SEP 0 2 1982

Docket No. 50-213
LS05-82-09-013

Mr. W. G. Counsil, Vice President
Nuclear Engineering and Operations
Connecticut Yankee Atomic Power Company
Post Office Box 270
Hartford, Connecticut 06101

Dear Mr. Counsil:

SUBJECT: SEP TOPIC III-2, WIND AND TORNADO LOADINGS
HADDAM NECK PLANT

Enclosed is an evaluation of SEP Topic III-2. This evaluation compares your facility as described in the Safety Analysis Report you supplied on December 14, 1981, and other information on Docket No. 50-213 with criteria used by the staff for licensing new facilities.

This evaluation will be a basic input to the integrated safety assessment of your facility. This topic may be changed in the future if your facility design is changed or if NRC criteria relating to this topic is modified before the integrated assessment is completed.

Sincerely,

Dennis M. Crutchfield, Chief
Operating Reactors Branch No. 5
Division of Licensing

SE04
DSU USE(02)
ADD:
S. BROWN

Enclosure:
As stated

cc w/enclosure:
See next page

9/2/82

OFFICE	SEPB:DL	SEPB:DL	SEPB:DL	SEPB:DL	ORB#5:PM	ORB#5:BC	AD:SA:DL
SURNAME	DPersinko:dk	SBrown	RHermann	WRussell	~~CTropf~~ J.Lyons	DCrutchfield	TIppolito
DATE	8/25/82	8/26/82	8/26/82	8/27/82	8/30/82	9/1/82	9/1/82

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SYSTEMATIC EVALUATION PROGRAM
TOPIC III-2

HADDAM NECK

TOPIC: III-2, WIND AND TORNADO LOADINGS

I. INTRODUCTION

The safety objective of this review is to assure that safety-related structures, systems and components are adequate to resist wind and tornado loadings including tornado pressure drop loading.

II. REVIEW CRITERIA

The review criteria governing this topic is General Design Criteria 2, design bases for protection against natural phenomena.

III. RELATED SAFETY TOPICS AND INTERFACES

1. Tornado missiles are reviewed in SEP Topic III-4.A.
2. Structures which are considered safety-related are given in SEP Topic III-1.
3. Wind and tornado parameters are given in SEP Topic II-2.A.
4. Design codes, criteria and load combinations are reviewed in SEP Topic III-7.B.

IV. REVIEW GUIDELINES

The currently accepted design criteria for wind and tornado loadings is outlined in Standard Review Plan Sections 3.3.1, 3.3.2, 3.8 and Regulatory Guides 1.76 and 1.117. Codes and standards used for the review of structures at the Haddam Neck facility are given in Enclosure 1 to this SER.

Site specific windspeed and tornado parameters were developed in Topic II-2.A and the appropriate values were identified for use as input to the wind and tornado loading analyses. Structures important to safety were reviewed in this topic to determine their ability to withstand these values from Topic II-2.A. Appropriate values for the Haddam Neck site are a 300 mph windspeed (corresponding to 230 psf dynamic pressure), a 2.25 psi (324 psf) differential pressure, and a 1.2 psi/sec rate of pressure drop. The evaluation and conclusions are based on a Safety Analysis Report supplied by the licensee, information available on Docket No. 50-213, and the information developed by the staff given in Enclosure 1 to this SER. Structural capacities were determined and are given in terms of strength and corresponding windspeed.

V. EVALUATION

Enclosure 1 is a report entitled, "Wind and Tornado Loadings" presenting our contractor's findings concerning the Haddam Neck facility. The report identifies limiting structural elements and their associated windspeed. The intent is to verify the SAR submitted by the licensee. No analyses were performed for safety-related systems and components. Systems and components important to safety not housed within safety-related structures should be addressed by the licensee.

Original Design and SAR Conclusions

According to the Safety Analysis Report and other information supplied by the licensee, structures at the site were designed for a straight wind velocity of 80 mph, per the Connecticut State Building Code (CSBC), 1957 as amended May 1, 1961. This corresponds to 20 psf between 50 and 100 feet above grade, and linearly increasing above 100 feet per the equation: 20 psf + 0.25 (h-20 feet). The equation results in a pressure of 23 psf at 150 feet. According to the CSBC, the force is distributed by applying 2/3 of the force normal to the windward face and 1/3 as a normal outward suction on the leeward face. Since no discussion of shape factors is contained in the CSBC, it is concluded that these forces were the actual applied forces on the structures. A 1/3 increase in stress was permitted for load combinations involving wind.

The licensee qualitatively described the ability of the plant to withstand tornado loads in his SAR. The SAR noted that plant modifications were performed in 1967 to increase the ability of plant structures to withstand tornado missiles. The SAR concluded that the turbine building, upper level of the Primary Auxiliary Building (PAB), upper level of the new and spent fuel building, service building (except control room), upper level of the screenwell, and auxiliary feedwater pumphouse would be adversely affected by the site-specific tornado loads.

The SAR also concluded that the following safe shutdown systems would be exposed upon loss of the siding:

1. ADV, steam generator vents, and other vent paths
2. Auxiliary feed pumps
3. Water sources - DWST, PWST, and primary water transfer pump
4. Service water system
5. CVCS
6. Emergency power systems (AC, DC) for the above equipment
7. Instrumentation for the above equipment

The SAR concluded that the effects of the tornado would be as follows:

1. Screenwell House - service water system would be exposed due to loss of siding.
2. Auxiliary Feedwater Pumphouse - portions of main steam and feed would be exposed due to loss of siding.
3. Service Building - switchgear room would be exposed due to loss of siding.
4. PAB and Turbine Building - loss of turbine building siding or siding on the PAB would not affect safe shutdown capability.
5. New and Spent Fuel Building - Loss of siding would expose the pool; however, GE has analyzed this for tornado effects and has concluded that there would be no significant water removal.

Discussion

Current criteria for straight wind loading is given in Standard Review Plan 2.3.1 which references ANSI A58.1. Current criteria requires design for straight wind with a probability of exceedance in one year of 10^{-2} and of 10^{-7} for a tornado. Straight wind loads differ from tornado loads in that straight wind loads are considered in different load combinations, have different load factors in ultimate strength design of concrete and have different acceptance criteria than tornado wind loads. Additionally, straight wind design includes such aspects as gust factors and variation of force with height whereas tornado design does not. Buildings at Haddam Neck were originally designed as stated previously. ANSI A58.1 specifies a 10^{-2} wind which is approximately 85 mph at an elevation of 30' above grade. Per current criteria, load combinations involving dead, live, wind, pipe reactions, and thermal are allowed a 30% increase in allowable stresses for concrete structures if working stress method are used and a 50% increase in stress for steel structures if elastic design methods are used. The original design by the licensee utilized working stress design methods for steel and concrete design; therefore, the load factors used in the original design are the same as current criteria.

It is not known what wind load was used in the original design below elevation 30' above grade. The magnitude of the straight wind loads, including localized effects, used in the original design is less than that required by current criteria which specifies ANSI A58.1, 1972, Exposure C and results in a basic windspeed of 85 mph. It should be noted that according to the site-specific wind study given in SEP Topic II-2.A, the 10^{-2} basic windspeed at elevation 30' is 62 mph. Also, Exposure C is intended for flat, open country whereas Haddam Neck is located in wooded, rolling terrain so that Exposure B is more appropriate. The original design wind loads at Haddam Neck are greater than the loads imposed by a basic windspeed of 62 mph with an ANSI A58.1, 1972, Exposure B distribution (except below elevation 50', where no original design information is available) for both global and local loads. Furthermore from calculations performed by the staff, it appears that the as-built structures at Haddam Neck are adequate to resist ANSI A58.1, 1972, 85 mph basic windspeed loads for Exposure C, with the exception of the siding.

The 1/3 increase in allowable stress utilized by the licensee does not imply structural failure since increases of 30% and 50% in allowable stress above code allowable are permitted for load combinations involving all operating loads (dead load, live load, wind load, operating pipe reaction loads, and thermal loads). Since it is uncertain whether pipe reaction loads, thermal loads, and snow loads were included in the original design in combination with wind loads, it may be possible to overstress some structural elements if these loads are combined with wind.

Although this is possible, it is unlikely to occur for structures that are able to withstand the design tornado loads since these loads are significantly more demanding than the wind load and would, therefore, provide margin to accomodate pipe reaction loads and thermal loads when combined with wind.

The staff has analyzed the primary auxiliary building, diesel generator annex and control room. The results in terms of limiting windspeed at which acceptance criteria for limiting structural elements is exceeded is given below.

Structure	Element**	Cause of Failure***	Wind Speed (mph)	Corresponding**** Pressure (psf)
Primary Auxiliary Building	Reinforced concrete walls and columns	-	-	-
	14B22 roof beam	2	82	35
		3	97	33
		1	138	48
	W8x24 column	2	122	76
		1	193	95
	12W27 roof beam	2	143	104
		1	241	149
Diesel Generator Annex	Reinforced concrete walls and roof slab	- None	- >300	-

Structure	Element**	Cause of Failure***	Wind Speed (mph)	Corresponding**** Pressure (psf)
Control Room	Reinforced concrete walls and roof slab	>300	>300	-
	Steel bracing system	-	-	-

* The ratings of some structural components are not definitive; rather, they are estimates based on approximate modeling.

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

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

****Pressure given is either velocity pressure or differential pressure.

The values presented above are given for tornado dynamic pressure (otherwise known as velocity pressure), differential pressure, and high straight wind pressure. The allowable stresses for the tornado loads are according to SRP Section 3.8 which permits stress increases above code allowables for certain types of extreme loadings. The straight wind (non-tornado generated) capacity is also given because it becomes the controlling event for tornado velocities under 80 mph at Haddam Neck.

The straight wind capacity is calculated based on straight wind criteria (e.g., wind velocities vary with height). The capacity given has been normalized to 30 feet above grade since this is the elevation at which basic wind pressures are given for straight winds and because the report performed by McDonald for SEP Topic II-2.A has normalized the straight wind probability curve to elevation 30'. It should be noted that the straight wind capacities given above have not included the 50% increase in stress allowables for steel since the increase is only permitted for the load combination including pipe reaction loads and thermal loads. If it can be shown that these loads do not significantly add to the loads applied to the wind resisting structure, wind velocity for steel can be increased by approximately 22%.

The results obtained by the staff generally support the qualitative assessment by the licensee. It was found that the reinforced concrete portions of the PAB, the diesel generator annex and control room are adequate to withstand the design tornado loads. It was found that the steel portion of the PAB cannot withstand the full loads imparted by the design basis tornado. Additionally, from previous analyses, it is concluded that the siding would also not be capable of withstanding the design basis tornado loads. It should be noted that foundations and soil pressures were not investigated by the staff. Since the loads being imparted are greater than the original design, it may be possible that foundations or soil pressures may be limiting.

