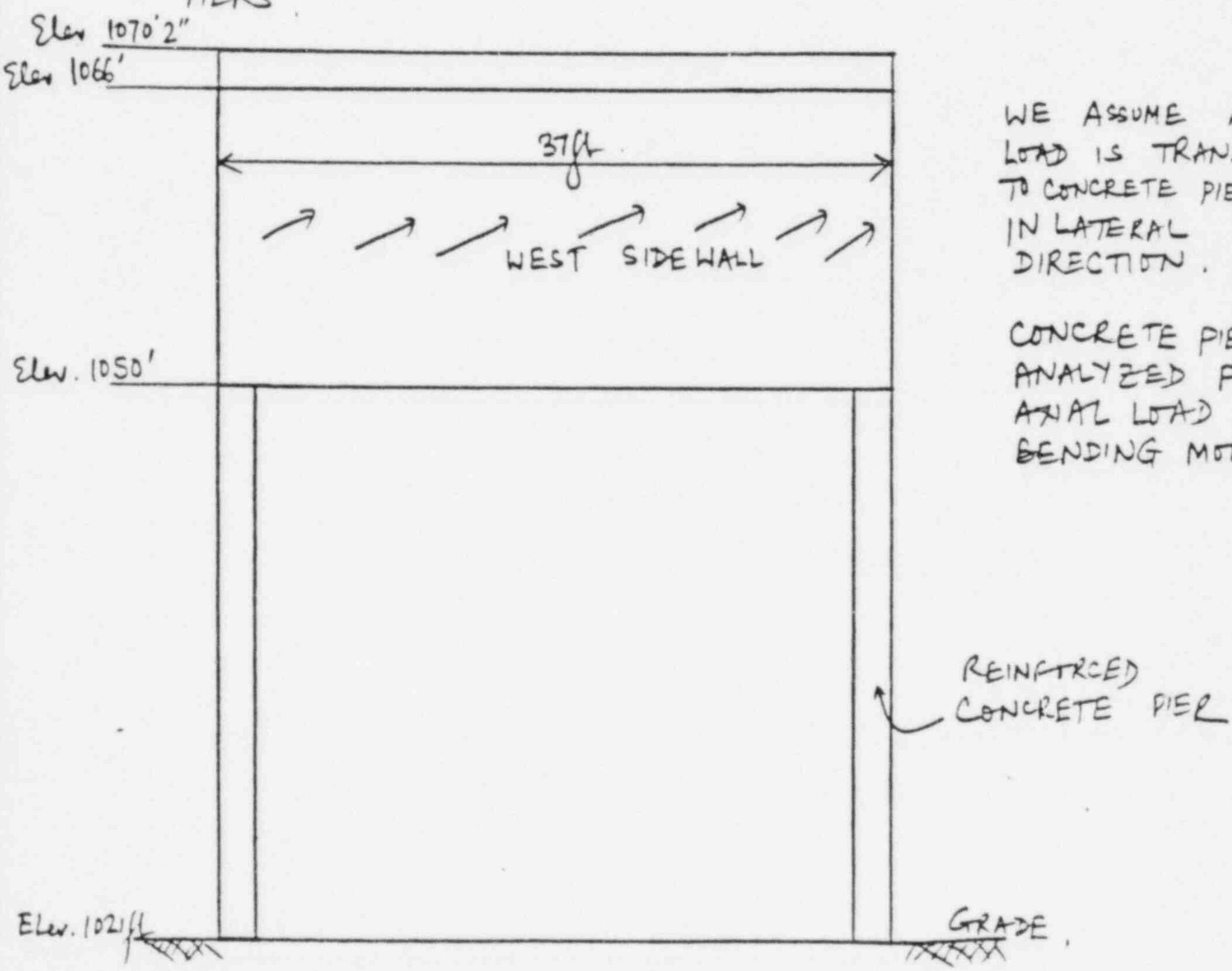
$\Rightarrow$ Corresponding wind speed = 63 mph

APPENDIX D

CONTROL ROOM DESIGN REVIEW CALCULATIONS

Title CONTROL ROOM AT YANKEE ROWE.

