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September 1, 1982

Docket No. 50-219
LS05-82-09-005

Mr. P. B. Fiedler
Vice President and Director - Oyster Creek
Oyster Creek Nuclear Generating Station
Post Office Box 388
Forked River, New Jersey 08731

Dear Mr. Fiedler:

SUBJECT: SEP TOPIC III-2, WIND AND TORNADO LOADINGS
OYSTER CREEK NUCLEAR GENERATING STATION

Enclosed is our final evaluation of SEP Topic III-2. This evaluation compares your facility as described in the Safety Analysis Report you supplied on May 7, 1981, and other information on Docket No. 50-219 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(51)

Enclosure:
As stated

cc w/enclosure:
See next page

ADD:
R. Fell

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Mr. P. B. Fiedler

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SYSTEMATIC EVALUATION PROGRAM

TOPIC III-2

OYSTER CREEK

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 Oyster Creek 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 Oyster Creek site are a 250 mph windspeed (corresponding to 160 psf dynamic pressure) and a 1.5 psi (216 psf) differential pressure. The evaluation and conclusions are based on a Safety Analysis Report supplied by the licensee, information available on Docket No. 50-219, 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 Oyster Creek facility. The report identifies limiting structural elements and their associated windspeed. 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 supplied by the licensee on May 7, 1981, structures at the site were designed for windspeed of 100 mph from 0-50 feet above grade and 125 mph from 50-150 feet above grade. This corresponds to a building pressure of 40.3 psf and 62.8 psf respectively as given in response dated 12/19/67 to staff questions. These values are total applied building load which include a gust factor of 1.1 and a shape factor of 1.3 (.8 windward + .5 leeward face). Excluding shape factors and back calculating from the values given results in an upstream pressure of 31 psf below 50 feet above grade and 48 psf from 50-150 feet above grade. Although not specifically stated, it is assumed that 31 psf and 48 psf were used in the design of the wall panels since this would be in agreement with normal wind design procedures. Allowable stresses were increased by 1/3 for load combinations of dead load, live load, and wind load. The load combination involving wind was dead load plus live load plus wind load.

Although the facility has not been designed for tornado loads the licensee has given maximum permissible wind velocity and depressurization loads based on maintaining stresses less than 90% of yield for reinforcing steel and 85% of the ultimate concrete strength and including dead loads plus normal operating loads. These values are given below.

Structure	Wind (mph)	Pressure (psi)
Reactor building - exterior conrete walls	300	2.0
Reactor building - insulated metal siding	160	0.53
Reactor building - roof decking	280	0.68
Reactor building steel for craneway enclosure	190*	0.68
Control room - north wall	160	0.53
remainder	300	2.0
Intake structure	300	2.0
Ventilation stack	180	2.0
Diesel generator and oil tank vaults**	300	2.0

* Based on siding drag - without siding, steelwork can withstand 300 mph.

** Based on other information provided by the licensee dated April 30, 1982, the licensee states that the north and south walls of the diesel generator building are capable of withstanding a 240 mph tornado wind; the east and west walls can withstand 168 mph; and the roof can withstand 88 mph. These values include the effect of tornado missile loading. The values will be higher for wind loading considered separately. No values are given for differential pressure.

The SAR also states that the outdoor service water pumps and startup transformer are capable of withstanding 200 mph winds and 2 psi pressure drop. The licensee also concludes that the likelihood of damage to the spent fuel pool in the pool area due to tornado effects is small.

The ventilation stack was designed per ACI-505 with a design wind gust velocity of 110 mph at the base and increasing with height in accordance with the relation used in ACI-505.

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 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 Oyster Creek were originally designed for 100 mph winds (40.3 psf total load) from 0-50' and 125 mph (63 psf total load) from 51-150' above grade. ANSI A58.1 specifies a 10^{-2} wind of approximately 103 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 methods 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. The magnitude of the straight wind loads, excluding localized effects, used in the original design is comparable to that required by current criteria. The ANSI A58.1 code requires higher localized forces below 50' above grade and above 75' above grade than used in the original design; however, using the basic 10^{-2} windspeed identified in Topic II-2.A (78 mph) and using ANSI A58.1 rules for developing forces at various heights results in the original design exceeding these forces for both local and global forces at all elevations. The 1/3 increase in allowable stress 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). Additionally, decreasing the original design loads such that the 1/3 increase in stress is not allowed results in a load higher than the load applied from the results of SEP Topic II-2.A for a 10^{-2} wind. This load would be higher for all elevations globally and almost all elevations locally. Since it is uncertain whether pipe reaction 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 loads and would, therefore, provide margin to accommodate pipe reaction loads and snow loads with the exception of some roofs. Roof members designed to carry axial loads as a result of wind will have their axial load carrying capacity significantly reduced if vertical loads that induce bending are present. It should be noted that straight wind design criteria relied upon is that presented by the licensee in their Safety Analysis Report and Attachment B to the licensee response to staff questions dated December, 1979 for buildings and Amendment 22 to the FDSAR for the vent stack. The original design straight wind loads applied to the stack are comparable to the requirements of ANSI A58.1, 1972 for a basic windspeed of 103 mph and are in excess of that required based on the site specific windspeed of 78 mph from SEP Topic II-2.A. The staff determined capacity of 164 mph uniformly distributed with height would result in a higher applied load at any section than that obtained from ANSI A58.1 for a 103 mph straight wind at 30' above grade.

The staff has analyzed the reactor building and vent stack to determine its ability to resist the design tornado loads. Although the vent stack was not designed for tornado loads and it is currently not a Category 1 structure from a systems approach, it is a unique structure whose failure can affect Category 1 structures. The results of the analysis are shown below in terms of limiting windspeeds. For elements found to have low tornado wind resistance, the element was examined for straight wind design per the requirements of ANSI A58.1. The wind capacity then reported is based on stress limits for wind design with no allowable stress increase; therefore, there would be additional capacity than that shown:

TABLE 1

Structure	Element*	Cause of** Failure	Wind speed (mph)	Corresponding Pressure (psf)
Reactor building	Roof beams	3	68	17
		2	61	19
		1	102	27
	Columns above operating floor	2	174	154

* The first element identified for each structure is the limiting element. Additional elements that have also been found to be inadequate are subsequently listed. Note that this table does not imply that all inadequate elements have been identified or that entries are listed with respect to the most critical loading combination. 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.

The results indicate that the reactor building below the operating floor is adequate to withstand the design tornado wind and pressure loads. Above the operating floor, structural elements were found to be inadequate to withstand the postulated tornado loads with the limiting elements given in Table 1. The concern above the operating floor is the spent fuel pool since there are no other safety-related systems or components above this level. The failure of structural elements on the spent fuel pool need to be considered.

The ventilation stack was analyzed by 1) working stress methods with ACI code allowables and 2) working stress methods allowing stress in the extreme outer steel to reach yield and extreme fiber concrete to reach .85 f_c'. The results in terms of windspeed (mph) are given below.

Stack	WSD (code allowables)	WSD (f_y, 85 f_c' allowables)
Stack cylinder	138	164

The conclusions reached by the staff agree with values presented by the licensee for the reactor building below the operating floor. Above the operating floor, values for steel capacity obtained by the staff are significantly less than that presented by the licensee. Ventilation stack capacity obtained by the staff is less than that presented by the licensee (164 mph vs. 180 mph).

The staff was unable to perform capacity calculations for other structures due to a lack of information. The staff concludes that there is inadequate justification for some of the conclusions reached by the licensee in the SAR. These are discussed below.

1. The SAR states that "generally, safety-related equipment is enclosed within safety-related structures." The licensee should review components not enclosed within safety-related structures to assure that all such components have been identified and their capacity determined.

2. The licensee has not presented information to support their statement that service water pumps and startup transformer are capable of withstanding 200 mph winds and 2 psi depressurization nor has any information been provided to support capacity values for the intake structure, oil tank vaults, control room and diesel generator building.

3. The licensee has not provided bases to support their conclusion that stack failure upon the reactor building would not impair the ability to safely shutdown the reactor. Failure of the stack on the spent fuel pool or other Category 1 structures or components has not been

addressed. Circumferential stresses in the stack were not analyzed due to lack of information concerning placement of circumferential reinforcing.

4. The licensee has not determined the consequences on the spent fuel pool of superstructure failure.

5. Capacities for exterior masonry walls have not been given. Since no capacities have been given, any exterior masonry walls should be assumed to fail during a tornado and consequences determined.

6. The effect of failure of non-Category 1 structures on Category 1 structures (e.g., turbine building on control building) has not been addressed by the licensee.

7. Roof decks have not been analyzed by the staff due to lack of information. It is expected that such roofs will have inadequate ability to withstand internal pressure loads. The licensee should determine consequences of their failure.