The licensee has stated that there are no exterior safety-related maso: , ılls; therefore, the usually low capacity walls most likely are not a concern at Haddam Neck for wind and tornado loads. However, in order to conclude that masonry walls are not a concern at Haddam Neck, the licensee should determine whether there are interior compartments or walls that will be subject to velocity or differential pressure upon failure of a weak exterior wall such as siding.

The capacity of the structural portion (structural frame and siding) should be determined in order to conclude that the spent fuel pool will not be impacted by its failure. Alternatively, the structure can be shown acceptable if it can be shown that such a failure is bounded by a previously analyzed impact upon the pool.

Roof decks consisting of built-up roofing as opposed to structural roof slabs made of concrete were not investigated by the staff. It is expected that such roofs will have minimal resistance to differential pressure.

VI. CONCLUSIONS

It is concluded that portions of some structures cannot withstand the postulated design basis tornado load of 300 mph wind and 2.25 psi pressure drop.

The staff concludes that portions of the PAB, the diesel generator annex and the control room can adequately resist design basis tornado loads.

The licensee should: 1) implement modifications for the following structures to meet the design basis tornado loads, 2) demonstrate that the consequences of their failure if subjected to tornado loads are acceptable, or 3) demonstrate adequate resistance for smaller tornado loadings and that the risk associated from larger tornado loadings is acceptable.

1. Upper portion of the primary auxiliary building.
2. Ventilation stack.
3. Interior masonry walls protected by exterior walls with minimal tornado resistance (e.g., siding).
4. Auxiliary feedwater pumphouse (structural portion and siding system).
5. Screenwell house (structural portion and siding system).
6. Service building (structural portion and siding system).
7. Roof decks on Category 1 structures.
8. Siding system on any other Category 1 structures.
9. New and spent fuel pool super structure.

For safety related components not inside qualified structures, the licensee should either demonstrate acceptability for tornado loads or that the consequences of failure if subjected to tornado loads are acceptable.

It should be determined whether operating pipe reaction loads, thermal loads and snow loads were considered with wind in the original design. If these loads were not, the effect of combining them should be addressed.

The licensee should demonstrate that foundations and soil capacities are greater than original design and that they are not limiting.

The need to implement modifications or perform additional analysis in order to assure that structures, systems and components can adequately resist wind and tornado loads will be determined during the integrated assessment.

TECHNICAL EVALUATION REPORT

WIND AND TORNADO LOADINGS (SEP, III-2)

CONNECTICUT YANKEE ATOMIC POWER COMPANY
HADDAM NECK NUCLEAR POWER PLANT

NRC DOCKET NO. 50-213

NRC TAC NO. 41605

NRC CONTRACT NO. NRC-03-79-118

FRC PROJECT C5257

FRC ASSIGNMENT 14

FRC TASK 406

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August 11, 1982

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ANO not designated

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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 Connecticut Yankee Atomic Power Company (CYAPCO) for the Haddam Neck Nuclear Power Plant, Unit 1 [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 adeq acy 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.

FRC was originally charged with auditing the design calculations supporting the conclusions of the Haddam Neck SAR. However, these calculations were not provided by CYAPCO. 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 Haddam Neck 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 can then be used to assess the conclusions reported in the SAR.

1.3 PLANT-SPECIFIC BACKGROUND

The review of the Haddam Neck SAR was begun in May 1982. Prior to that time, CYAPCO responded to NRC requests for information by providing architectural-engineering structural drawings. Additional sources of information were a CYAPCO letter with an addendum to the SAR on the SEP structural topics [3] and the plant final safety analysis report [4].

In the SAR, CYAPCO reviewed only the minimum systems and components required to accomplish a plant shutdown and to maintain a safe shutdown condition. CYAPCO concludes by inspection that loadings reflecting the site-specific tornado would adversely affect the turbine building; the upper levels of the primary auxiliary building, the new and spent fuel building, and the screenwell; the service building (with the exception of the control room); and the auxiliary feedwater pumphouse. The conclusions stated by CYAPCO in the SAR are summarized in Table 1.

The structures to be evaluated in this review were identified on the basis of the SAR and the addendum to the SAR submitted by the Licensee. The primary auxiliary building, the control room, and the diesel generator building were identified as the priority review structures.

The original wind loading criteria of the Haddam Neck plant did not include tornado loadings. The wind load used in the design of all structures averaged approximately 28 psf, in accordance with the State of Connecticut Basic Building Code [5]. The stresses due to wind loads were evaluated at a

Table 1. Summary of Conclusions from Haddam Neck SEP Topic III-2 SAR

	Class I Structures*	Postulated Effects of Hypothetical Tornado**
1.	Screenwell House	Possible loss of exterior siding would expose the service water system, including the pump motors.
2.	Auxiliary Feedwater Pumphouse	Possible loss of exterior siding on the north and south sides would expose portions of the main steam and feedwater systems.
3.	Service Building	Possible loss of exterior siding would expose the switchgear room. The control room would not be affected.
4.	Turbine Building	Possible loss of exterior siding would not affect the plant's ability to achieve and maintain a safe shutdown condition.
5.	Primary Auxiliary Building	Possible loss of exterior siding in the upper level of the building would not affect the plant's ability to achieve and maintain a safe shutdown condition.
6.	New and Spent Fuel Building	Possible loss of exterior siding would expose the spent fuel pool but would not result in significant water removal from the pool.

*It has been assumed that the wind and tornado loads would not govern the reanalysis of the containment, diesel generator building, and the portions of safety-related structures not included above.

**The above review did not include missile effects.

3-1/3% increase over code allowables. According to the SAR, the Haddam Neck plant was evaluated for a tornado defined as a 300-mph horizontal wind; certain modifications to the plant were implemented as a result.

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 analyses 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 Haddam Neck 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 Haddam Neck 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]

3. TECHNICAL EVALUATION

3.1 GENERAL INFORMATION

The structures included in this review are the primary auxiliary building, the diesel generator annex, and the control room. These structures are classified seismically as Category I Nuclear Safety Related. The plan of the building arrangement at the Haddam Neck site is as shown in Figure 1.

The DBT characteristics taken as a basis for analysis are the following (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). 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 venting, the tornado loads are still assumed to exist so that the strength of other, stronger structural elements can be analyzed.

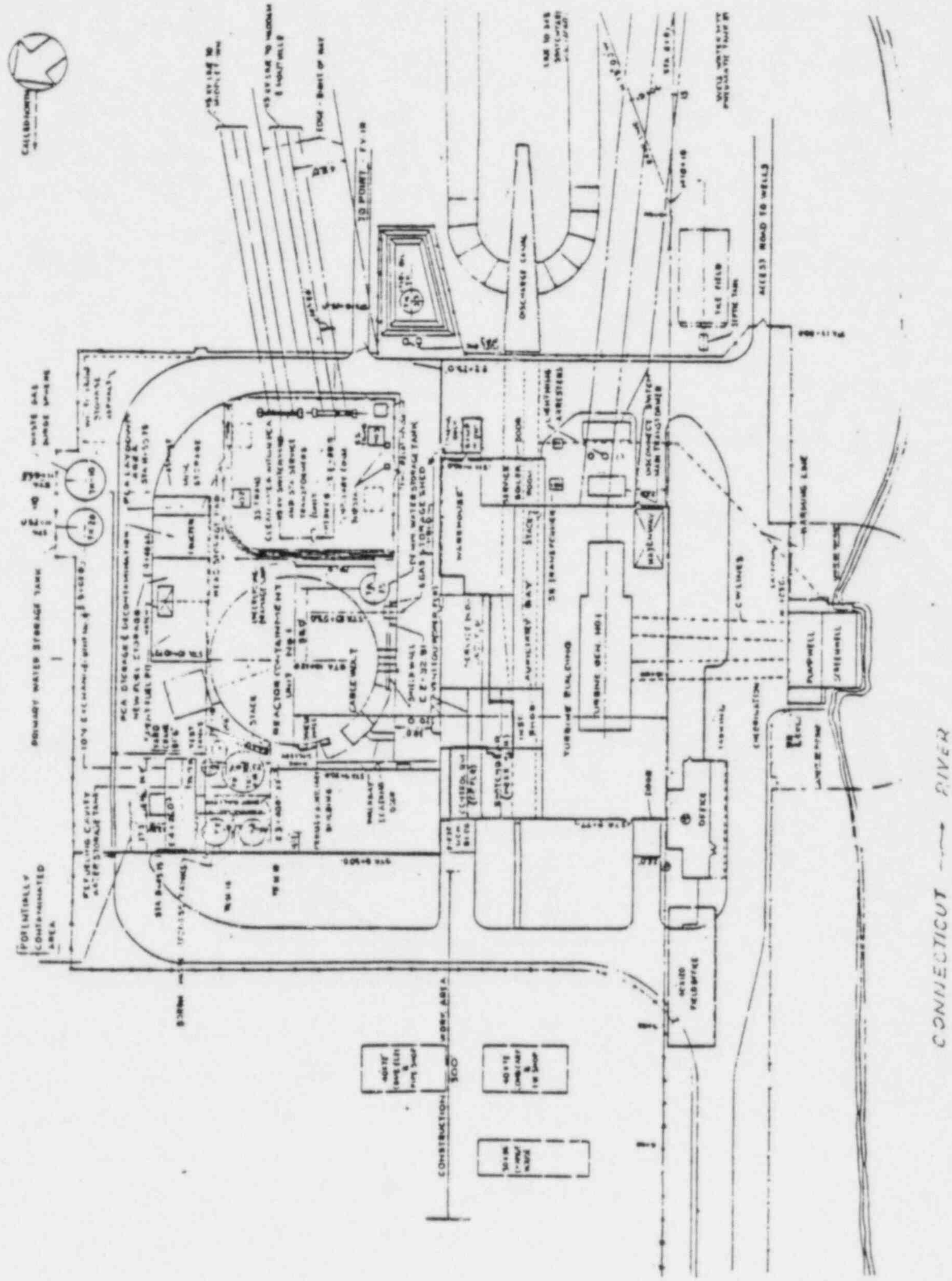

Figure 1. Site Plan

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 PRIMARY AUXILIARY BUILDING

3.2.1 Evaluation

The primary auxiliary building is located north of the reactor containment to which it is connected by a pipe gallery. This structure is constructed of steel and reinforced concrete. The roof consists of slabs resting on reinforced concrete walls, and steel decks resting on a roof steel system (El. 53 ft 8-1/2 in). The roof steel is supported by steel columns which terminate and rest on concrete columns and slabs at elevation 35 ft

6 in. The steel columns support girts to which siding is attached. The floors of this structure and all of the walls below elevation 35 ft 6 in are composed of reinforced concrete. The adjacent grade elevation is 21 ft.

The west side of the primary auxiliary building is adjacent to the diesel generator building. The east, north, and south sides are exposed to the atmosphere.