THE CONTROL ROOM IS LOCATED ABOVE ELEVATION 1052 ft WITH THE TOP OF ROOF AT ELEVATION 1070 ft 2 in. THE SOUTH WALL IS 3FT THICK REINFORCED CONCRETE STRUCTURE ABOVE GRADE. EAST AND WEST SIDE WALLS ARE 3FT THICK SUPPORTED ON CONCRETE PIERS. WE ANALYZE WEST SIDEWALL & CONCRETE PIERS



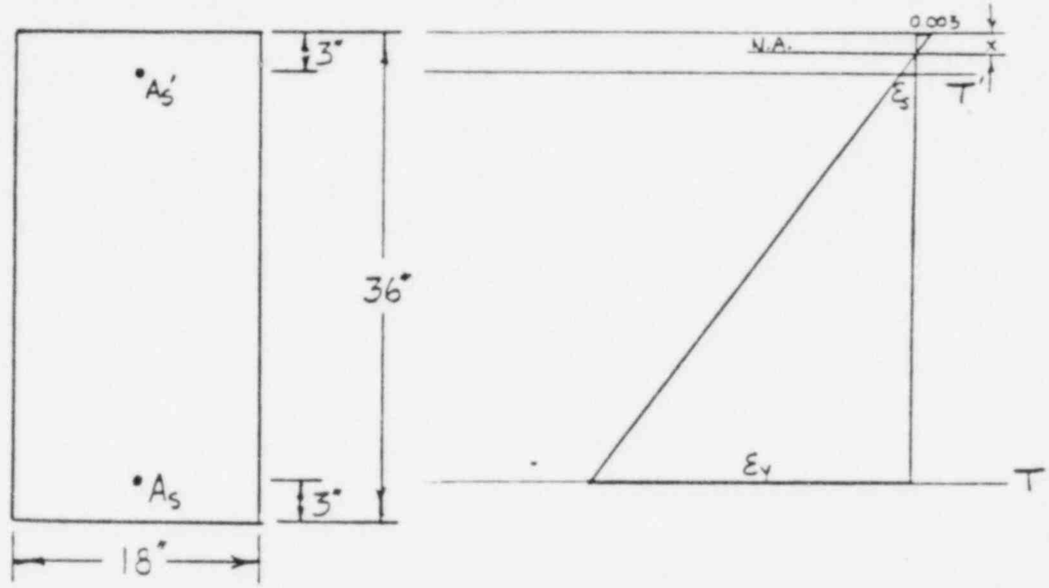
WE ASSUME ALL LOAD IS TRANSFERED TO CONCRETE PIERS IN LATERAL DIRECTION.

CONCRETE PIERS ANALYZED FOR AXIAL LOAD AND BENDING MOMENT.

CONTROL - ROOM

West Side Wall Capacity in lateral direction.

Consider a 18" wide section of the 3' thick wall.



Each steel = #8 bar $\qquad A_s = A_s' = 0.79\ in^2$

Concrete $\quad f_c' = 2.5$ ksi $\qquad \varepsilon_c = 0.003$

Steel $\quad f_y = 40$ ksi $\qquad \varepsilon_y = 1.38 \times 10^{-3}$

Compression:

$$C_c = 0.85 f_c' (0.85x) b$$
$$= (0.85)(2.5)(0.85x)(18)$$
$$= 32.51\, x$$

$$f_s = \frac{40\,(3-x)}{(33-x)}$$

Tension:

$$T' = A_s' f_s$$
$$= (0.79)\left[\frac{40\,(3-x)}{(33-x)}\right]$$

$$= \frac{31.6\,(3-x)}{(33-x)}$$

$$T = A_s f_y$$
$$= (0.79)(40)$$
$$= 31.6 \text{ KIPS}$$

For equilbrium $\quad C_c = T + T'$

$$32.51x = 31.6 + \frac{31.6\,(3-x)}{(33-x)}$$

$$32.51x\,(33-x) = 31.6\,(33-x) + 31.6\,(3-x)$$
$$1072.83x - 32.51x^2 = 31.6\,(36-2x)$$
$$1072.83x - 32.51x^2 = 1137.6 - 63.2x$$