8. Except for the stack, foundation and soil capacities were not investigated by the staff. The licensee should assure that foundation and soil capacities are not limiting for all structures except the stack.

VI. CONCLUSIONS

It is concluded that some structures and portions of others cannot withstand the postulated tornado loadings of 250 mph and 1.5 psi.

In two cases where the licensee results indicated that structural capacity is less than required to resist design tornado loads (i.e., reactor building above operating floor and vent stack), the staff has calculated capacity less than the value presented by the licensee. For the reactor building below the operating floor, the staff agrees with the licensee that design tornado loads can adequately be resisted.

The licensee should either implement modifications for the following structures or demonstrate that the consequences of their failure in the event of tornado loads is acceptable:

1. Reactor building structure above the operating floor.

2. Failure of non-Category 1 structures upon Category 1 structures (e.g., turbine building, vent stack).

3. Safety related equipment not inside qualified structures.

4. Exterior masonry walls.

5. Roof decks on Category 1 structures.

The licensee should provide a description of the methods and sample calculations used to qualify the following:

1. Service water pumps and start-up transformer.

2. Intake structure and oil tanks.

3. Control building.

4. Diesel generator building.

The licensee should assure that foundation and soil capacities are considered in determining limiting capacities for all Category 1 structures and structures whose failure may affect Category 1 structures. The staff has already investigated the stack foundation and found it not limiting.

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

The need for modifications in order for structures, systems and components to adequately resist wind and tornado loads will be determined during the integrated assessment.

TECHNICAL EVALUATION REPORT

WIND AND TORNADO LOADINGS (SEP, III-2)

JERSEY CENTRAL POWER AND LIGHT COMPANY
OYSTER CREEK NUCLEAR GENERATING STATION

NRC DOCKET NO. 50-219

NRC TAC NO. 41606

NRC CONTRACT NO. NRC-03-79-118

FRC PROJECT C5257

FRC ASSIGNMENT 14

FRC TASK 402

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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 Jersey Central Power and Light Company (JCP&L) for the Oyster Creek Nuclear Generating 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 concern of SEP Topic III-2. The SAR should identify the limiting elements of the structural design and specify the loading conditions and threshold wind speeds at which buildings and components fail. As part of Assignment 14, Franklin Research Center (FRC) is assessing the adequacy and accuracy of the SARs. Typical items that are reviewed are the tornado load calculations and combinations, the structural acceptance criteria, and the method of analysis. In order to verify the conclusions on structural strength, an independent tornado analysis on a sample of Class I structures and components is also conducted.

1.3 PLANT-SPECIFIC BACKGROUND

The review of the Oyster Creek SAR was begun in April 1982. Prior to that time, JCP&L responded to NRC requests for information by providing architectural-engineering structural drawings. Additional sources of information were a JCP&L letter on the SEP structural topics [3] and the plant final safety analysis report [4]. The conclusions stated by JCP&L in the SAR are summarized in Table 1. Correspondence with NRC [5] established the reactor building and ventilation stack as the priority review structures.

The original wind loading criteria of the Oyster Creek structural systems were the structural load provisions of the American Standard Association codes that were in effect at the time of plant design. These provisions called for a graded wind load of 30 psf at a 30 ft elevation, to 45 psf at elevations above 100 ft. The structural acceptance criteria permitted stress levels at a 33-1/3% increase over code allowables. The criteria for this review are stated in Section 2 of this report.

Table 1. Summary of Conclusions from Oyster Creek Topic III-2 SAR*

Class I Structures	Wind (mph)	Pressure (psi)
Reactor Building Exterior Concrete Walls	300	2.0
Reactor Building Insulated Metal Siding	160	0.53
Reactor Building Roof Decking	280	0.96
Reactor Building Steel for Craneway Enclosure	190**	0.68
Control Room - North Wall	160	0.53
Remainder	300	2.0
Intake Structure	300	2.0
Ventilation Stack	180	2.0
Battery Room (interior room)		
Diesel Generator and Oil Tank Vaults	300	2.0

*The table lists the various Class I structures with their respective maximum permissible wind velocity and depressurization values. The allowable stresses do not exceed 90% of yield for reinforcing steel and 85% of the ultimate concrete strength and include the combined effect of dead loads plus normal operating loads.

**Based on siding drag - without siding steelwork can withstand 300 mph.

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 Oyster Creek 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 Oyster Creek 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 ventilation stack and the reactor building. These structures are classified seismically as Category I Nuclear Safety Related. The plan of the building arrangement and an isometric drawing of the Oyster Creek site are shown in Figures 1 and 2.

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

Maximum wind speed	250 mph
Maximum pressure drop	1.5 psi
Rate of pressure drop	0.6 psi/sec
Core radius	150 ft.

These characteristics yield a dynamic pressure of 160 psf. For application of this pressure to external flat surfaces of structures, the shape coefficients are 0.80 for windward walls (positive pressure), 0.50 for leeward walls (suction), and 0.70 for roofs (suction). The shape coefficient for the cylindrical ventilation stack is 0.70. Gust factors for tornado loadings are taken as unity.