The lateral wind loads are applied to the structure through the girt-column connections. The columns transmit this loading to the roof steel and to the concrete structure below. Bracing in the roof steel plan transmits the lateral forces to vertical bracing. The steel columns have been modeled as simply supported and subjected to combined bending and axial loads. The analysis of steel columns can be found on pages A-18 and A-19.

The roof steel consists of beams spanning in the north-south direction supported by girders spanning in the east-west direction. The roof deck is assumed to remain partially intact to the roof beams and to transmit uplift loadings. The roof steel decking was not analyzed due to lack of information, but the roof beams are included in this study and analyzed for uplift pressure loading. The analysis of roof beams can be found on pages A-20 through A-22.

The reinforced concrete walls below elevation 35 ft 6 in are 12-in-thick walls with both vertical and horizontal reinforcement. These walls frame into concrete floor slabs, beams, columns, and interior concrete walls. Each wall panel has been analyzed as a two-way slab that tranfers loads in the horizontal direction to the nearest columns or interior wall, and in the vertical direction to the adjacent floor slabs. The analysis of walls can be found on pages A-1 through A-12. An analysis of a reinforced concrete column supporting axial loads and lateral wall panel reactions can be found on pages A-13 through A-17.

3.2.2 Conclusion

All of the concrete elements examined in this review have adequate resistance to tornado loadings. Inadequate components of the roof steel are the 14B22 beams and the 12W27 beams. The 14B22 beams have a limit rating of

0.24 psi (82 mph) for differential pressure, 48.4 psf (138 mph) for tornado dynamic pressure, and 32.9 psf (97 mph) for high wind dynamic pressure. The 12W27 beams have a limit rating of 0.72 psi (143 mph) for differential pressure and 149 psf (241 mph) for tornado dynamic pressure. The W8x24 column Fy-13-1/4 has a limit rating of 0.527 psi (122 mph) for differential pressure and 94.9 psf (193 mph) for tornado dynamic pressure.

3.3 DIESEL GENERATOR ANNEX

3.3.1 Evaluation

The diesel generator building is located to the northwest of the reactor containment. This building has two sections: the older section of the diesel generator building, which is adjacent to the primary auxiliary building, and the diesel generator annex. The annex houses the diesel generators.

The sides of the diesel generator annex are 2-ft-thick reinforced concrete walls with vertical and horizontal reinforcements on each face. The roof is a 2-ft-thick concrete slab with reinforcement in both directions and on both faces. The high point of the concrete slab is at elevation 42 ft 3 in, while the adjacent grade is at elevation 20 ft 10 in. The walls and the reinforced concrete foundation mat are integral; the foundation mat is built at grade level.

The roof slab is subjected to uplift pressure during a tornado strike but is not analyzed since the dead weight of the slab is comparable to the pressure loadings. The reinforced concrete walls are analyzed as two-way slabs; this analysis can be found on pages B-1 to B-5.

3.3.2 Conclusion

The reinforced concrete roof slab and the reinforced concrete walls of the new diesel generator building can safely withstand the tornado loadings.

3.4 CONTROL ROOM

3.4.1 Evaluation

The control room is located in the service building northwest of the reactor containment. The walls of the room are constructed of reinforced concrete, 20 in thick on the north and east sides and 16 in thick on the south and west sides. The roof slab is 22 in thick and is supported by reinforced concrete beams and the walls. The concrete structure is in turn supported by braced structural steel framing. The operating floor is at elevation 59 ft 6 in and the top of the roof slab is at elevation 77 ft 9 in. The adjacent grade is at elevation 20 ft 9 in.

The west side of the control room is adjacent to the turbine building. The north, east, and south sides are exposed to the atmosphere.

Lateral loads are applied to the concrete walls and are transmitted to the structural steel framing at the level of the operating floor slab. These forces are then carried to the foundation by the steel bracing.

The roof slab is subjected to uplift pressures during a tornado strike but is not analyzed since the dead weight of the slab is comparable to the applied loading. The reinforced concrete walls are subjected to positive and negative pressures and are analyzed as one-way slabs in the vertical direction; this analysis can be found on pages C-1 to C-4. The steel braces resist the total lateral load on the structure and induce axial loads in the columns. The review of these members are found on pages C-5 to C-7.

3.4.2 Conclusion

The reinforced concrete roof slab, the structural steel bracing, and the reinforced concrete walls of the control room can safely withstand the tornado loadings.

4. CONCLUSIONS

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

Table 2. Strength Summary of the Structural Components Analyzed*

Structure	Element**	Cause of Failure***	Wind Speed (mph)
Primary Auxiliary Building	Reinforced concrete walls and columns	-	-
	14B22 roof beam	2	82
		3	97
		1	138
	W8x24 column	2	122
		1	193
	12W27 roof beam	2	143
		1	241
Diesel Generator Annex	Reinforced concrete walls and roof slab	-	-
Control Room	Reinforced concrete walls and roof slab	-	-
	Steel bracing system	-	-

*The ratings of some structural components are not definitive; rather, they are estimates based on approximate modeling.

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

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

In the SEP Topic III-2 SAR, CYAPCO excluded the diesel generator annex as a possible endangered structure during a tornado strike. This report confirms that conclusion. It is also confirmed here that large areas of the primary auxiliary building are immune to tornado damage, thus supporting the CYAPCO conclusion that this structure would maintain its safety-related function. As suggested by CYAPCO, the siding system (girts, panels, and fasteners) have limited tornado loading resistance. During an earlier review [24], it was found that a typical siding system would fail under low tornado differential pressures. The extent to which such a failure would impede plant safe-shutdown capability is addressed in the plant SAR.

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TER-C5257-400

APPENDIX A

PRIMARY AUXILIARY BUILDING DESIGN REVIEW CALCULATIONS

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THE EAST SIDE WALL OR THE N-LINE WALL IS EXPOSED. THIS EAST SIDE HAS REINFORCED CONCRETE WALL UPTO ELEV. 35'6" ABOVE GRADE ELEV. 21'

THE N-LINE WALL THICKNESS VARIES BETWEEN 2ft & 1ft. A SECTION BETWEEN 11 3/4 AND 11 1/4 COLUMN LINES IS 1FT THICK WITH LEAST REINFORCEMENT. i.e. # 4 BARS @ 12"

VERTICAL HEIGHT = 35'6" - 21' = 14'6"

HORIZONTAL LATERAL WIDTH = 13'

BOTH VERTICAL & HORIZONTAL REINFORCEMENT IS SAME

WE ANALYZE THESE WALLS AS TWO-WAY FLAT PLATE.

AS PER ACI 318-77 CHAPTER 13

$$\text{ABSOLUTE SUM OF MOMENT} = \frac{wl^2}{8}$$

FOR 1FT. WIDE SECTION, DIFF. PRESSURE CASE

$w = 324$ lbs/ft.

$$\therefore M = \frac{324}{1000} \frac{(14.5)^2}{8} = 8.52 \text{ k-ft.}$$

DISTRIBUTE MOMENT ACCORDING TO FACTORS GIVEN

FOR WORST CASE THE $M_{ACTUAL} = 0.75M = 0.75 \times 8.52$

$= 6.39$ k-ft

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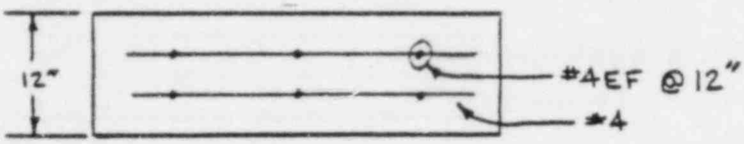
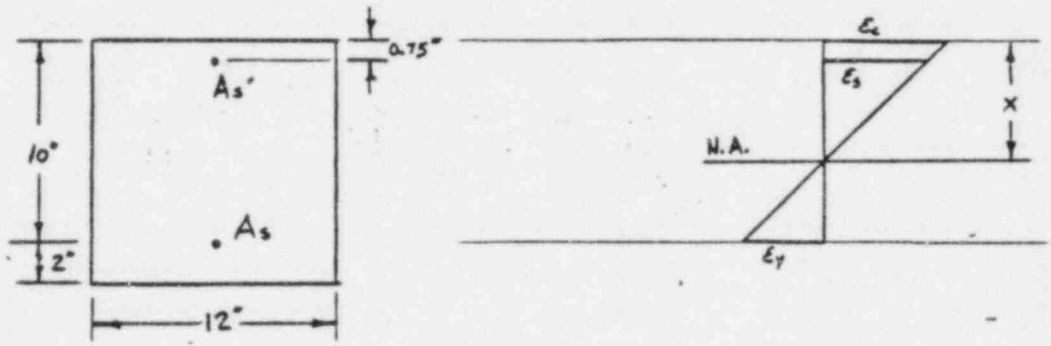
Title REINFORCED CONCRETE WALL — PRIMARY AUX BUILDING (N-LINE WALL)

N — LINE

Wall Section - between 35'6" & 21' (grade)



Find moment capacity of 1ft. wide section of wall



concrete $f'_c = 3$ ksi

steel $f_y = 40$ ksi

$A_s = A_s' = 1\#4 \text{ bar} = 0.20 \text{ in}^2$

From strain geometry

$$\varepsilon_s = \frac{\varepsilon_y (x - 0.75)}{(10 - x)} \qquad f_s = \frac{40 (x - 0.75)}{(10 - x)}$$

Tension $T = A_s f_y$

$= (0.20)(40)$

$\therefore T = 8$ ksi

Compression $C_c = 0.85 f'_c (0.85x) b$

$= (0.85)(3)(0.85x)(12)$

$\therefore C_c = 26.01\,x$

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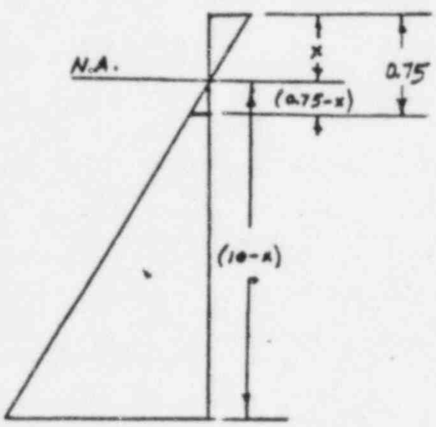
Title: REINFORCED CONCRETE WALL - PRIMARY AUX BUILDING (N - LINE WALL)

Assume Upper Steel in Tension

$$\varepsilon_s = \frac{\varepsilon_y\,(0.75 - x)}{(10 - x)} \qquad f_s = \frac{40\,(0.75 - x)}{(10 - x)}$$

$$T' = A_s' f_s = 0.2\left[\frac{40\,(0.75 - x)}{(10 - x)}\right] = \frac{8\,(0.75 - x)}{(10 - x)}$$



For equlibrium

$$C_c = T + T'$$

$$26.01\,x = 8 + \frac{8\,(0.75 - x)}{(10 - x)}$$

$$26.01x\,(10 - x) = 8(10 - x) + 8\,(.75 - x)$$

$$260.1\,x - 26.01\,x^2 = 80 - 8x + 6 - 8x$$

$$26.01\,x^2 - 276.1\,x + 86 = 0$$

$$x = \frac{276.1 \pm \sqrt{(276.1)^2 - 4(26.01)(86)}}{2\,(26.01)}$$

$$= \frac{276.1 \pm 259.39}{52.02}$$

$$x = .32''$$

$$\therefore\ C_c = 26.01\,x$$
$$= 26.01\,(.32)$$
$$= 8.32 \text{ KIPS}$$

$$\therefore\ T' = \frac{8\,(0.75 - .32)}{(10 - .32)}$$
$$= .355 \text{ KIPS}$$

$\therefore$ Moment capacity of section will be

$a = 0.85\,x = .272$

$$M = T\,(10 - a/2) + T'\,(0.75 - a/2)$$
$$= 8\,(10 - .136) + .355\,(0.75 - .136)$$
$$= 79.13 \text{ K-in.}$$
$$\therefore\ M_{ALLOW} = 6.59 \text{ K-ft.}$$

$$\therefore\ M_{ALLOW} > M_{ACTUAL} = 6.39 \text{ k-ft}$$

$\therefore$ O.K.