$$32.51x^2 - 1136.03x + 1137.6 = 0$$

$$x = \frac{1136.03 \pm \sqrt{(1136.03)^2 - 4(32.51)(1137.6)}}{2(32.51)}$$

$$= \frac{1136.03 \pm 1068.94}{65.02}$$

$$= 1.032 \text{ in.}$$

$$\therefore\; C_c = 32.51x$$
$$= 32.51(1.032)$$
$$= 33.55 \text{ KIPS}$$

$$T' = \frac{31.6\,(3-x)}{(33-x}$$
$$= \frac{31.6\,(3-1.032)}{(33-1.032)}$$
$$= 1.945 \text{ KIPS}$$

$a = 0.85x$
$= .877$

$$M = T\,(33 - a/2) + T'(3 - a/2)$$
$$= 31.6\,(33-.439) + 1.945\,(3-.439)$$
$$M = 1033.9 \text{ k-inch}$$

$$\therefore\; M_{allow} = 86.16 \text{ k-ft.}$$

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	By RA	Date MAY '82	Ch'k'd MLP	Date 6/82	Rev.	Date

Title: CONTROL ROOM — WALL SECTION

Calculation of actual moment for 3 ft thick wall in lateral direction.

[300 mph
2.25 psi $\simeq$ 324 psf

$$W = 324 \times 1.5 = 486 \text{ psf}$$

$$L = 38.5 - 1.5 = 37 \text{ ft.}$$

$$M = \frac{WL^2}{8}$$

$$= 486 \frac{(37.0)^2}{8}$$

$$\therefore M_{actual} = 83.167 \text{ k-ft.}$$

$$\therefore M_{actual} < M_{allow}$$

FORM CS-FRC-81

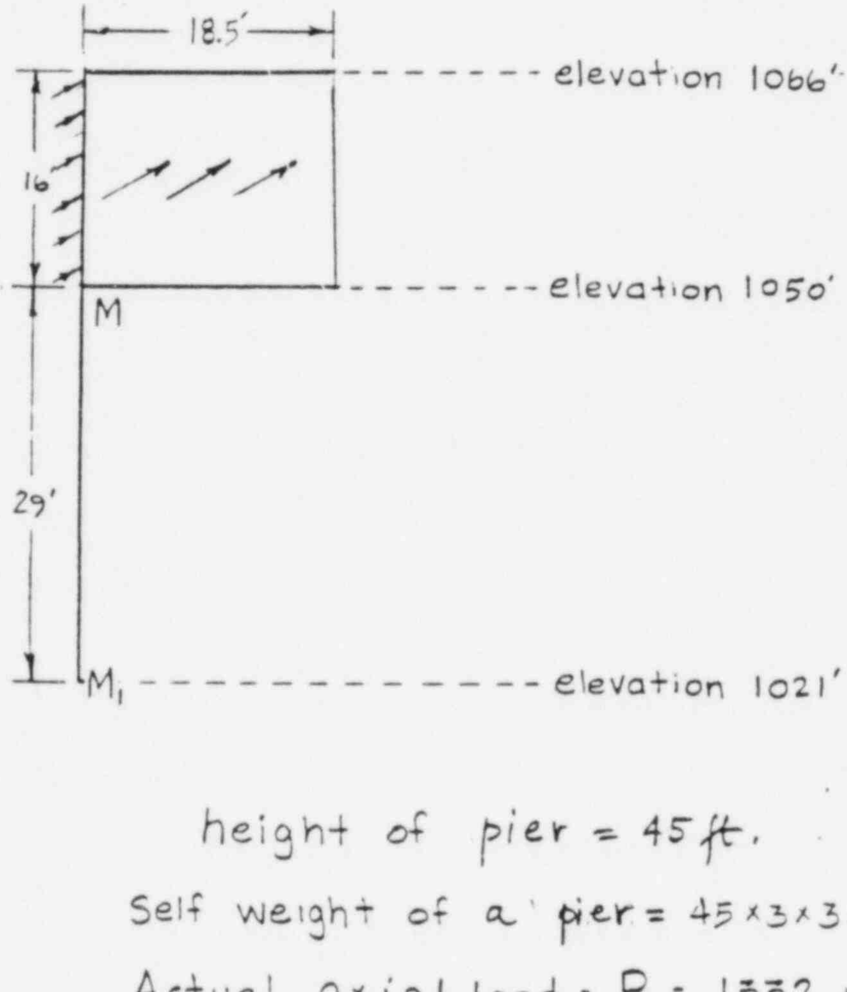
Title: CONTROL ROOM - SUPPORTING CONCRETE PIER SECTION