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

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

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

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

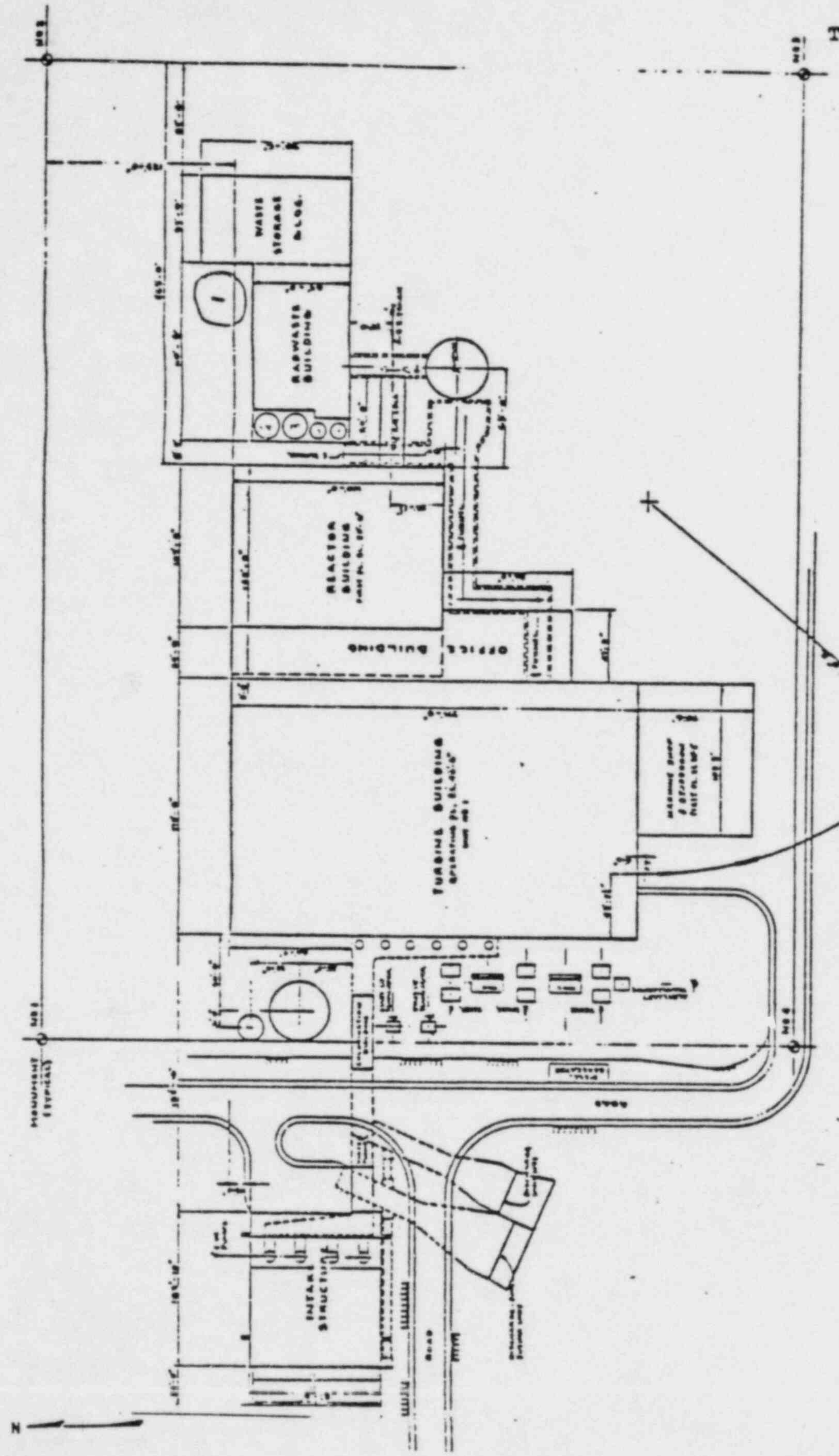

Figure 1. Site Plot Plan

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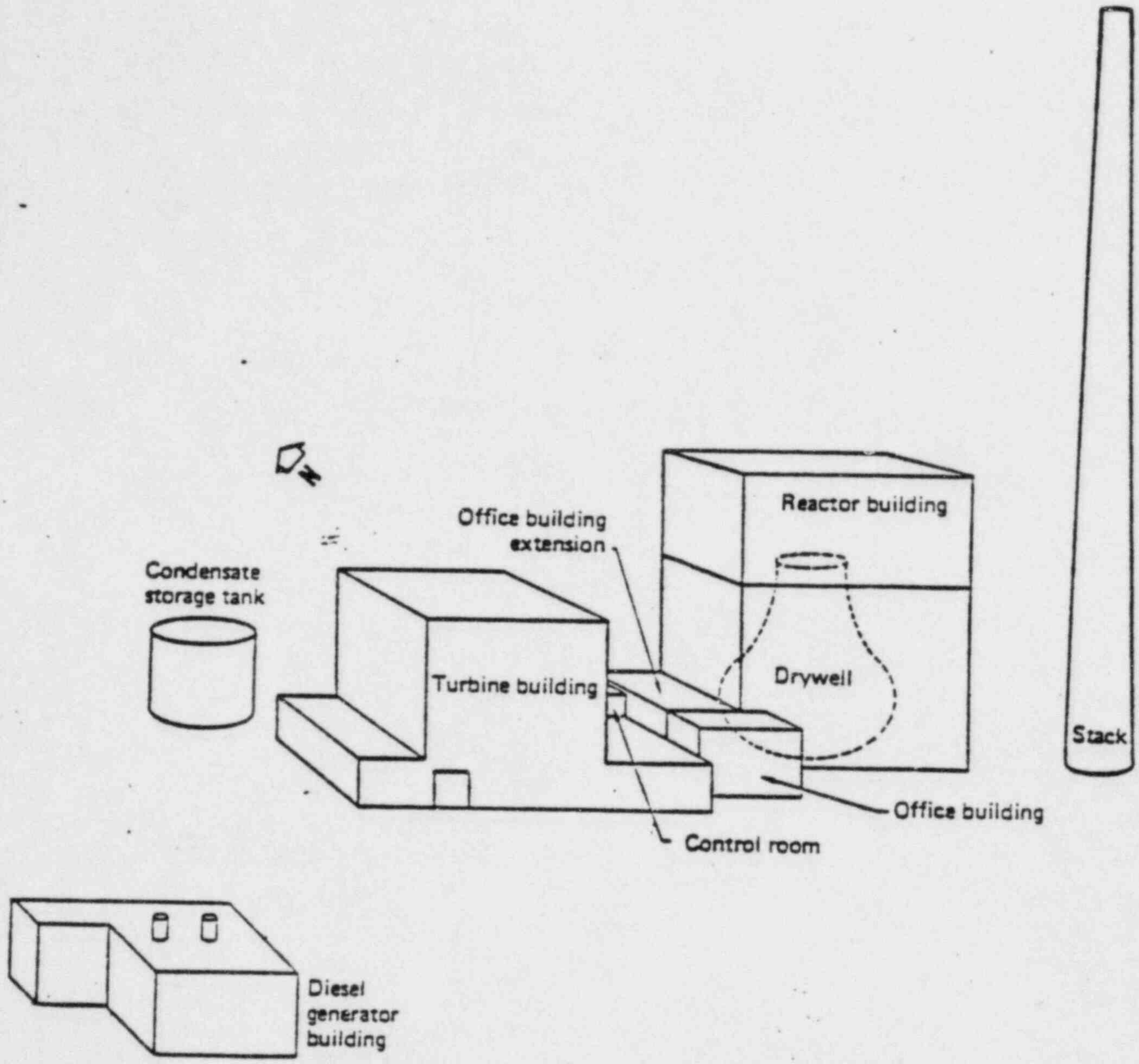

Figure 2. Isometric of Oyster Creek Plant Showing Major Structures

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

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

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

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

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

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

3.2 REACTOR BUILDING

3.2.1 Evaluation

The reactor building is primarily a reinforced concrete structure with the exception of a steel roof deck, which is supported by structural steel framing. The high point of this building is at elevation 169 ft 3 in, while adjacent grade is at elevation 23 ft. The reactor building can be divided into two sections: the main section, which is below the service floor level (elevation 119 ft 3 in), and the service floor enclosure (above elevation 119 ft 3 in).

The exterior of the main section consists of thick reinforced concrete walls framing into the concrete floor slabs, beams, and columns. These elements were examined for the critical load case of differential pressure loads. Each panel is assumed to transfer loads in the horizontal direction to the nearest columns and in the vertical direction to the adjacent thickened floor slabs. The analysis of the panels can be found on pages A-12 through A-14 of Appendix A. Since the vertical section showed adequate reserve moment capacity, the capacity of a horizontal section was not calculated.

The concrete columns of the main section were checked for stability and capacity. The critical members were chosen on the basis of the smallest section and least reinforcement. The members selected were columns A-1 and A-4 (see building plan on page A-2). The dead and live loads on the columns were taken as those values reported in Reference 4. The wind loads acting on the wall panels adjacent to the columns are transferred to the columns as reactions, producing moments in the columns. The columns are taken as supported between successive beam and floor slabs. The column analysis and calculation are given on pages A-1 through A-11.

The service floor enclosure consists of insulated metal wall panels and UK 16-16 Q roof sections supported by a framework of columns and trusses. To analyze the capacity of the roof steel, the steel deck is assumed to remain intact. Also, when examining the capacity of the columns, it is assumed that the metal panels do not fail. The analysis of the structural steel elements of the service floor enclosure are presented on pages A-15 through A-21.

3.2.2 Conclusion

The reinforced concrete elements of the main section can safely withstand the tornado loadings. The limiting elements of the roof steel are the 12B19 beams, which have limit ratings of 0.131 psi (61 mph) for differential pressure, 26.7 psf (102 mph) for tornado dynamic pressure, and 16.8 psf (68 mph) for high wind dynamic pressure. The limiting elements of the roof supports are the north and south side steel columns which have a limit rating of 1.07 psi (174 mph) for differential pressure, but can withstand the full tornado and high wind dynamic pressures.

3.3 VENTILATION STACK

3.3.1 Evaluation

The ventilation stack is an unlined, free-standing axisymmetric reinforced concrete structure. The top of the stack is at elevation 391 ft with grade at elevation 23 ft. The outer diameter at the top is 9 ft 6 in with 6-in-thick concrete. The outer diameter at the top of the foundation (elevation -3 ft) is 31 ft 8-3/4 in with 18-in-thick concrete. For calculation of the diameter of the stack at intermediate sections, it is assumed that the outer surface is smooth. In the event of a tornado strike, the stack can be subjected to pressure as high as 112 psf, corresponding to a 250 mph tornado and 0.7 shape factor.

The stack has been analyzed by a working stress design technique. This method is given in the American Concrete Institute Code (ACI) 307-79 [24]. Two sections, one at 233 ft and the other at 278.1 ft below the top of stack, were found to be the most critical. These sections were reanalyzed by increasing the allowable stresses in reinforcing steel to its yield strength (fy) and in concrete to 0.85 times the compressive strength (f'c). The stack analysis is given on pages B-1 through B-4 in Appendix B.

The section at 338.2 ft below the top of stack has three openings. For the purpose of analysis, the openings were grouped to form one equivalent opening, with an opening angle of $\beta = 49.8°$. Also, the section at 386.5 ft below the top of stack, which is below grade, has two openings. These two openings were also grouped for the purpose of analysis yielding an equivalent opening angle of $\beta = 42.1°$. Additional reinforcement is provided around each opening. Analysis shows that even under the conservative assumption of grouping the openings together, these sections are not the most critical.

The foundation pedestal is hexagonal in shape with each side measuring 18 ft 7-1/2 in. The pedestal is a reinforced concrete block 7 ft thick, and rests on a soil with a bearing capacity of 13 ksf. To determine the resistance to overturning moment, the axial stress due to the dead weight of the stack and concrete pedestal, plus the flexural stresses due to the wind forces, were balanced against the soil bearing capacity. These calculations are shown on pages B-5 to B-6.