FORM CS-FRC-81

THE NORTH SIDE WALL OR THE 13¼-LINE WALL IS EXPOSED. THE NORTH SIDE HAS REINFORCED CONCRETE WALL UPTO ELEV 35' 6" ABOVE GRADE ELEV 21' BETWEEN COL. LINES G & H THE REINFORCED CONCRETE WALL CONTINUES UPTO ELEV. 53' 10".

THE CONCRETE WALLS CAN BE ANALYZED AS TWO-WAY FLAT PLATES. AS PER ACI 318-77, CHAPTER 13

$$\text{ABSOLUTE SUM OF MOMENTS} = \frac{w\ell^2}{8}$$

THIS MOMENT SHOULD BE DISTRIBUTED ACCORDING TO FACTORS GIVEN IN SECTION 13.6.3.

a) REINFORCED CONCRETE WALL SECTION BETWEEN ELEV. 53'10" & 35'6"

THE WALL IS 10" THICK

THE HORIZONTAL REINFORCEMENT IS #4 BARS @ 12"

FOR DIFFERENTIAL PRESSURE CASE, FOR 1 FT. WIDE SECTION

$w = 324$ lbs/ft.

LATERAL WIDTH BETWEEN COL. LINES G & H

$\ell = 24$ ft.

$$\therefore\ M = \frac{324}{1000}\,\frac{(24)^2}{8} = 23.328 \text{ k-ft.}$$

THIS MOMENT WHEN DISTRIBUTED ACCORDING TO DISTRIBUTION FACTORS, FOR WORST CASE WILL BE, $M_{ACTUAL} = 0.75M$

$= 0.75 \times 23.328 = \underline{17.496 \text{ k-ft}}$

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Title: REINFORCED CONCRETE WALL – PRIMARY AUX. BUILDING.

13 1/4 – LINE WALL CONT'D.

b) REINFORCED CONCRETE WALL SECTION BETWEEN ELEV. 35'6" & GRADE.

FOR DIFFERENTIAL PRESSURE CASE, FOR 1FT WIDE SECTION, WIND LOAD $w = 324$ lbs/ft.

LATERAL WIDTH BETWEEN COL LINES E & G
$l = 29'\,9''$

$$\therefore M = \frac{324}{1000}\,\frac{(29.75)^2}{8}$$

$$= 35.845 \text{ k-ft.}$$

THIS MOMENT WHEN DISTRIBUTED ACCORDING TO DISTRIBUTION FACTORS

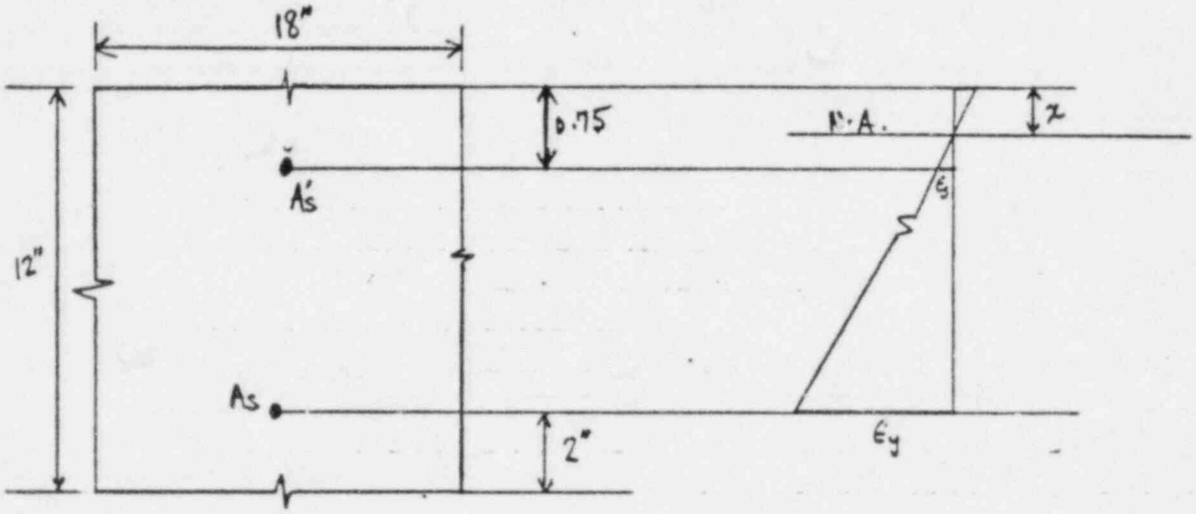
FOR WORST CASE WILL BE $M_{actual} = 0.75M$

$$\therefore M_{actual} = \underline{26.884 \text{ k-ft}}.$$

Title REINFORCED CONCRETE WALL - PRIMARY AUX. BUILDING. (13 1/4 - LINE WALL)

13 1/4 - LINE WALL BETWEEN ELEV 35'6" & GRADE ELEV 21'

FIND MOMENT CAPACITY OF 18" THICK WALL SECTION WITH REINFORCEMENT #5 BAR IN EACH FACE @ 12"
CONSIDER A 12" WIDE SECTION TAKEN LATERALLY.



All steel bars are #5 @ 12"

Steel $f_y = 40\,ksi$

$f_c' = 3\,ksi$

Area of steel $A_s = A_s' = 0.31$

Tension $T = A_s f_y$

$= 0.31 \times 40$

$\therefore T = 12.4$ kips

Assume both steel are in tension

From strain geometry

$$\epsilon_s = \epsilon_y \frac{(0.75 - x)}{(10 - x)} \Rightarrow f_s = 29{,}000\,\epsilon_s$$

$$= \frac{40(0.75 - x)}{(10 - x)} = \frac{30 - 40x}{(10 - x)}$$

Tension in upper steel $T' = A_s' f_s$

$$= 0.31 \frac{(30 - 40x)}{(10-x)}$$

$$\therefore T' = \frac{9.3 - 12.4x}{(10-x)}$$

Compression

$$C_c = 0.85 f_c' (0.85x) b$$

$$= 0.85 (3)(0.85x) 18$$

$$\therefore C_c = 39.015x$$

For Equilibrium $C_c = T + T'$

$$\therefore 39.015x = 12.4 + \frac{9.3 - 12.4x}{(10-x)}$$

$$39.015x(10-x) = 12.4(10-x) + 9.3 - 12.4x$$

$$390.15x - 39.015x^2 = 124 - 12.4x + 9.3 - 12.4x$$

$$39.015x^2 - 414.95x + 133.3 = 0$$

$$x = \frac{+414.95 \pm \sqrt{(-414.95)^2 - 4(39.015)(133.3)}}{2(39.015)}$$

$$x = \frac{414.95 \pm 389.07}{2(39.015)} = .331581$$

$$\therefore C_c = 12.936633$$

$$T' = \frac{9.3 - 12.4x}{10 - x} = 0.5366332$$

MOMENT CAPACITY

$a = 0.85x$
$= 0.2818438$

$$M = T(d - a/2) + T'(0.75 - a/2)$$

$$= 12.4(10 - 0.1409219) + 0.5366(0.75 - 0.1409219)$$

$$= 12.4(9.8590781) + 0.5366(0.6090781)$$

$$= 122.25257 + 0.3268313$$

$$= 122.5794$$

$$\therefore M = 10.214952$$

$$\therefore M_{allow} = 10.215 \text{ K-ft}$$

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Title REINFORCED CONCRETE WALL - PRIMARY AUX. BUILDING (13/4-LINE WALL)

Convert Mallow to Corresponding WIND SPEEDS

- Allowable Wind Load

$$W = \frac{8\, M_{allow}}{0.75(\ell^2)} = \frac{8(10.215)(10^3)}{0.75(29.75)^2}$$

$\therefore W = 123.10937$ psf

a) For Differential Pressure,

Allowable Pressure = 0.855 psi

$\Rightarrow$ Corresponding WIND SPEED

$$= \sqrt{\frac{123.0212}{0.00511}} = 155.22 \text{ mph}$$

b) For Tornado Dynamic Pressure

Allowable Pressure $= \frac{123.1094}{0.8} = 153.89$ psf

(USE Shape Factor = 0.8)

$\Rightarrow$ Corresponding WIND SPEED

$$= \sqrt{\frac{153.88671}{0.00256}} = 245.2 \text{ mph}$$

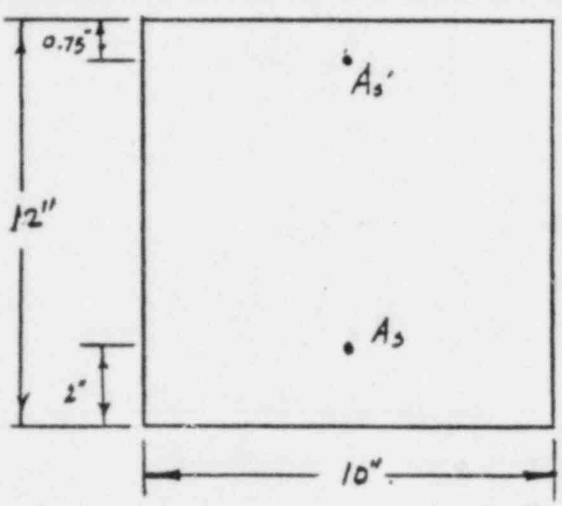
FORM CS-FRC-81

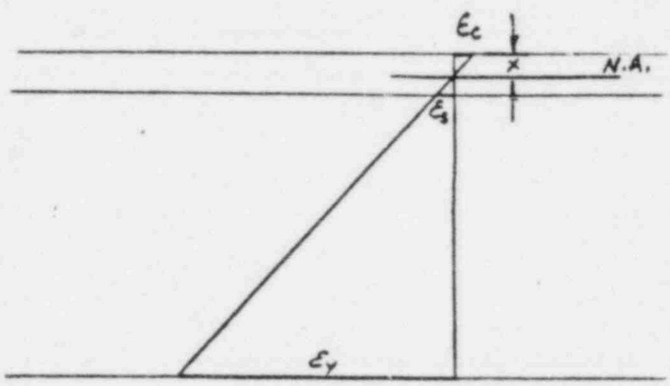
Title REINFORCED CONCRETE WALL – PRIMARY AUX BUILDING