Concrete Piers supporting the Control Room on west side

wt. of wall slab = 37' × 16' × 3' (0.15) = 266.4 KIPS

half side wt. on one pier = 133.2 KIPS

h = 1066' − 1050' = 16'

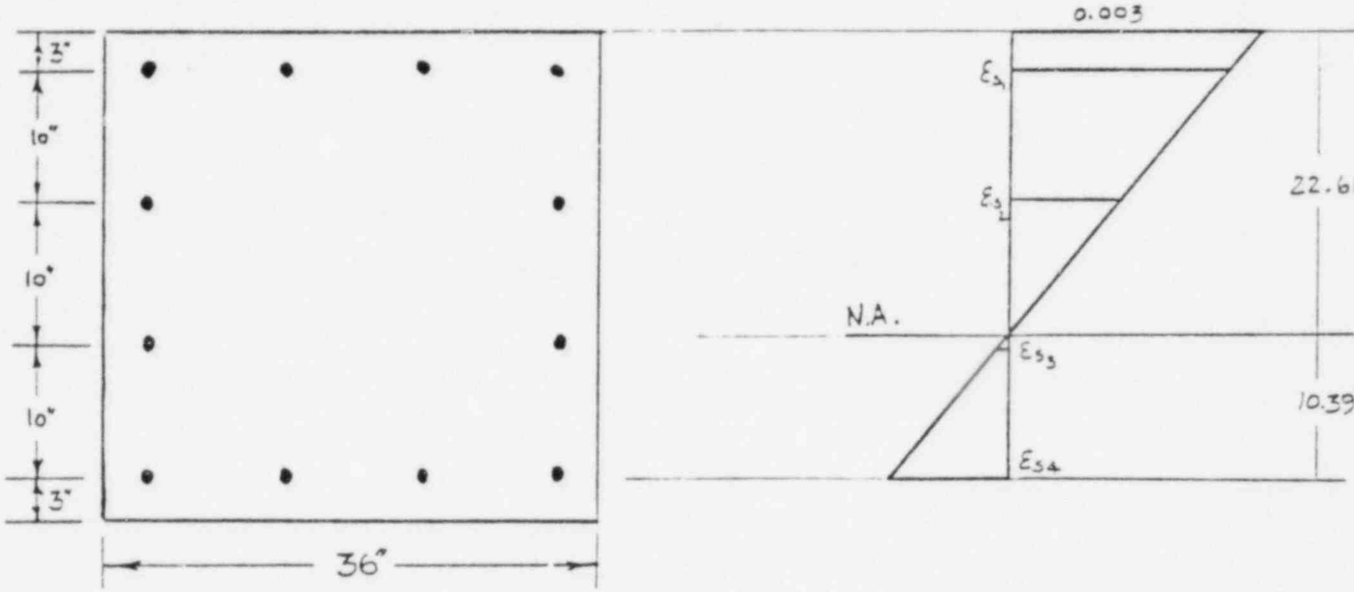


height of pier = 45 ft.