3.3.2 Conclusion

It was found that the stack cannot withstand the full tornado dynamic pressure. Using the allowable stresses in reinforcement and concrete, according to ACI 307-79 [24], it was found that the sections at 233 ft and 278.1 ft below the top of stack were limiting and can withstand a tornado dynamic pressure of only 48.8 psf at a 138 mph wind speed. The section at 278.1 ft below the top of the stack was found to be the limiting section when the code allowable stress was increased to 0.85 f'c in concrete and yield stress in reinforcing steel. For these increased stress levels, this section can withstand tornado dynamic pressure of 68.6 psf at a 164 mph wind speed.

The equilibrium model for the stack foundation shows that the foundation was not the limiting design component. The stack foundation can resist an overturning movement due to a dynamic pressure of 93.7 psf at a 191 mph wind speed.

4. CONCLUSIONS

The results of the tornado structural analysis for the reactor building and ventilation stack are summarized in Table 2.

Table 2. Strength Summary of the Structural Components Analyzed

Structure	Element*	Cause of Failure**	Wind Speed (mph)
Reactor Building	12B19 Roof Beams	3	68
		2	61
		1	102
	North and South Side Steel Columns	2	174
	Concrete Walls and Columns	-	-
Ventilation Stack***	Concrete Shaft (When analyzed by ACI 307-79 allowables)	1	138
	Concrete Shaft (When analyzed with increase in allowable stresses)	1	164
	Concrete Foundation	1	191

*The first element identified for each structure is the limiting element. Additional elements that have been found to be inadequate are subsequently listed. Note that this table does not imply that all inadequate elements have been identified or that entries are listed with respect to the most critical loading combination. 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.

***The resistance of the stack to circumferential stresses cannot be determined because the placement of the hoop reinforcement is not known.

It is pointed out that the roof deck itself was not analyzed but that the underlying structural support, the 12B19 roof beams, were included in this study. Also, while the wall panels were not analyzed, the panel fasteners, girts, and panels of a comparable siding system reviewed in a previous study [25] were found to have limited tornado loading resistance (limiting element failed at 48 mph wind speed for tornado differential pressure).

While not specifically reviewed, an additional area of concern is the control room, which is located on the east side of the turbine building (see Figure 2). The north and south walls of the control room are exposed to the atmosphere. The south wall and part of the north wall are constructed of reinforced concrete block, a structural component which typically has limited tornado resistance. The common wall (west) between the turbine building and the control room is also constructed of concrete block and has glass panels that will be subjected to differential pressure loadings with failure of the skin of the turbine building.

A comparison of Table 1 with Table 2 and the above comments shows that for other than the concrete structures, the strengths claimed for structural components in the Oyster Creek SAR are in conflict with and greater than the strengths found by the analysis presented herein.

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APPENDIX A

REACTOR BUILDING DESIGN REVIEW CALCULATIONS

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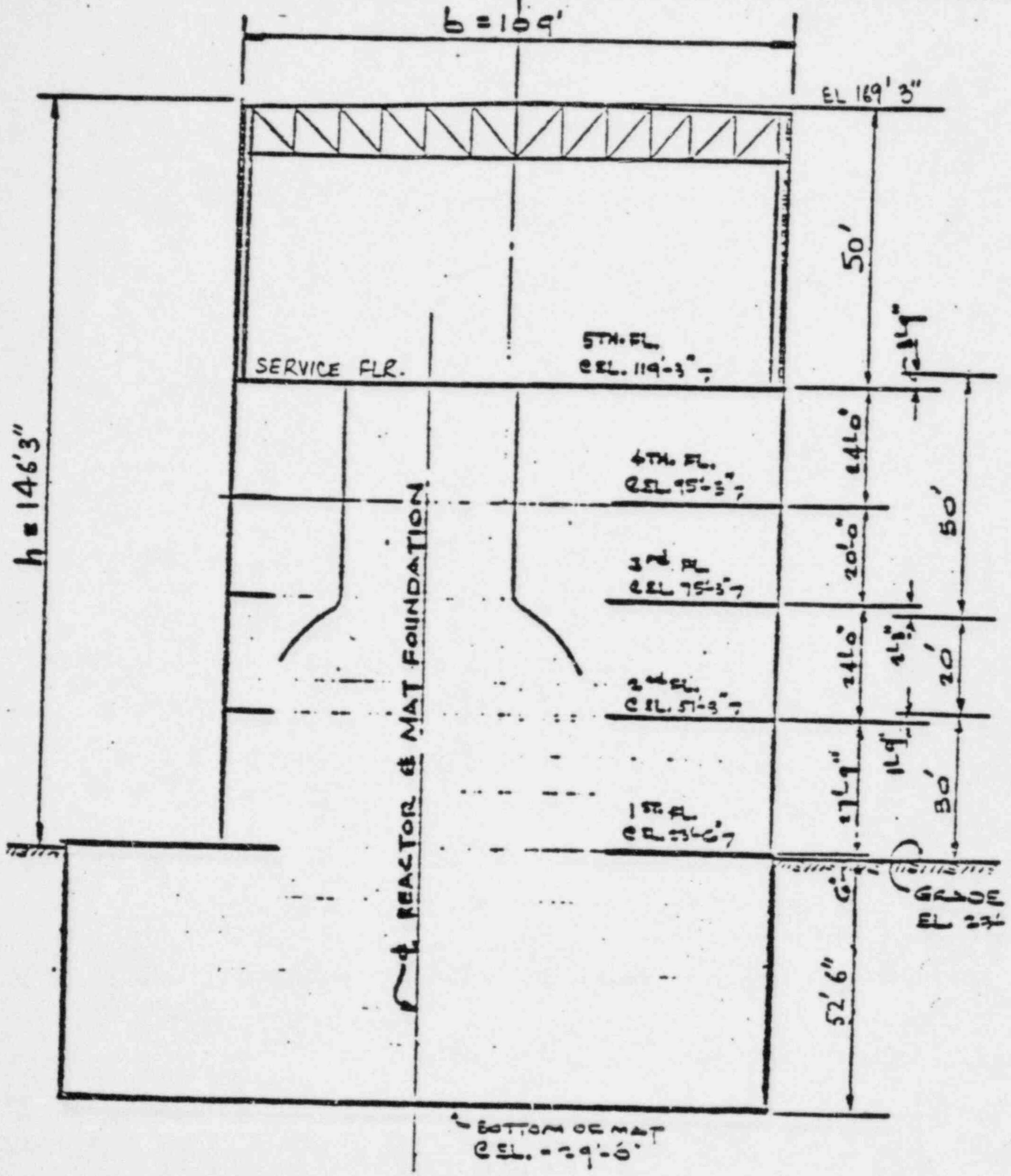
Project	02G - C5257- 01			Page	A-1
By RA	Date APRIL '82	Ch'k'd CB	Date 5/5	Rev.	Date

Title: REACTOR BUILDING – OYSTER CREEK CROSS-SECTION



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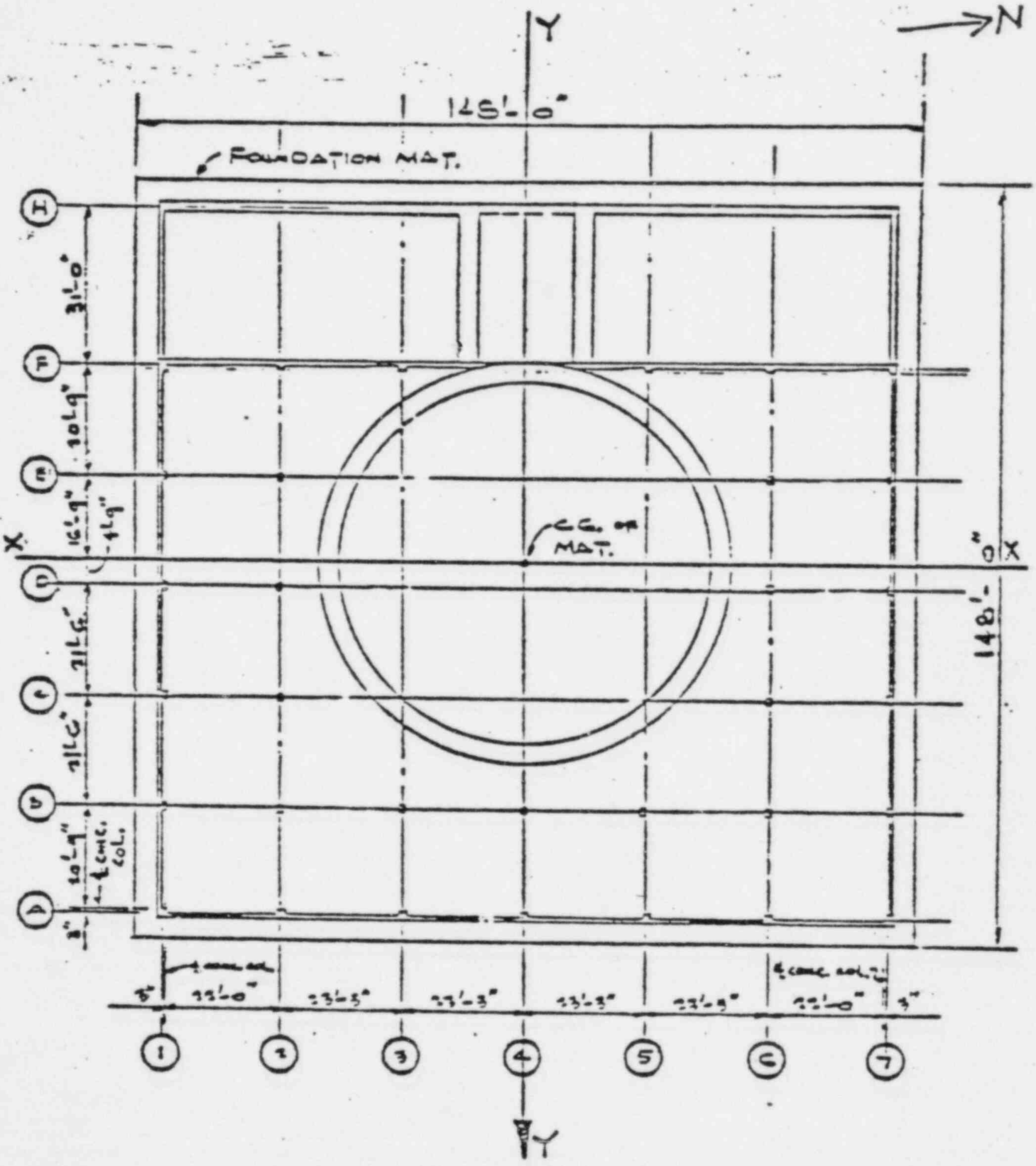
Project 02G - C5257-01		Page A-2
By RA Date APRIL '82	Chk'd CB Date 5/5	Rev. Date