13¼ – LINE WALL

Find moment capacity of 10" thick wall section with reinforcement #4 bar each face @ 12", between elev 53'10" & 35'6"

Consider a 12" wide section taken laterally





$A_s = A_s' =$ Area of #4 bar $= 0.20\ in.^2$

Tension $\quad T = A_s f_y$
$= (0.2)(40)$
$\therefore T = 8$ KIPS

Assume upper steel in Tension. From strain geometry

$$\varepsilon_s = \frac{\varepsilon_y (0.75 - x)}{(10 - x)} \qquad f_s = \frac{40(0.75 - x)}{(10 - x)} = \frac{30 - 40x}{(10 - x)}$$

$$T' = A_s' f_s = \frac{0.2(30 - 40x)}{(10 - x)}$$

$$\therefore T' = \frac{6 - 8x}{(10 - x)}$$

Compression $\quad C_c = 0.85 f_c' (0.85x) b$
$= 0.85(3)(0.85x)10$
$\therefore C_c = 21.675x$

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

$$21.675x = 8 + \frac{(6-8x)}{(10-x)}$$

$$21.675x(10-x) = 8(10-x) + 6 - 8x$$

$$216.75x - 21.675x^2 = 80 - 8x + 6 - 8x$$

$$21.675x^2 - 232.75x + 86 = 0$$

$$x = \frac{232.75 \pm \sqrt{(232.75)^2 - 4(21.675)(86)}}{2(21.675)}$$

$$x = \frac{232.75 \pm 216.14}{43.35}$$

$$x = .38''$$

$$\therefore C_c = 21.675x$$
$$= 21.675(.38)$$
$$C_c = 8.24 \text{ KIPS}$$

$$\therefore T' = \frac{6-8x}{(10-x)} = \frac{6-8(.38)}{(10-.38)} = .31 \text{ KIPS}$$

$\therefore$ Moment capacity will be

$d = 12 - 2 = 10$
$a = 0.85x$
$= 0.85(.38)$
$= .323$

$$M = T(d - a/2) + T'(0.75 - a/2)$$
$$= 8(10 - .1615) + .31(0.75 - .1615)$$
$$M = 78.89 \text{ k-in.}$$
$$= 6.57 \text{ k-ft}$$

$$\therefore M_{ALLOW} = 6.574 \text{ k-ft.}$$

WE FIND $\quad M_{ALLOW} < M_{ACTUAL} = 17.496$ k-ft.

NOW WE CONVERT M_{ALLOW} TO CORRESPONDING WIND SPEEDS.

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Title: REINFORCED CONCRETE WALL — PRIMARY AUX. BUILDING. (13'/4 - LINE WALL

$M_{ALLOW} = 6.574$ k-ft.

$\therefore$ ALLOWABLE WIND LOAD: $w = \dfrac{8M}{0.75 l^2}$

$= \dfrac{8 \times 6.574}{0.75 (24)^2} \times 1000$

$\therefore w = 121.741$ lbs/ft.

FOR 1FT - WIDE SECTION PRESSURE = 121.741 psf.

a) FOR DIFF. PRESSURE, ALLOWABLE PRESSURE = 0.85 psi

$\Rightarrow$ CORRESPONDING WIND SPEED $= \sqrt{\dfrac{121.741}{0.00511}} = \underline{154.35 \text{ mph}}$

b) FOR TORNADO DYNAMIC PRESSURE $= \dfrac{121.741}{0.8}$
(USE 0.8 SHAPE FACTOR)

$= 152.2$ psf

$\Rightarrow$ CORRESPONDING WIND SPEED $= \sqrt{\dfrac{152.2}{0.00256}} = \underline{243.8 \text{ mph}}$

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Title: REINFORCED CONCRETE COLUMN — PRIMARY AUXILIARY BUILDING

IN WALL ANALYSIS, WE HAVE ASSUMED THAT ALL WALLS ACT AS TWO-WAY FLAT SLABS AND TRANSFER ALL LOADS IN LATERAL DIRECTION TO THE REINFORCED CONCRETE COLUMNS

WE ANALYZE A 2'x2' COLUMN SECTION ON EAST SIDE. THIS COLUMN IS BETWEEN ELEV 35'6" AND GRADE AT ELEV 21' THE HEIGHT OF COLUMN IS 14'6"

THIS LATERAL WIDTH OF 22.75 ft LOAD IS TRANSFERED TO THE COLUMN. ASSUMING COLUMN TO BE SIMPLY SUPPORTED AT GRADE LEVEL ELEV 21' AND ELEV 35'6" THE MOMENT CAN BE TAKEN AS $M = \frac{wl^2}{8}$

FOR PRESSURE DROP OF 324 psf FOR LATERAL WIDTH OF 22.75' WE HAVE LOAD $w = 22.75 \times \frac{324}{1000}$

$= 7.371$ lbs/ft.

HEIGHT OF COLUMN IS 14.5 ft. $\therefore\ l = 14.5$ ft.

$$\therefore\ M = \frac{7.371\,(14.5)^2}{8}$$

$$\therefore\ M = 193.72 \text{ k-ft}$$

$\therefore$ MAX. ACTUAL MOMENT $M_{actual} = 193.72$ k-ft.

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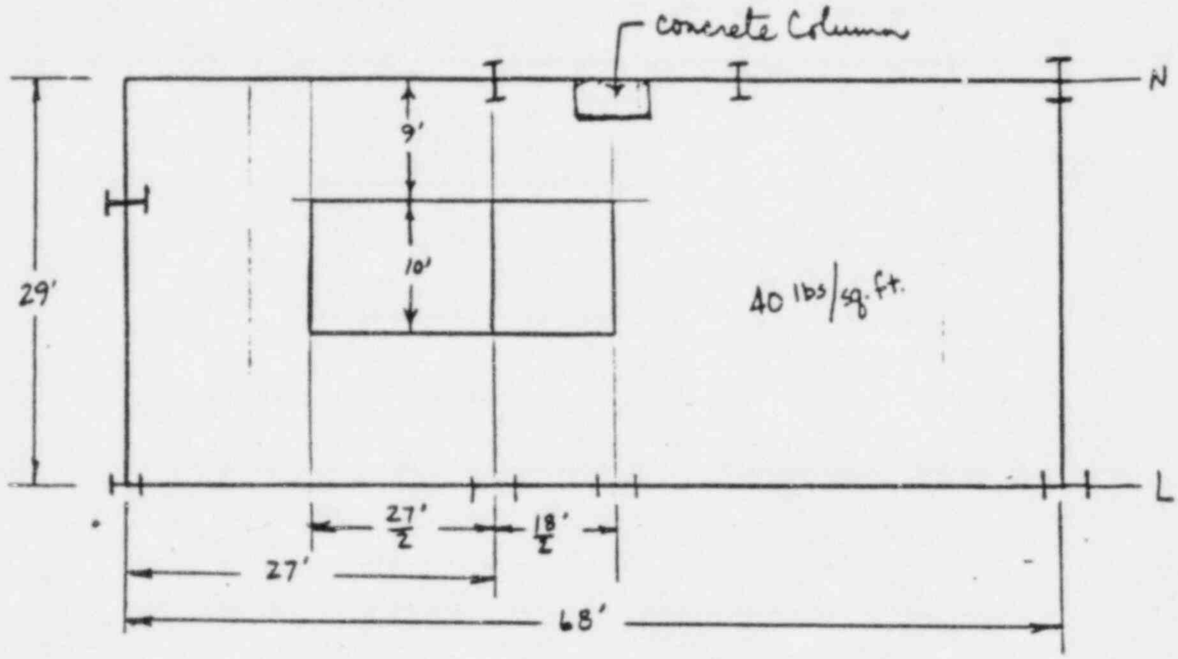
Title REINFORCED CONCRETE COLUMN ON EAST SIDE — PRIMARY AUXILIARY BUILDING.

Calculation of axial load on concrete column.

a) Axial load from roof level



Roof L.L. $= 9' \times 9' + 9' \times 10' + 13.5' \times 10'$
$= 306 \text{ ft}^2$

Load $= (40 \text{ psf})(306 \text{ ft}^2) = 12240$ lbs
$= 12.24$ kips

b) Axial load from floor at elev. 35'6"

TOTAL Area = 29' x 68'
12" thick concrete slab

L.L. = 250 psf

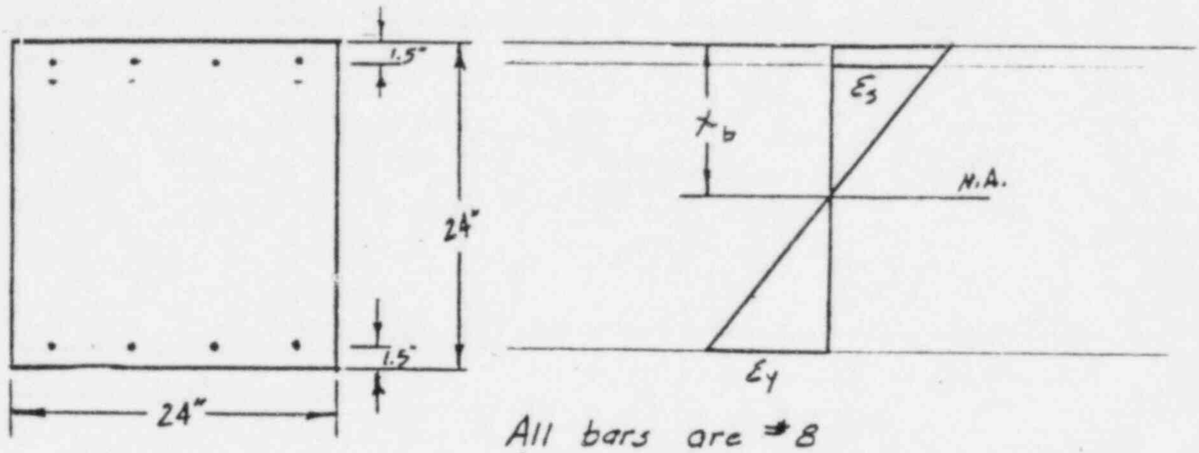
Concrete slab wt. = $150 \text{ lbs/ft}^3 \Rightarrow 150 \text{ lb/ft}^2$ for 1 ft. thick slab

∴ Total load = 400 psf

$\left(\frac{29}{2}\right) 400 = 5800 \text{ lb/ft}$ $\quad (5800)(68') = \frac{394400}{6} \text{ lbs} \simeq 66 \text{ kips}$

Title REINFORCED CONCRETE COLUMN – EAST SIDE OF PRIMARY AUX. BLDG.