Self weight of a pier = 45 × 3 × 3 (.15) = 60.75 KIPS

Actual axial load = P = 133.2 + 60.75
= 193.95 KIPS

Title CONTROL ROOM — SUPPORTING CONCRETE PIER SECTION



All bars are #9

$$X_b = \frac{0.003 \times 33}{0.003 + 1.38 \times 10^{-3}}$$

$$X_b = 22.61''$$

$\varepsilon_{s_4} = \varepsilon_y \quad \therefore \quad f_{s_4} = f_y = 40 \text{ ksi}$

$\varepsilon_{s_3} = \dfrac{\varepsilon_y (0.39)}{10.39} \quad \therefore \quad f_{s_3} = \dfrac{40 (0.39)}{10.39} = 1.5 \text{ ksi}$

$\varepsilon_{s_2} = \dfrac{\varepsilon_y (9.61)}{10.39} \quad \therefore \quad f_{s_2} = \dfrac{40 (9.61)}{10.39} = 37.0 \text{ ksi}$

$\varepsilon_{s_1} = \varepsilon_y \quad \therefore \quad f_{s_1} = 40 \text{ ksi}$

Compression: $C_c = 0.85 f'_c (0.85 x_b) b$

$= 0.85(2.5)(0.85)(22.61)(36)$

$\therefore C_c = 1470.21$ KIPS

$C_s = (4 \text{ bars } \#9)(40 - 0.85(2.5)) + (2 \text{ bars } \#9)(37.0 - 0.85(2.5))$

$= (4)(1.0)(37.875) + (2)(1.0)(34.875)$

$\therefore C_s = 221.25$ KIPS

Tension: $T_4 = (4 \text{ bars } \#9) f_y$

$= 4(40)$

$T_4 = 160$ kips

$T_3 = (2 \text{ bars } \#9) f_{s3}$

$= 2(1.5)$

$T_3 = 3.$ kips

$T_{TOT} = 163.0$ KIPS

$P_b = C_c + C_s - T_{TOT}$

$= 1470.21 + 221.25 - 163.0$

$P_b = 1528.46$ kips

For symmetric section, we find plastic centroid at mid-depth of the section.