Title REACTOR BUILDING OYSTER CREEK — COL. LINE PLAN



PLAN

FORM CS-FRC-81

THE CRITICAL CASE WILL BE THE PRESSURE DROP CASE. A PRESSURE OF 216 PSF WILL ACT ON CONCRETE WALLS AND COLUMNS.

WE ANALYZE THE COLUMNS A-1 AND A-4 ASSUMING ALL LOAD IN LATERAL DIRECTION IS TRANSFERRED ONTO THE COLUMNS. WE ALSO CHECK THE EAST WALL SECTION FOR TRANSFER OF MOMENT IN VERTICAL DIRECTION.

COLUMN A-1 AND A-4 ARE ASSUMED TO BE SIMPLY SUPPORTED BETWEEN TWO FLOORS. IT IS ASSUMED THAT ALL SHEAR IS TRANSFERRED THROUGH FLOOR SLABS AND OTHER CONCRETE CONSTRUCTION.

CONSIDER 1FT. WIDE STRIP OF THE WALL IN VERTICAL DIRECTION

$l = 24'$ Between 4^{th} & 5^{th} Floor

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

$w = 216 \text{ psf} \times 1 \text{ ft}$
$= 216$

$$= \frac{216}{1000} \times \frac{(24)^2}{8}$$

$$\therefore M_{W45} = 15.552 \text{ K-ft}$$

$l = 27'9"$ Between 1^{st} & 2^{nd} Floor

$$M = 0.216 \frac{(27.75)^2}{8}$$

$$\therefore M_{W12} = 20.792 \text{ K-ft.}$$

WE ALSO ASSUME THAT ALL FLOOR LOADS ARE TRANSFERRED AS AXIAL LOADS ON THE COLUMNS.

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	By RA Date APRIL '82	Ch'k'd LB Date 5/5	Rev. Date

Title REACTOR BUILDING - REINF. CONC. - BELOW SERVICE FL. ELEV 119' 3"

NOW FIND THE ACTUAL MOMENTS ON THE COLUMNS

THE COLUMNS ARE SPACED 23' 3" APART ON EAST SIDE OF THE REACTOR BLDG.

FOR COLUMN A-4 $W = 216 \times 23.25$

$= 5022$ lbs/ft

$w = 5.022$ k/ft.

$l = 27' 9"$ Between 1^{st} & 2^{nd} Flr.

$$\therefore M_{c12} = \frac{5.022(27.75)^2}{8}$$

$$M_{c12} = 483.407 \text{ k-ft}$$

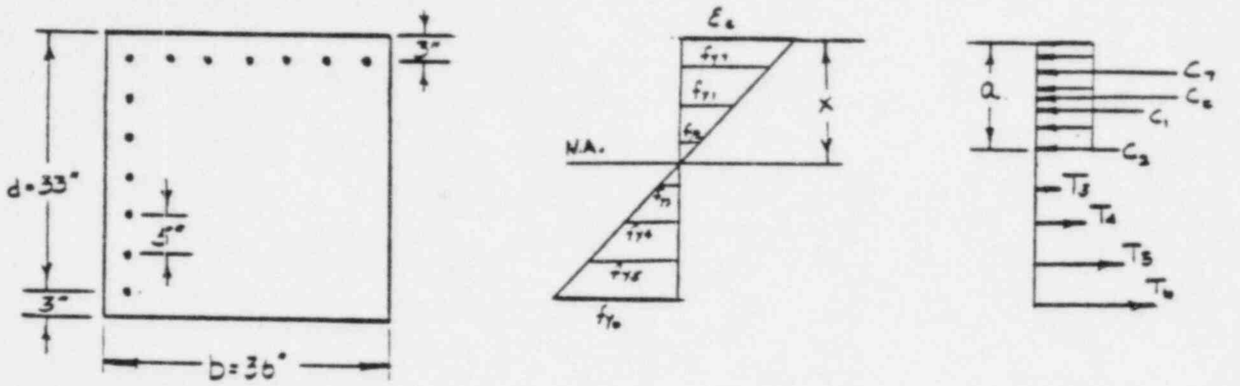
$l = 24'$ Between 4^{th} & 5^{th} Flr

$$\therefore M_{c45} = \frac{5.022(24)^2}{8}$$

$$M_{c45} = 361.584 \text{ k-ft.}$$

FORM CS-FRC-81

BALANCE CONDITION



Assume 3" cover on both sides for balanced condition

STEEL $f_y = 40$ KSI $\quad \varepsilon_y = 1.379 \times 10^{-3}$

CONCRETE $f'_c = 3$ KSI $\quad \varepsilon_c = 0.003$

First we find the balanced condition case

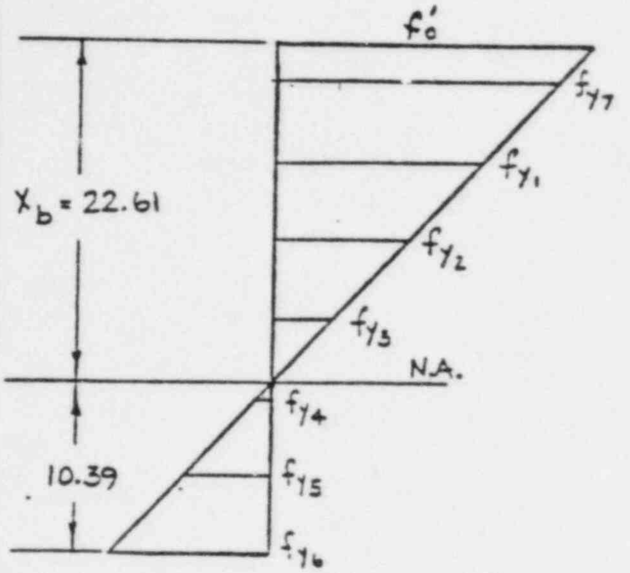
Balance Neutral Axis $\quad x_b = \left(\dfrac{\varepsilon_c}{\varepsilon_c + \varepsilon_y}\right) d$

$d = 33''$

$$x_b = \left(\frac{0.003}{0.003 + 1.379 \times 10^{-3}}\right) 33$$

$x_b = 22.61$ inches

Title REACTOR BLDG. - CAP. OF COL WEAKEST SECTION



From strain geometry:

$f_{y6} = 40$ ksi

$\frac{f_{y5}}{5.39} = \frac{40}{10.39}$ $f_{y5} = 20.75$ ksi

$\frac{f_{y4}}{.39} = \frac{40}{10.39}$ $f_{y4} = 1.5$ ksi

$\frac{f_{y3}}{4.61} = \frac{40}{10.39}$ $f_{y3} = 17.75$ ksi

$\frac{f_{y2}}{9.61} = \frac{40}{10.39}$ $f_{y2} = 37.0$ ksi

$\frac{f_{y1}}{14.61} = \frac{40}{10.39}$ $f_{y1} > f_y$

$\therefore f_{y1} = 40$ ksi

$\therefore f_{y7} = 40$ ksi

Now we find the allowable axial load P_b for balanced condition

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

$= (0.85)(3)(0.85)(22.61)(36)$

$= 1764.26$ kips

STEEL

Tension $= (7 \text{ of } \#11 \text{ bars}) f_{y6} + (2 \text{ bars}) f_{y5} + (2 \text{ bars}) f_{y4}$