∴ column could be checked for 66 + 12.24 = 78.24 KIPS



All bars are #8
Area of #8 = 0.79 in²

steel $f_y = 40$ ksi

Concrete $f_c' = 3$ ksi

For balance condition, neutral axis

$$x_b = \frac{0.003\,(d)}{0.003 + 1.379 \times 10^{-3}} \qquad d = 24 - 1.5 = 22.5$$

$$= \frac{0.003\,(22.5)}{4.379 \times 10^{-3}}$$

$$\therefore x_b = 15.41"$$

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

$= 0.85(3)(0.85)(15.41)(24)$

$C_c = 801.63$ kips

$C_s = A'_s (f_y - 0.85 f'_c)$

$= 4(.79)(40 - 0.85(3))$

$C_s = 118.34$ kips

Tension $T = A_s f_y$

$= 40 \times .79 \times 4$

$T = 126.4$ kips

FORM CS-FRC-81

For equlibrium

$$P_b = C_c + C_s - T$$
$$= 801.63 + 118.34 - 126.4$$
$$= 793.57 \text{ kips}$$

$$a = .85(x)$$
$$= .85(15.41)$$
$$= 13.1$$

$$M_b = C_c(12 - a/2) + C_s(12) + T(12)$$
$$= 801.63(12 - 6.55) + 118.34(12) + 126.4(12)$$
$$= 7305.76 \text{ k-in.} = 608.8 \text{ k-ft.}$$

But actual load can be maximum about 80 kips.

then $\epsilon_s = \dfrac{\epsilon_y (x - 1.5)}{(22.5 - x)}$ $\qquad f_s = \dfrac{40 (x - 1.5)}{(22.5 - x)}$

$$C_s = A_s' \left[\frac{40 (x - 1.5)}{(22.5 - x)} - 2.55 \right]$$

$$C_c = 0.85(3)(0.85x)(24)$$
$$= 52.02x$$

$$T = A_s f_y = 126.4 \text{ kips}$$

$$P = C_c + C_s - T$$
$$80 = 52.02x + 4(.79)40 \frac{(x - 1.5)}{(22.5 - x)} - 4(.79)2.55 - 126.4$$

$$80 = 52.02x + \frac{126.4(x - 1.5)}{(22.5 - x)} - 8.058 - 126.4$$

$$0 = 52.02x(22.5 - x) + 126.4(x - 1.5) - 214.458(22.5 - x)$$
$$= 1170.45x - 52.02x^2 + 126.4x - 189.6 - 4825.305 + 214.458x$$

$$52.02x^2 - 1511.308x + 5014.905 = 0$$

$$x = \frac{1511.308 \pm \sqrt{(1511.308)^2 - 4(52.02)(5014.905)}}{2(52.02)}$$

$$x = \frac{1511.308 \pm 1113.8}{104.04}$$

$x = 3.82$ inches

$\therefore C_c = 52.02\, x$
$= 52.02\,(3.82)$
$= 198.72$ KIPS

$$C_s = 4(.79)\left[\frac{40\,(x-1.5)}{(22.5-x)} - 2.55\right]$$

$$= 4(.79)\left[\frac{40\,(3.82-1.5)}{(22.5-3.82)} - 2.55\right]$$

$= 7.64$ KIPS

$T = 126.4$ KIPS

The section is similar, so plastic centroid is at mid-depth of the section.

$\therefore$ Moment capacity will be

$a = 0.85x$
$= 0.85(3.82)$
$= 3.247$

$M = C_c(12 - a/2) + C_s(12) + T(12)$
$= 198.72(12 - 1.6235) + 7.64(12) + 126.4(12)$
$= 3670.5$ k-inch.
$= 305.9$ k-ft.

$\therefore$ Allowable moment $M_{allow} = 305.9$ k-ft

$\therefore M_{allow} > M_{actual} = 193.72$ k-ft

$\therefore$ O.K

Title: STEEL COLUMNS — PRIMARY AUXILIARY BUILDING.

All columns are 8WF31, but F-line columns are 8WF24
We check column (F_y - 13¼) which is 8WF24
Steel column: analyzed 8WF24

between 53'8½" & 35'8"

unbraced height $h = 18'\ ½"$
$\therefore l = 216.5"$

Area $A = 7.06\ in^2$ $I_{xx} = 82.5\ in^4$
$d = 7.93\ in$ $r_{xx} = 3.42\ in$
$r_{yy} = 1.61\ in$

For $k=1$, $\frac{kl}{r_{min}} = \frac{216.5}{1.61} = 134.47$ KSI

$$F_a = \frac{12\pi^2 E}{23(kl/r)^2} = 8.258 \text{ KSI}$$

$r_T = 1.78$ in.

$$\frac{l}{r_T} = \frac{216.5}{1.78} = 121.63 > 119.0\sqrt{C_b} \qquad C_b = 1.0$$

Use $F_b = \frac{12 \times 10^3 C_b}{ld/A_f} = \frac{12 \times 10^3}{216.5(3.07)} = 18.0545$

Calculation of axial load.
Area supported by column = 15'8" x 13'6"

Wt. of Beams	14B22	13'6"	297 lbs
	10WF30	7'5"	222.5 lbs
	8B10	6'	66.0 lbs
Roof Deck (1½")	4.77 psf (assumed)	15'8"x13'6"	1008.855 lbs
			1588.355 lbs

If we add 40 psf live load, axial load = 10048.355 lbs

Title STEEL COLUMNS — PRIMARY AUXILIARY BUILDING

FOR NO LIVE LOAD CASE USE AXIAL LOAD = 2 kips

$$\therefore f_a = \frac{2}{7.06} = 0.2833 \text{ ksi}$$

$$\therefore \frac{f_a}{F_a} = \frac{0.2833}{8.258} = 0.0343 < 0.15$$

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

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

$$= (1 - 0.0343)\, 18.0545$$

$$\therefore f_b = 17.4352 \text{ ksi}$$

$$\therefore M = \frac{f_b I}{y} = \frac{(17.4352)\, 82.5}{7.93/2} = 362.7742 \text{ k-inch}$$

Allowable moment $\therefore M = 30.2312$ k-ft.

Convert moment to allowable wind load $M = \frac{wl^2}{8}$

$$\therefore w = \frac{8 \times 30.2312 \times 1000}{(18.04167)^2} = 743.00511 \text{ lbs/ft.}$$

Supported lateral width = 15'8" $\Rightarrow$ allowable press. = $\frac{743.00511}{15.6667}$

$p = 47.426$ psf.

Increase allowables by 1.6 for tornado loading, pressure = 1.6×47.426 = 75.8814 psf

a) $\therefore$ Allowable differential pressure = 0.527 psi

$\Rightarrow$ corresponding wind speed = $\sqrt{\frac{75.8814}{0.00511}}$ = 121.86 mph

b) $\therefore$ Allowable Tornado dynamic pressure = $\frac{75.8814}{0.8}$ = 94.852 psf
(Use 0.8 shape factor)

$\Rightarrow$ corresponding wind speed = $\sqrt{\frac{94.852}{0.00256}}$ = 192.5 mph

Title: ROOF STEEL - PRIMARY AUXILIARY BUILDING.

Roof Beam 14B22

Between column line 13'/4 & 12

Area supported = 27' x 6'7"

Unbraced length = 27'

Area $A = 6.49\ in^2$ $\quad I_{xx} = 198\ in^4$ $\quad r_{xx} = 5.53\ in.$

$d = 13.72\ in$ $\quad I_{yy} = 7.00\ in^2$ $\quad r_{yy} = 1.04\ in.$

$r_T = 1.26\ in.$

For $k=1$, $\dfrac{kL}{r_{min}} = \dfrac{27 \times 12}{1.04} = 311.54 > C_c = 126.1$ For $F_y = 36$ ksi

$$\frac{l}{r_T} = \frac{324}{1.26} = 257.14 > 119\sqrt{C_b} \quad \text{FOR } C_b = 1.0$$

$$F_b = \frac{170 \times 10^3 C_b}{(l/r_T)^2} = \frac{170 \times 10^3}{(257.14)^2} = 2.57 \text{ ksi}$$

$$F_b = \frac{12 \times 10^3 C_b}{l\, d/A_f} = \frac{12 \times 10^3}{27 \times 12 \times 8.19} = 4.52 \text{ ksi}$$

Use $F_b = 4.52$ ksi

$$F_b = \frac{My}{I} \Rightarrow M = \frac{F_b I}{y} = \frac{(4.52)(198)}{(13.72/2)} = 130.46 \text{ k-in.}$$

$$= 10.87 \text{ k-ft.}$$

For $M = 17.27$ k-ft.

$$W = \frac{8M}{L^2} = \frac{8(10.87)}{(27)^2} = .12 \text{ k/ft}$$

$$= 120 \text{ lb/ft.}$$

$$\therefore \text{pressure} = \frac{120.0}{6'7"} = \frac{120.}{(6.583)} = 18.23 \text{ psf}$$

Increase the steel allowable by 1.6 for tornado loadings.

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Title ROOF STEEL — PRIMARY AUXILIARY BUILDING

Roof Beam 12 WF 27

I 27.6 K between 11¼ & 10¼ along H. column line

Supported area = 6' 6½" × 23'

Unbraced length = 23'

Area $A = 7.95\ in^2$ $\quad I_{xx} = 204\ in^4$ $\quad r_{xy} = 5.07$ in.
$d = 11.96$ in $\quad I_{yy} = 18.3\ in^4$ $\quad r_{yy} = 1.52$ in.
$r_T = 1.74$ in.

$$\frac{l}{r_T} = \frac{23 \times 12}{1.74} = 158.62 > 119\sqrt{C_b} \quad \text{for } C_b = 1.0$$

Use $F_b = \dfrac{12 \times 10^3 C_b}{l\, d/A_f} = \dfrac{12 \times 10^3}{23 \times 12 \times 4.60} = 9.45$ ksi

only bending case

$$F_b = \frac{My}{I}$$

$$\therefore M = \frac{F_b I}{y} = \frac{(9.45)(204)}{(11.96/2)} = 322.4 \text{ K-in.}$$
$$= 26.87 \text{ K-ft.}$$

$$W = \frac{8M}{l^2} = \frac{8(26.87)}{(23)^2} = .406 \text{ K-ft.}$$

$$\therefore \text{pressure} = \frac{406}{6.5417} = 62.06 \text{ psf.}$$

For beam failing in uplift pressure, add the steel decking self wt. to get allowable pressure
Assume the steel deck weighs about 4.77 psf.