$M_b = C_c(18 - a/2) + C_{s_1}(15) + C_{s_2}(5) + T_3(5) + T_4(15)$

$a = .85(22.61) = 19.22$

$= 1470.21(18 - 9.61) + 151.5(15) + 69.75(5) + 3(5) + 160(15)$

$= 12335.06 + 2272.5 + 348.75 + 15 + 2400$

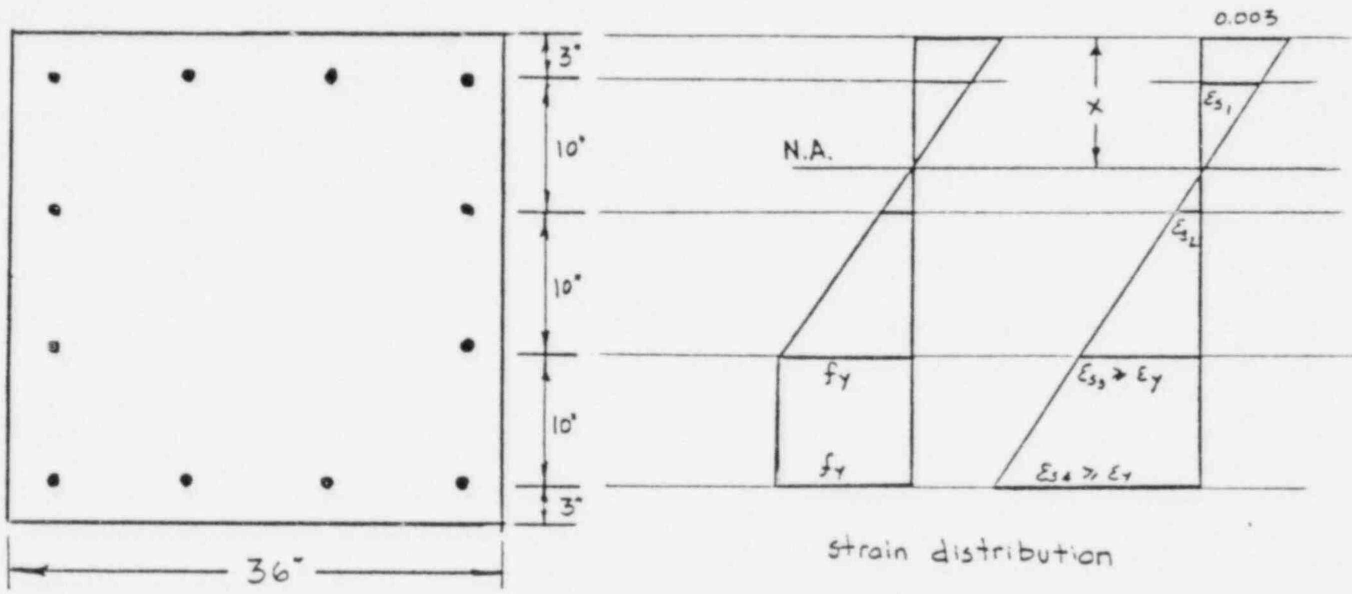
$\therefore M_b = 17371.31$ k·inch

$\therefore M_b = 1447.61$ k·ft.

Title: CONTROL ROOM – SUPPORTING CONCRETE PIER SECTION

But for our case use $P = 225$ kips

$$C_c = 0.85 f_c' (0.85x)(36)$$
$$= 0.85(2.5)(0.85x)(36)$$
$$C_c = 65.025\,x \text{ kips}$$



strain distribution

$$\varepsilon_{s_1} = \frac{0.003}{x}(x-3) \qquad f_{s_1} = 29{,}000\,\varepsilon_{s_1} = \frac{87}{x}(x-3)$$

$$\varepsilon_{s_2} = \frac{0.003}{x}(13-x) \qquad f_{s_2} = 29{,}000\,\varepsilon_{s_2} = \frac{87}{x}(13-x)$$

Compression in steel $\quad C_s = (4 \times 1.0)\left(\frac{87}{x}(x-3) - 0.85(2.5)\right)$

$$= \frac{(4)(87)}{x}(x-3) - 4(0.85)(2.5)$$

$$C_s = \frac{348(x-3)}{x} - 8.5$$

Tension $\quad T_2 = (2 \times 1.0)\,\frac{87}{x}(13-x)$

$$T_2 = \frac{174}{x}(13-x)$$

$$T_3 = 2 \times 40 = 80 \text{ KIPS}$$

$$T_4 = 4 \times 40 = 160 \text{ KIPS}$$

$$T_{TOT} = \frac{174}{x}(13-x) + 240$$

Now for equilibrium

$$P = C_c + C_s - T$$

$$225 = 65.025x + \frac{348}{x}(x-3) - 8.5 - \frac{174}{x}(13-x) - 240$$

$$473.5x = 65.025x^2 + 348(x-3) - 174(13-x)$$

$$473.5x = 65.025x^2 + 348x - 1044 - 2262 + 174x$$

$$65.025x^2 + 48.5x - 3306 = 0$$

$$x = \frac{-48.5 \pm \sqrt{(48.5)^2 + 4(65.025)(3306)}}{2(65.025)}$$

$$x = 6.77''$$

$$\therefore C_c = 65.025x$$
$$= 65.025(6.77)$$
$$= 440.22 \text{ KIPS}$$

$$f_{s_1} = \frac{87}{x}(x-3)$$
$$= \frac{87(6.77-3)}{6.77}$$
$$= 48.45 > 40 \quad \therefore \text{ use } f_{s_1} = 40 \text{ ksi}$$

$$f_{s_2} = \frac{87(13-6.77)}{6.77}$$
$$= 80.06 > 40 \quad \therefore \text{ use } f_{s_2} = 40 \text{ ksi}$$