$= (7 \times 1.56)(40) + (2 \times 1.56)(20.75) + (2 \times 1.56)(1.5)$

$T = 506.22$ kips

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Title: REACTOR BLDG. - CAPACITY OF COL. WEAKEST SECTION

Compression:

$$C_s = (7\,\text{bars} + 2\,\text{bars})(40-(.85)(3)) + (2\,\text{bars})(37-(.85)(3)) + (2\,\text{bars})(17.75-(.85)(3))$$

$$= (9\times1.56)(40-2.55) + (2\times1.56)(37-2.55) + (2\times1.56)(17.75-2.55)$$

$$C_s = 680.71 \text{ kips}$$

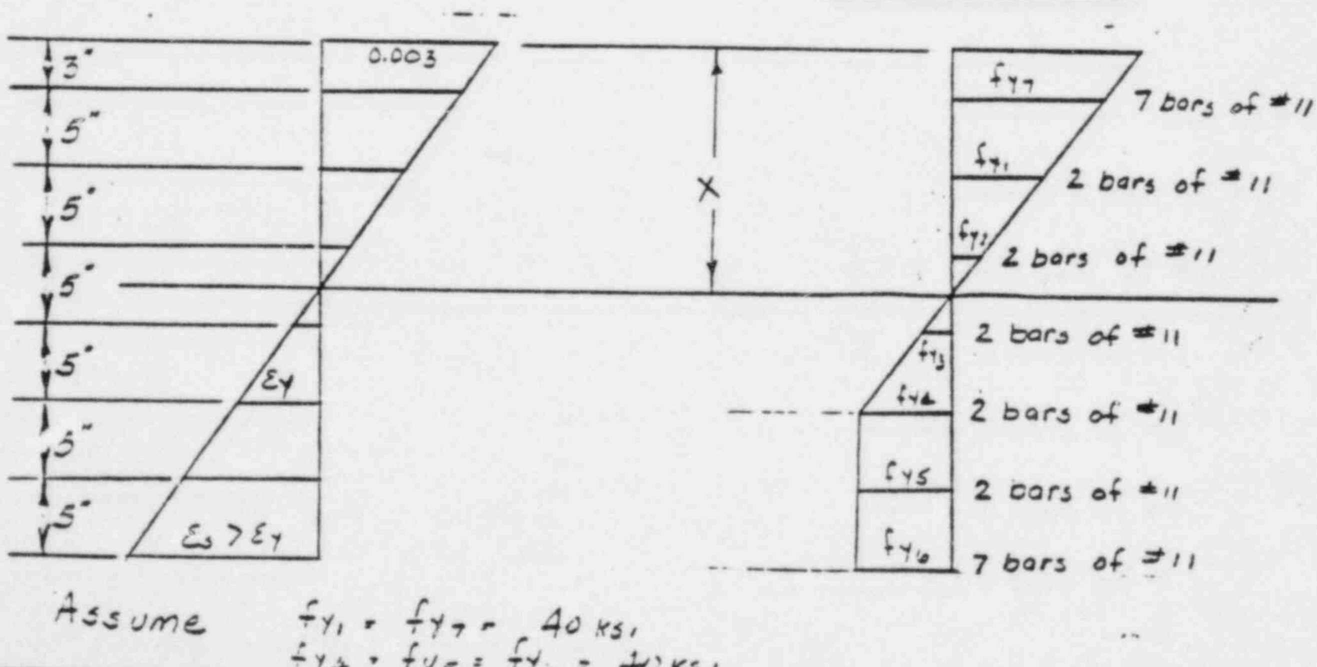
$$P_b = C_c + C_s \quad T$$

$$= 1764.26 + 680.71 - 506.22$$

$$P_b = 1938.75 \text{ kips}$$

since $0.8\ L.L = 115 \Rightarrow$ L.L. = 143.75
D.L. = 775.0
918.75 LOAD

Design column for axial load $\underline{P_a = 920 \text{ kips}}$



Assume $f_{y1} = f_{y7} = 40$ ksi
$f_{y2} = f_{y_-} = f_{y_-} = 40$ ksi

from strain geometry:

$$\frac{\varepsilon_{y_3}}{(18-x)} = \frac{0.003}{x}$$

$$\varepsilon_{y_3} = \frac{0.003\,(18-x)}{x} \qquad f_{y_3} = \frac{87\,(18-x)}{x}$$

$$\varepsilon_{y_2} = \frac{0.003\,(x-13)}{x} \qquad f_{y_2} = \frac{87\,(x-13)}{x}$$

$$\varepsilon_{y_1} = \frac{0.003\,(x-8)}{x} \qquad f_{y_1} = \frac{87\,(x-8)}{x}$$

STEEL

$$\text{Tension} = (7\text{ bars})f_{y_6} + (2\text{ bars})f_{y_5} + (2\text{ bars})f_{y_4} + (2\text{ bars})f_{y_3}$$

$$= \left[(7+2+2)\times 1.56\right]40 + (2\times 1.56)\left[\frac{87(18-x)}{x}\right]$$

$$= 686.4 + \frac{271.44\,(18-x)}{x}$$

Compression

$$C_s = (2\text{ bars})f_{y_2} + (2\text{ bars})f_{y_1} + (7\text{ bars})f_{y_7}$$

$$= (2\times 1.56)\left[\frac{87(x-13)}{x} + \frac{87(x-8)}{x}\right] + (7\times 1.56)\times 40$$

$$= 271.44\left(\frac{2x-21}{x}\right) + 436.8$$

$$C_c = 0.85\,f'_c\,(0.85\,x)(36)$$

$$= 78.03\,x$$

Title REACTOR BUILDING – REINF. CONC. COL. WEAKEST SECTION CAPACITY

$P = 920$ kips

$P = C_s + C_c - T$

$920 = \frac{271.44}{x}(2x-21) + 436.8 + 78.03x - 686.4 - \frac{271.44}{x}(18-x)$

$920 = 78.03x - 249.6 + \frac{271.44}{x}(2x-21-(18-x))$

$0 = 78.03x^2 - 1169.6x + 271.44(3x-39)$

$0 = 78.03x^2 - 1169.6x + 814.32x - 10586.16$

$0 = 78.03x^2 - 355.28x - 10586.16$

$x = \frac{355.28 \pm \sqrt{(355.28)^2 + 4(78.03)(10586.15)}}{2(78.03)}$

$x = \frac{355.28 \pm 1852.13}{156.06}$

$x = 14.14$ inches

$\therefore C_c = 78.03x = 78.03(14.14) = \underline{1103.7 \text{ kips}}$

$\therefore C_s = \frac{271.44}{x}(2x-21) + 436.8 = \frac{271.44}{14.14}(2 \cdot 14.14 - 21) + 436.8$

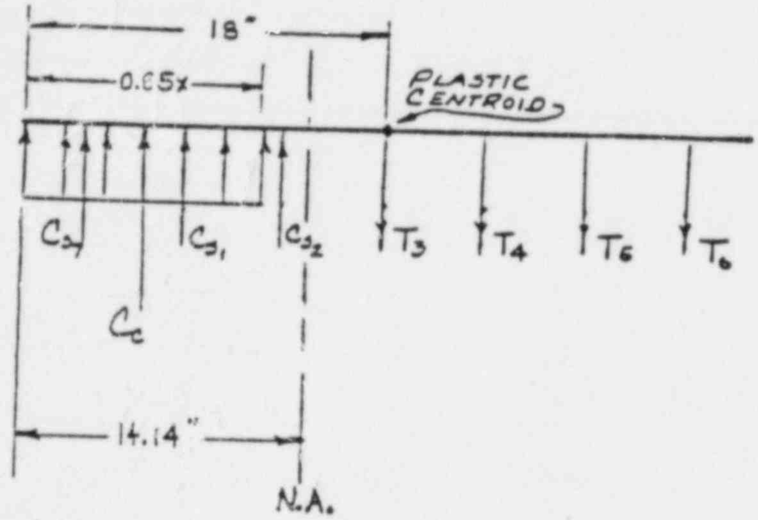
$= \underline{576.68 \text{ kips}}$

$\therefore T = 686.4 + \frac{271.44}{x}(18-x) = 686.4 + \frac{271.44}{14.14}(18-14.14)$

$= \underline{760.39 \text{ kips}}$

Title REACTOR BLDG. – CAPACITY OF COL. WEAKEST SECTION

Assume that the plastic centroid is at Mid-depth of section.