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Title: ROOF STEEL – PRIMARY AUXILIARY BUILDING.

Roof Beam 14B22 cont'd.

For uplift pressure add steel decking wt. about 4.77 psf

$\therefore$ Allowable pressure $= 1.6(18.23) + (4.77) = 33.9$ psf.

a) $\therefore$ Allowable differential pressure $= 0.235$ psi

$\Rightarrow$ corresponding wind speed $= \sqrt{\frac{33.9}{0.00511}} = \underline{81.4 \text{ mph}}$

b) $\therefore$ Allowable Tornado dynamic pressure $= \frac{33.9}{0.7} = 48.4$ psf
(Use 0.7 shape factor)

$\Rightarrow$ correspondin wind speed $= \sqrt{\frac{48.4}{0.00256}} = \underline{138 \text{ mph}}$

c) Allowable High wind dynamic pr $= \frac{18.23+4.77}{0.7} = 32.86$ psf
(Table 5, ANSI A58.1)

$\Rightarrow$ wind speed $= \underline{97.39 \text{ mph}}$

Calculation of allowable wind speeds for Beam 12WF27

For uplift pressure add steel decking wt. about 4.77 psf

Allowable pressure $= 1.6(62.06) + (4.77) = 104$ psf

a) $\therefore$ Allowable differential pressure $= .723$ psi

$\Rightarrow$ corresponding wind speed $= \sqrt{\frac{104}{0.00511}}$ $\underline{143 \text{ mph}}$

b) $\therefore$ Allowable Tornado dynamic pressure $= \frac{104}{0.7}$
(Use 0.7 shape factor)

$= 149$ psf

$\Rightarrow$ corresponding wind speed $= \sqrt{\frac{149}{0.00256}} = \underline{241 \text{ mph}}$

APPENDIX B

DIESEL GENERATOR ANNEX DESIGN REVIEW CALCULATIONS

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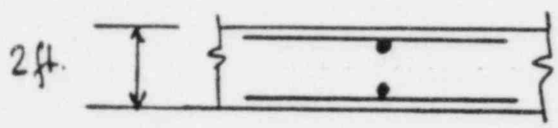
Title: DIESEL GENERATOR BLDG – CONCRETE ROOF SLAB.

THE CONCRETE ROOF SLAB ON THE NEW GENERATOR BLDG. IS 2FT THICK.



ALL REINFORCING BARS ARE #11 @ 10"

ASSUME UNIT WT. OF REINFORCED CONCRETE = 150 lb/ft^3

$\therefore$ SELF WT OF 2FT. THICK SLAB = 2×150

= 300 lb/ft^2

UPLIFT PRESSURE DUE TO PRESSURE DROP = 324 lb/ft^2

$\therefore$ NET PRESSURE UPLIFT ON SLAB = 24 psf.

NO FURTHER ANALYSIS NEEDED AS SLAB ALREADY DESIGNED FOR SELF WT OF 300psf AND REINFORCEMENT IS SAME IN BOTH TOP AND BOTTOM FACE.

THE CONCRETE WALLS ARE 2 ft. THICK.

REINFORCEMENT IS #11 BARS @ 10" ON BOTH FACES, VERTICAL AND HORIZONTAL.

FOR PRESSURE DROP CASE, LOW PRESSURE IN BLDG.

$\therefore$ PRESSURE ACTING ON WALL = 324 psf.

WALL CAN BE ANALYZED AS TWO-WAY FLAT PLATES TRANSFERING MOMENT IN VERTICAL AND HORIZONTAL DIRECTION

ON 10" WIDE SECTION MOMENT IN LATERAL DIRECTION WIDTH OF 57'9" WILL BE CRITICAL

$$\text{DISTRIBUTED LOAD} = \frac{324}{1000} \times \frac{10}{12} = 0.27 \text{ kcf.}$$

$$\therefore \text{TOTAL STATIC MOMENT } M = \frac{wl^2}{8}$$

$l = 57.75$ ft

$$\therefore M = \frac{0.27(57.75)^2}{8}$$

$$M_{actual} = 112.558 \text{ k-ft}$$

DISTRIBUTE M_{actual} ACCORDING TO ACI 318-77, SECTION 13.6.3
MAX DISTRIBUTION FACTOR WILL BE 0.75

$$\text{MAX. } M_{actual} = 0.75 \times 112.558$$

$$\therefore M_{actual} = 84.42 \text{ k-ft.}$$

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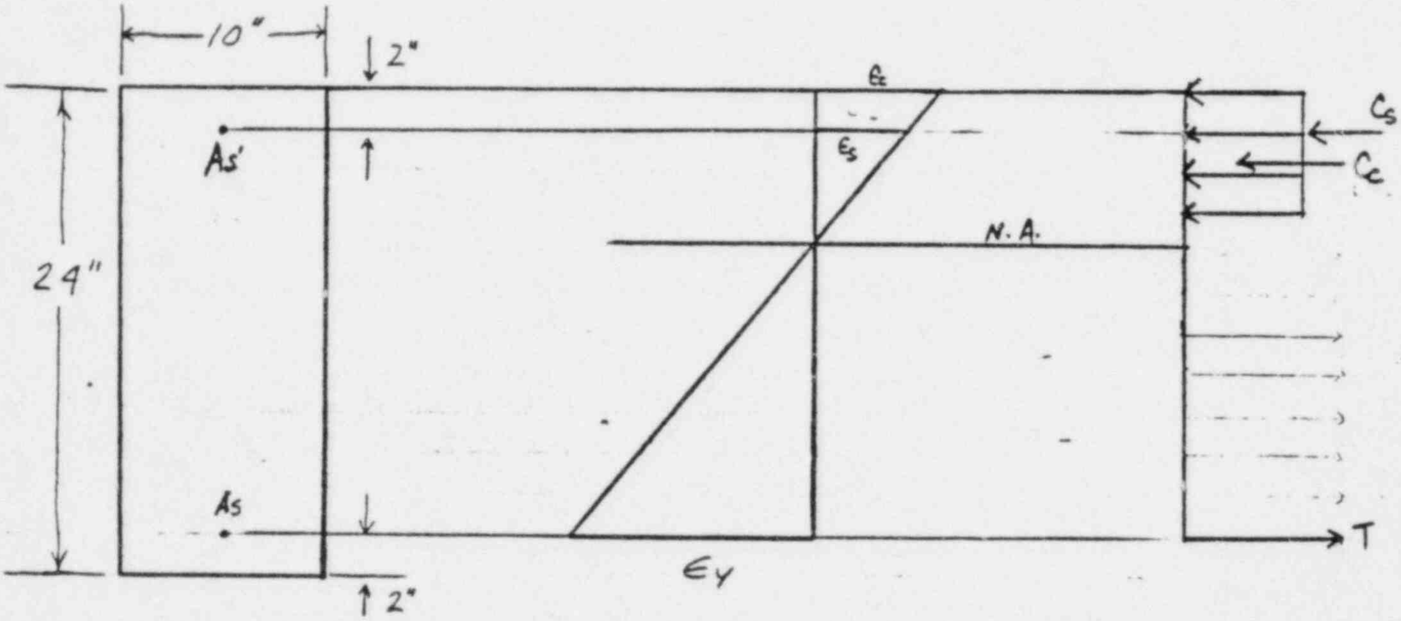
Title: CONCRETE WALL SECTION – DIESEL GENERATOR BLDG.

MOMENT CAPACITY of NORTH + WEST SIDE WALL.

- Reinforcement is #11 BARS @ 10".

- CONSIDER A 10" WIDE SECTION of The WALL.



CONCRETE $f_c' = 3000$ psi
STEEL $f_y = 40{,}000$ psi

11 BARS ONLY.

$A_s = A_s' = 1.56\ in^2$ AREA of #11 BAR

TENSION

$T = A_s f_y$
$= 1.56 \times 40$
$T = 62.4$ KIPS

COMPRESSION

$$C_c = 0.85 f_c' (0.85x) b$$
$$= 0.85 \times 3.0 \times (0.85x) \times 10.0$$
$$C_c = 21.675x \text{ Kips}$$

FROM STRAIN GEOMETRY,

$$\epsilon_s = \frac{0.003}{x}(x-2) \Longrightarrow f_s = \frac{87(x-2)}{x}$$

$$C_s = A_s' \left[\frac{87(x-2)}{x} - 2.55 \right]$$

$$= 1.56 \left[\frac{87(x-2)}{x} - 2.55 \right]$$

$$\therefore C_s = 135.72 \frac{(x-2)}{x} - 3.978$$

FOR EQUILIBRIUM,

$$T = C_s + C_c$$

$$\therefore 62.4 = 21.675x + 135.72 \frac{(x-2)}{x} - 3.978$$

$$66.378 = 21.675x + 135.72 \frac{(x-2)}{x}$$

$$66.378x = 21.675x^2 + 135.72x - 271.44$$

$$21.675x^2 + 69.342x - 271.44 = 0$$

Title: CONCRETE WALL SECTION - DIESEL GENERATOR BLDG.

$$x = \frac{-69.342 \pm \sqrt{(69.342)^2 + 4(21.675)(271.44)}}{2(21.675)}$$

$$= \frac{-69.342 \pm 168.3513}{43.35}$$

$$x = 2.2839516''$$

$$C_c = 21.675x$$

$$\therefore \quad C_c = 49.504651 \text{ Kips}$$

$$\therefore \quad f_s = \frac{.87(x-2)}{x} = 10.81629 \text{ Ksi}$$

$$C_s = A_s'(f_s - 0.85 f_c')$$
$$= 1.56(10.81629 - 2.55)$$

$$\therefore \; C_s = 12.895349 \text{ Kips}$$

Therefore Moment Capacity,

$$M = C_c(d - a/2) + C_s(20)$$

$$= 49.504651\left(22 - \frac{1.9613589}{2}\right) + 12.895349(20)$$

$$M = 1041.0492 + 257.90698 \quad \text{K-in.}$$

$$M = 108.24635 \text{ K-ft.}$$

$$\therefore \quad M_{ALLOW} = 108.25 \text{ K-ft.} > M_{ACTUAL} = 84.42 \text{ K-ft}$$

$\therefore$ WALL O.K.

APPENDIX C

CONTROL ROOM DESIGN REVIEW CALCULATIONS

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Title: CONTROL ROOM — HADDAM NECK UNIT 1

CONTROL ROOM LOCATED ABOVE OPERATING FLOOR ELEV 59' 6"

REINFORCED CONCRETE ROOF SLAB 22" THICK AT ELEV 77' 9 1/16"

CONTROL ROOM ROOF DESIGNED FOR 40 psf LIVE LOAD

NORTH & EAST SIDE REINFORCED CONCRETE WALLS ARE 20" THICK

SOUTH & WEST SIDE REINFORCED CONCRETE WALLS ARE 16" THICK

THE CONTROL ROOM IS SUPPORTED ON STRUCTURAL STEEL FRAME.