Title CONTROL ROOM – SUPPORTING CONCRETE PIER SECTION

Find N.A. again

$$225 = 65.025x + (4 \times 40) - [2(40) + 2(40) + 4(40)] - 8.5$$

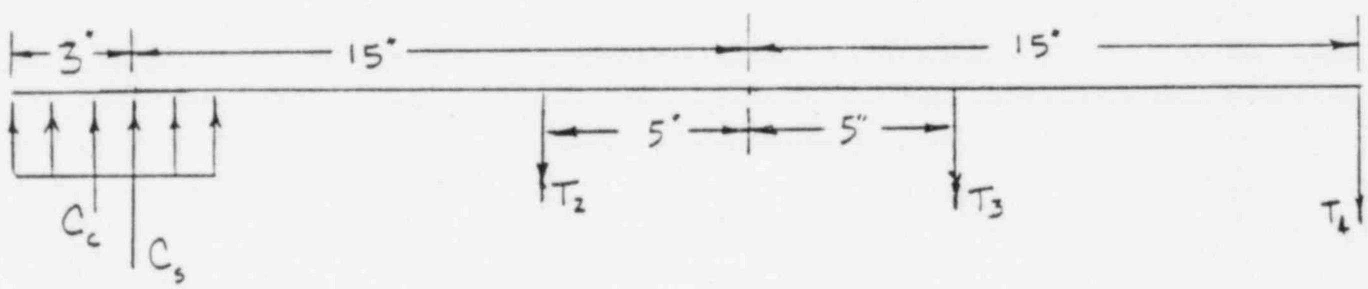
$$225 + 80 + 80 + 8.5 = 65.025$$

$$x = \frac{393.5}{65.025}$$

$$x = 6.05''$$

Find again $C_c = 65.025x$
$= 65.025(6.05)$
$= 393.4$ KIPS

Find Moment capacity



$$M = C_c(18 - a/2) + C_s(15) + T_3(5) + T_4(15) - T_2(5)$$
$= 393.4(18 - 2.57) + 151.5(15) + 80(5) + 160(15) - 80(5)$
$= 6070.16 + 2272.5 + 400 + 2400 - 400$
$= 10742.66$ k-inch

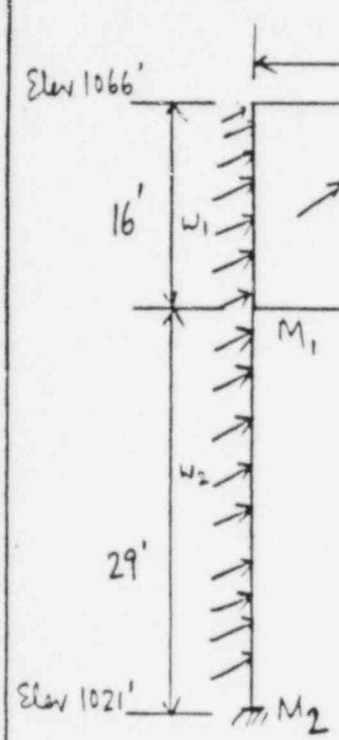
$a = .85x$
$= 5.14$

$\therefore M = 895.22$ k-ft

$\therefore M_{allow} = 895.22$ k-ft.

Title: CONTROL ROOM - SUPPORTING CONCRETE PIER SECTION.

CALCULATION OF EQUIVALENT PRESSURE FOR ALLOWABLE MOMENT CAPACITY OF THE CONCRETE PIER.



Suppose equivalent pressure = w psf

Let the moments be M_1 & M_2

$w_1 = 18.5w$ lbs/ft.

$w_2 = 3w$

$$M_1 = 18.5w(16)\left(\frac{16}{2}\right)$$

$$M_2 = 18.5w(16)(29+8) + 3w(29)\left(\frac{29}{2}\right)$$

We find $M_{ALLOW} = 895.22$ k-ft.

$$\therefore\ 895.22 = \frac{18.5w}{1000}(16 \times 37) + \frac{3w}{1000}(29)\left(\frac{29}{2}\right)$$

$$\Rightarrow\ w = \frac{895.22}{12.2135}$$

Allowable pressure for concrete pier = 73.298 psf

Calculating the the corresponding wind speeds for various loadings.

a) For differential pressure, allowable pressure = 0.51 psi

$$\Rightarrow \text{Corresponding wind speed} = \sqrt{\frac{73.298}{0.00511}} = \underline{120\text{ mph}}.$$

b) For Tornado dynamic pressure, allowable = $\dfrac{73.298}{0.8}$ (Use 0.8 shape factor)

$= 91.623$ psf

$$\Rightarrow \text{corresponding wind speed} = \sqrt{\frac{91.623}{0.00256}} = \underline{189.2\text{ mph}}$$