$C_{s_7} = 7 \cdot 1.56 \cdot 40 = 436.8$
$C_{s_1} = 2 \cdot 1.56 \cdot 37.79 = 117.92$
$C_{s_2} = 2 \cdot 1.56 \cdot 7.04 = 21.96$

$T_4 = 2 \cdot 1.56 \cdot 40 = 124.8$
$T_5 = 2 \cdot 1.56 \cdot 40 = 124.8$
$T_6 = 2 \cdot 1.56 \cdot 40 = 436.8$

Moment Capicity, by taking moment about plastic centroid.

$$\circlearrowleft M = C_c\left(18 - \frac{0.85x}{2}\right) + 15(C_{s_7} + T_6) + 10(C_{s_1} + T_5) + 5(C_{s_2} + T_4)$$

$$= 1103.7\left(18 - \frac{0.85(14.14)}{2}\right) + 15(436.8 + 436.8) + 10(117.92 + 124.8) + 5(21.96 + 124.8)$$

$$= 13231.76 + 13104 + 2427.2 + 733.8$$

$M = 29,496.76$ k-inch

$M = 2,458.1$ k·ft $> M_{col} = 483.407$ k-ft

NO FURTHER INVESTIGATION REQD.

∴ O.K

SAME COLUMN SECTION, HAS INCREASED REINFORCEMENT.
For 28 bars of #11 (9 bars instead of 7)

$$T = (2 \times 1.56 \times 40) + 686.4 + \frac{271.44}{x}(18 - x)$$

$$= 811.2 + \frac{271.44}{x}(18 - x)$$

$$C_s = \frac{271.44}{x}(2x - 21) + 436.8 + (2 \times 1.56 \times 40)$$

$$= \frac{271.44}{x}(2x - 21) + 561.6$$

∴ x remains the same

$$T = 760.39 + (2 \times 1.56 \times 40)$$

$$= 760.39 + 124.8$$

$$= 885.19$$

$$C_s = 576.68 + 124.8$$

$$= 701.48$$

$$M = 29496.76 + 15(2 \times 124.8) = 33240.76 \text{ k·inch}$$

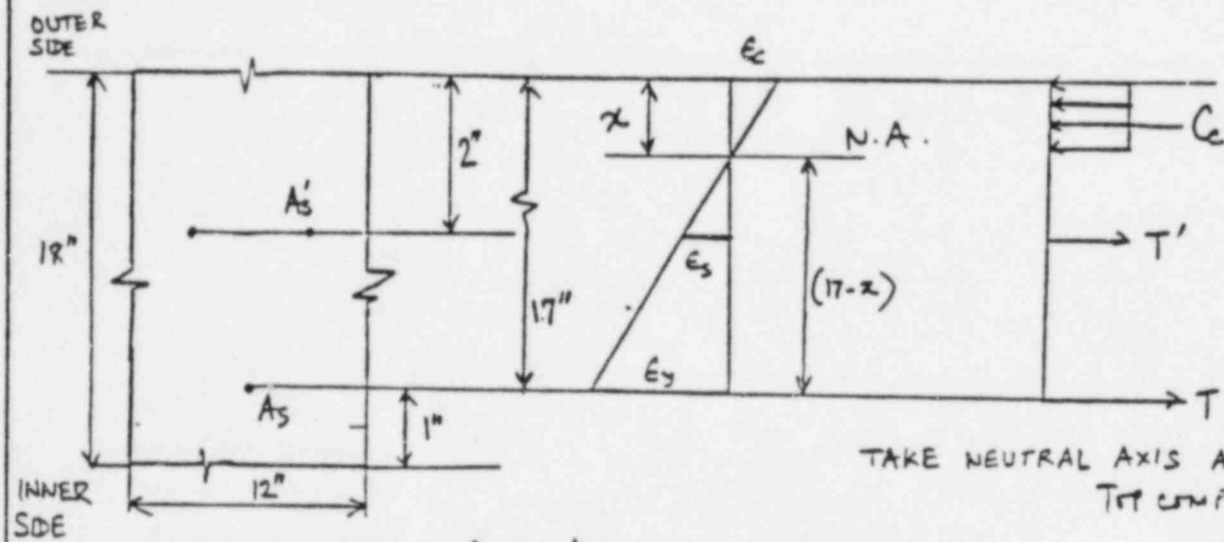
$$M = 2770.1 \text{ k·ft} > M_{col} = 483.407 \text{ k·ft}$$

∴ O.K

Title REACTOR BUILDING - EAST SIDE WALL SEC CAPACITY.

THE WALL SECTION CAPACITY IS FOUND FOR SECTION IN FLEXURE ONLY.

TAKE 1 FT WIDE WALL SECTION VERTICALLY



TAKE NEUTRAL AXIS AT X FROM TOP COMPRESSION FIBER.

All steel bars are #8

$A_s' = 2 \times 0.79 = 1.58 \text{ in}^2$

$A_s = 0.79 \text{ in}^2$

$f_c' = 3 \text{ ksi}$ Concrete

$f_y = 40 \text{ ksi}$ Steel

COMPRESSION

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

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

$\therefore C_c = 26.01x$

TENSION

$T = A_s f_y$

$= 0.79 \times 40$

$\therefore T = 31.6 \text{ ksi}$

FROM THE STRAIN GEOMETRY

$$\epsilon_y = 1.37931 \times 10^{-3}$$

$$\frac{\epsilon_y}{(17-x)} = \frac{\epsilon_s}{(2-x)}$$

$$\Rightarrow \epsilon_s = \left(\frac{2-x}{17-x}\right) 1.37931 \times 10^{-3}$$

$$f_s = 29{,}000\, \epsilon_s$$

$$\therefore f_s = 40\frac{(2-x)}{(17-x)}$$

$$T' = A_s' f_y$$

$$= 1.58 \times 40 \frac{(2-x)}{(17-x)}$$

$$\therefore T' = 63.2\frac{(2-x)}{(17-x)}$$

FOR EQUILIBRIUM $T' + T = C_c$

$$\therefore 31.6 + 63.2\frac{(2-x)}{(17-x)} = 26.01x$$

$$\Rightarrow 26.01x^2 - 536.97x + 663.6 = 0$$

$$\therefore x = 1.32025''$$

$$\therefore T' = 63.2\frac{(2-x)}{(17-x)}$$

$$T' = 2.7398 \text{ kips}$$

$$\therefore\ C_c = 26.01z$$

$$C_c = 34.3398 \text{ kips}$$

Now ~~the~~ Moment Capacity

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

$a = 0.85z$

$$= 31.6\left(17 - \frac{1.12222}{2}\right) + 2.7398\left(2 - \frac{1.1222}{2}\right)$$

$$\therefore M = 523.4113 \text{ k-inch}$$

$$= 43.618 \text{ k-ft.}$$

CAPACITY OF WALL WHEN INNER FACE IN TENSION

$M = 43.618 \text{ k-ft} > M_w = 20.792 \text{ k-ft}$

$\therefore$ WALL ALSO O.K.

THUS WE FIND THAT REINFORCED CONCRETE SECTION OF REACTOR BUILDING HAS HIGHER MOMENT CAPACITY (WALLS IN VERTICAL DIRECTION & THE COLUMNS) COMPARED TO ACTUAL MOMENT DUE TO PRESSURE DROP OUTSIDE LOADING OF 216 PSF INSIDE THE BUILDING, ~~FOR~~ THE ~~TORNADO~~ CASE.

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Project 02G-C5257-01 | Page A-15
By RA | Date APRIL 82 | Ch'k'd MLP | Date 4-82 | Rev. | Date

Title REACTOR BUILDING — STEEL ABOVE SERVICE FLOOR ELEV 119.5'

STEEL COLUMN ANALYSIS

[East & West Side More Critical
Higher Axial Load.]

- All columns are 14 WF 158 -
TO FIND THE AXIAL LOAD ON COLUMNS, TAKE WT OF STEEL IN TRUSS

Dead Weight of Steel in Half Truss

Member	Length	Wt/Unit Length	Weight
JL 6×4×3/4	53'-6"	47.2	2525.2
JL 6×4×3/4	53'-6"	47.2	2525.2
JL 5×3½×3/8	10.5'×2	20.8	436.8
JL 5×3½×5/16	10.5'×2	17.4	365.4
JL 4×3×1/4	10.5'×2	11.6	243.6
JL 5×3×7/16	13.8'×2	22.6	623.8
JL 4×3×3/8	13.8'×2	17.0	469.2
JL 4×3×1/4	13.8'×2	11.6	320.2
14 WF 61	23'3"	61.0	1418.2
6× 12 B 19	23'3"	19.0	2650.5
6× JL 5×3½×5/16	23'3"	17.4	2427.3
6× JL 3½×2½×1/4	25.5'	4.9	749.7
3× 8 WF 17	2.5'	17	127.5

TOTAL = 14882.6 lbs.