WEST SIDE WALL IS COMMON WITH TURBINE BUILDING

OTHER SIDES EAST, NORTH & SOUTH ARE EXPOSED.

SOUTH SIDE CONC. WALL BETWEEN COL. LINES 9 & 10 WILL BE EXPOSED IF SOUTH BLOCK WALL FAILS IN AN EVENT OF A TORNADO.

FOR PRESSURE DROP OF 2.25 psi UPLIFT ON ROOF = 324 psf

DEAD WT. DOWN OF ROOF SLAB 22" THICK = 275 psf

NET UPLIFT = 49 psf.

DESIGN WALL AS BEAM SECTION IN BENDING ONLY

REINFORCEMENT IS #5 BARS @ 12" EACH WAY ON EACH FACE.

STEEL IS $f_y = 40$ ksi

CONCRETE IS $f_c' = 3$ ksi

CONSIDER ONE FT. WIDE SECTION OF 16" REINF. CONC. WALL

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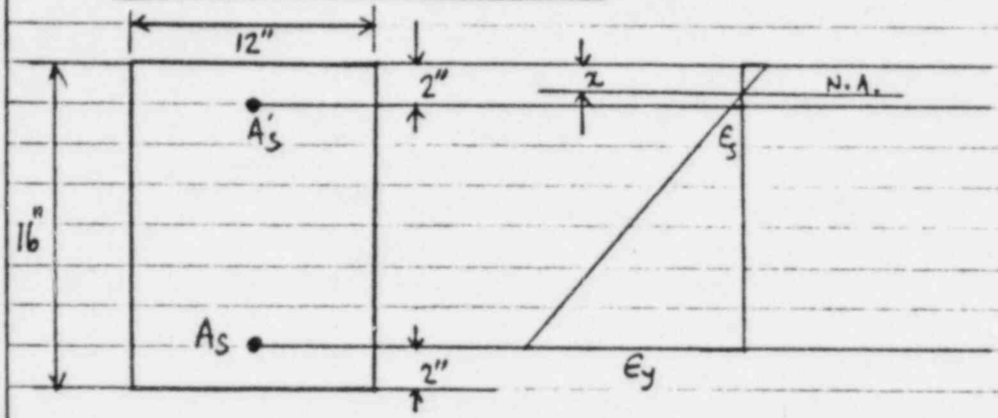
Title CONTROL ROOM - SOUTH SIDE REINF. CONC. WALL SECTION

1FT. WIDE SECTION



AREA OF STEEL $A_s = A_s' = 0.31 in^2$ (#5 BAR)

TENSION $T = A_s f_y$

$= 0.31 \times 40.$

$\therefore T = 12.4$ kips

ASSUME UPPER STEEL IS ALSO IN TENSION

$$\epsilon_s = \frac{\epsilon_y (2-x)}{(14-x)} \Rightarrow f_s = \frac{40(2-x)}{(14-x)}$$

$\therefore T' = A_s' f_s$

$$= 0.31 \times \frac{40(2-x)}{(14-x)}$$

$$\therefore T' = \frac{24.8 - 12.4x}{(14-x)}$$

COMPRESSION $C_c = 0.85 f_c' (0.85x) b$

$= 0.85 \times 3 (0.85x) 12$

$\therefore C_c = 26.01x$ kips

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Title: CONTROL ROOM - SOUTH SIDE REINF. CONC. WALL SECTION

FOR EQUILIBRIUM $C_c = T + T'$

$$\therefore 26.01x = 12.4 + \frac{24.8 - 12.4x}{(14-x)}$$

$$\Rightarrow 26.01x^2 - 388.94x + 198.4 = 0$$

$$\Rightarrow x = 0.5288 \text{ in.}$$

$$\therefore C_c = 26.01x$$
$$= 26.01 \times 0.5288$$
$$C_c = 13.754 \text{ kips}$$

$$T' = \frac{24.8 - 12.4x}{(14-x)}$$

$$\therefore T' = 1.3542 \text{ kips}$$

NOW FIND THE MOMENT CAPACITY OF THE SECTION

$$M = T(d - \tfrac{a}{2}) + T'(2 - a/2)$$

$a = 0.85x$
$= 0.4495$ in

$d = 16 - 2$
$= 14$ in.

$$\therefore M = 12.4\left(14 - \frac{0.4495}{2}\right) + 1.3542\left(2 - \frac{0.4495}{2}\right)$$
$$= 173.2173 \text{ K-inch}$$

$\therefore$ Moment Capacity $M_{allow} = 14.435$ K-ft

NOW WE CALCULATE ACTUAL MOMENT FOR PRESSURE DROP CASE 324 psf

Title CONTROL ROOM – SOUTH SIDE REINF. CONC. WALL SECTION

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

height of wall $l = 18.3$ ft.

load $w = 0.324$ ksf.

$$\therefore M = \frac{(0.324)(18.3)^2}{8}$$

$$M_{actual} = 13.5631 \text{ K-ft.}$$

$\therefore \; M_{actual} < M_{allowable}$

$\therefore$ WALL IS OK

Title CONTROL ROOM — STRUCTURAL STEEL FRAMING

THE CONTROL ROOM EXPOSED ON NORTH, SOUTH AND EAST SIDE. FOR WIND FORCES FROM NORTH OR SOUTH BRACING ALONG COLUMN LINES C AND F CARRY ALL THE LOAD TO THE FOUNDATION. C COL. LINE COMMON WITH TURBINE BUILDING. ASSUME FOR WIND FROM NORTH SIDE ALL LOAD BETWEEN COL. LINE D & F CARRIED BY F LINE BRACING SYSTEM. LOAD BETWEEN C & D CARRIED BY C LINE BRACING SYSTEM

Calculation of total Load

ON SOUTH SIDE WALL EXPOSED ABOVE SERVICE BUILDING ROOF ELEV. 41 ft.

∴ EXPOSED HEIGHT = 77.75 − 41 = 36.75 ft.

WIDTH OF EXPOSED PART OF WALL WILL BE BETWEEN COL. LINE D & F. (LOAD BETWEEN COL. LINE C & D WIDTH = 27 ft. CAN BE TAKEN BY C LINE)

WIDTH BETWEEN COL. LINE D & F = 60 ft.

EXPOSED AREA = 60 × 36.75 = 2205 ft^2

FOR 300 MPH, DYNAMIC TORNADO PRESSURE p = 230.4 psf

FOR WINDWARD FACE USE 0.8p & LEEWARD FACE USE 0.5p

∴ TOTAL PRESSURE = 0.8p + 0.5p = 299.52 psf.

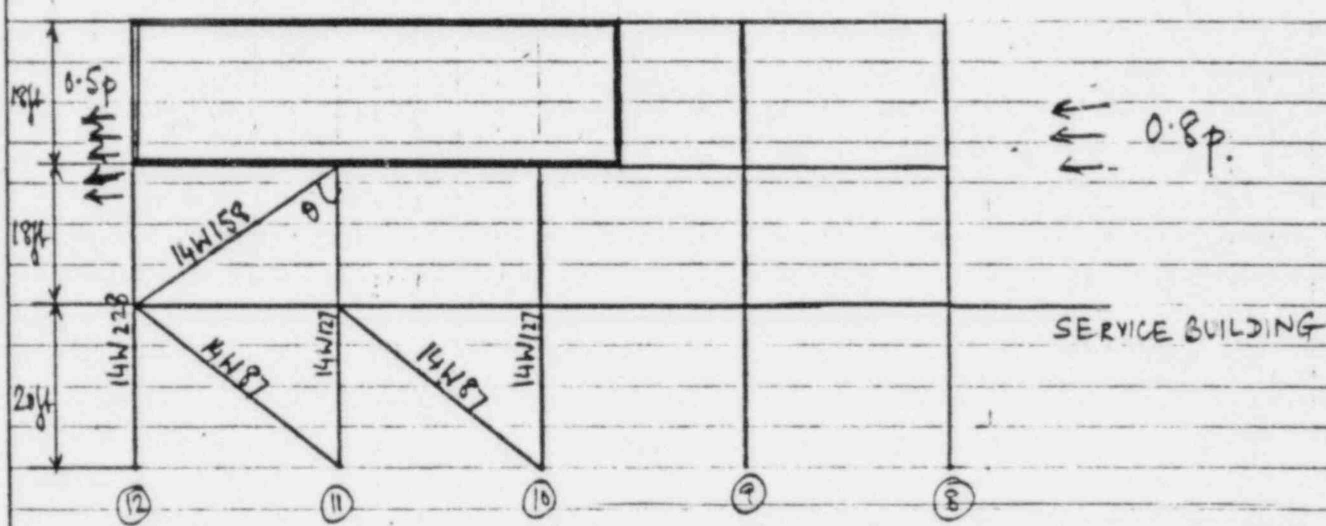
$$\therefore \text{ TOTAL LOAD } = \frac{299.52 \times 2205}{1000}$$

$$P_u = 660.442 \text{ kips}$$

<u>Elevation of F-line</u>

N ←

FIND CAPACITY FOR WIND COMING FROM SOUTH



4 Bays @ 25ft each

Steel Use A36 $F_y = 36$ ksi

Unbraced length of 14W158 $= \sqrt{25^2 + 18^2} \simeq 31$ ft.

Allowable axial load in compression for 31ft effective length 14W158 (AISC Manual table (pg 3-14)) C = 644 kips

Capacity of 14W158 in Lateral direction will be $C \cos\theta$

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Title: CONTROL ROOM – STRUCTURAL STEEL FRAMING

$\cos\theta = \frac{25}{31}$ $\sin\theta = \frac{18}{31}$

∴ Capacity of 14W158 in lateral direction $C_H = 644 \times \frac{25}{31}$
$= 519.355$ kips

For Tornado dynamic pressure increase allowable by 1.6

∴ $C_H = 1.6 \times 519.355 = 831$ kips

$C_H = 831$ kips $> P_H = 660.442$ kips load Due to Wind

ADDED CAPACITY IN BRACING FOR 2 14W87 MORE.

∴ ENOUGH RESERVE STRENGHT FOR NORTH & SOUTH WIND.

FOR WIND FROM EAST SIMILAR ANALYSIS INDICATES ENOUGH CAPACITY OF THE BRACING SYSTEM.

AXIAL LOAD IN COLUMN DUE TO BRACING REACTION.

$C_V = 644 \times \frac{18}{31} = 374$ kips

CAPACITY FOR AXIAL LOAD OF 14W127 COLUMN FOR 20FT. EFFECTIVE LENGTH (AISC MANUAL TABLE pg 3-15) = 636 kips

∴ COLUMN ALSO O.K.

FORM CS-FRC-81