= 14.9 kips

TO ACCOUNT FOR ROOF DECKING ALSO ADD ABOUT 5 kips
FOR AXIAL LOAD, USE 20 kips

FORM CS-FRC-81

Title REACTOR BUILDING – STEEL ABOVE SERVICE FL. ELEV. 119.5'

UNBRACED HEIGHT

Unbraced height of column $l = 141'\text{-}1'' - 119'\text{-}5'' = 21'8'' > 12\,b_f = 16.5'$

$$\frac{Kl}{r_T} = \frac{21'\,8''}{4} = \frac{260}{4} = 65.$$

for A36 STEEL $C_c = 126.1$
$F_y = 36$ KSI

$$\left(\frac{Kl}{r}\Big/C_c\right) = \frac{65}{126.1} = 0.515$$

$$F_a = \frac{\left[1 - \frac{1}{2}\left(\frac{KL/r}{C_c}\right)^2\right]F_y}{\frac{5}{3} + \frac{3}{8}\left(\frac{KL/r}{C_c}\right) - \frac{1}{8}\left(\frac{KL/r}{C_c}\right)^3} = \frac{31.217}{1.843}$$

$$F_a = 16.94 \text{ KSI}$$

$$\frac{l}{r_T} = \frac{260}{4.36} = 59.63$$

$$F_b = \left[\frac{2}{3} - \frac{F_y\,(l/r_T)^2}{1530 \times 10^3}\right]F_y$$

$$F_b = 20.99 \text{ KSI}$$

Axial Load = 20 kips

$f_a = 0.43$ ksi

$$\frac{f_a}{F_a} = \frac{.43}{16.94} = 0.025 < 0.15$$

Use $\frac{f_a}{F_a} + \frac{f_b}{F_b} \leq 1.0$

$$f_b = F_b\left(1 - f_a/F_a\right)$$

$$f_b = 20.99\,(1 - 0.025) = 20.46$$

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

$$M = \frac{f_b \cdot I}{y} = \frac{(20.46)(1900)}{(15/2)} = 5184.53 \text{ k.inch}$$

$$= 432.0 \text{ k.ft}$$

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

$$w = 8\,(432)/(21.667)^2 = 7.36 \text{ k/ft} = 7360 \text{ lb/ft.}$$

PANEL WIDTH ON EAST & WEST SIDE = 23' 3"

pressure $p = \frac{w}{23.25} = \frac{7360}{23.25} = 316.6$ psf

ASSUMPTION: THIS IS ASSUMING THAT THE SIDING DOES NOT FAIL AND CAN TRANSFER THE ABOVE LOAD TO THE COLUMNS.

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Title: REACTOR BUILDING – STEEL ABOVE SERVICE FLR. ELEV 119.5'

NOW CHECK STEEL COLUMNS ON NORTH & SOUTH SIDE.

14WF158 Unbraced height = 37' 9"

$$\frac{KL}{r_{xx}} = \frac{37.75 \times 12}{6.4} = 70.78125 < C_c = 126.1$$

For $F_y = 36$ ksi

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

$$F_a = \frac{(0.84246)\ 36}{1.8551}$$

$$\therefore F_a = 16.349232 \text{ ksi} \quad ✓$$

Assume Axial load about 10 kips

$$f_a = \frac{10}{Area} = \frac{10}{46.5} = 0.2151 \text{ ksi} \quad ✓$$

$$\therefore \frac{f_a}{F_a} = 0.013154 < 0.15 \quad ✓$$

Unbraced height $l = 37.75$ ft $> 12 b_f = 16.5'$

$$\therefore \text{ Use } F_b = \frac{12 \times 10^3 C_b}{l d/A_f} = 32.623 > 0.6 F_y$$

Take $C_b = 1.0$

$$\therefore \text{ Use } F_b = 0.6 F_y = 22 \text{ ksi}$$

$$\frac{f_a}{F_a} + \frac{f_b}{F_b} \leq 1.0 \quad \text{can be used here}$$

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Title REATOR BUILDING – STEEL ABOVE SERVICE FLR. ELEV 119.5'

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

$$= 0.98685 \times 22$$

$$\therefore f_b = 21.710619 \text{ ksi}$$

$$M = \frac{f_b I}{y} = \frac{21.710619 \times 1900}{(15/2)} = 5500.0234 \text{ k-inch}$$

$$\therefore M = 458.3352.8 \text{ k-ft}.$$

$$M = \frac{wl^2}{8} \Rightarrow w = \frac{8M}{l^2} = \frac{8 \times 458.33528}{(37.75)^2}$$

$$\therefore w = 2.5729975 \text{ k/ft}$$

This takes load laterally of 26' 9"

$$\therefore \text{Pressure } p = \frac{w}{26.75}$$

$$= \frac{2572.9975}{26.75}$$

$$\therefore p = 96.187 \text{ psf}.$$

ROOF STEEL *

Critical Member 12B19 Supports 8'11" x 23'3" area

A36 steel
Area = $5.59\ in^2$
$d = 12.16\ in$
$I_{xx} = 130\ in^4$ $\quad r_{xx} = 4.82\ in.$
$I_{yy} = 3.76\ in^4$ $\quad r_{yy} = 0.82\ in.$
$r_T = 1.01\ in.$

$$\left(\frac{KL}{r}\right)_{min.} = \frac{(23.25)(12)}{0.82} = 340.24$$

TAKE K=1

$$\frac{KL}{r_T} = \frac{(23.25)(12)}{1.01} = 276.24 \quad > 119\sqrt{C_b} = 119$$

TAKE $C_b = 1$

$$\text{Use } F_b = \frac{170 \times 10^3 \times C_b}{(l/r_T)^2} = \frac{170 \times 10^3}{(276.24)^2}$$

$$F_b = 2.23\ ksi$$

$$F_b = \frac{M}{I} \cdot y$$

$$M = \frac{F_b \cdot I}{y} = \frac{(2.23)(130)}{(12.16/2)} = 47.68\ k\text{-inch}$$

$$= 3.97\ k \cdot ft$$

* ROOF STEEL CONSISTS OF MAIN TRUSSES IN EAST-WEST DIRECTION AND SUPPORTING SWAY TRUSSES IN NORTH-SOUTH DIRECTION ROOF DECKING UK 16-16 Q DECK IS WELD TO TOP CHORD OF ROOF STEEL. NOW IF ROOF DECKING DOES NOT FAIL, ALL LOAD IS TRANSFERRED TO TOP CHORD IN WHICH 12B19 IS CRITICAL MEMBER.

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Title
REACTOR BUILDING – STEEL ABOVE SERVICE FL. ELEV 119.5'

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

$$W = \frac{8M}{l^2} = \frac{8(3.97)}{(23.25)^2} = .059 \text{ k/ft}$$
$$= 59 \text{ lb/ft}$$

$$\text{Pressure } \frac{W}{8'11''} = \frac{59}{8.917} = 6.62 \text{ psf.}$$

ASSUMPTION : THIS IS TRUE IF WE ASSUME THAT ROOF DECKING DOES NOT FAIL AND ALL LOAD IS TRANSFERRED TO 12B19

FOR ALLOWABLE PRESSURE ADD THE SELF WT OF BEAM & ROOF DECK

ASSUME WT. OF ROOF DECKING = 3 psf

SELF WT. OF BEAM = 19 lbs/ft.
CONVERT THIS TO PRESSURE $= \frac{19}{8.9167}$

$= 2.13084$ psf

∴ TOTAL ALLOWABLE PRESSURE FOR

UPLIFT $= 6.62 + 3 + 2.131$

$= \underline{11.751 \text{ psf}}$

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APPENDIX B

VENTILATION STACK DESIGN REVIEW CALCULATIONS

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FOR THE DESIGN CALCULATIONS ACI 307-79 WAS USED. THIS USES WORKING STRESS DESIGN. FOR THE CONCRETE WE USED ALLOWABLE STRESSES ACCORDING TO CODE ACI 307-79.

FOR WIND AND DEAD LOAD STRESSE ALLOWABLES:

a) STRESS IN CONCRETE, AT OUTSIDE DIAMETER

$$f_{cw} \leq 0.25 f_c'$$

FOR STACK $f_c' = 5000\,psi$

b) STRESS IN THE VERTICAL REINFORCEMENT ON THE WINDWARD SIDE

$f_{sw} \leq 12.5\,ksi$ FOR $F_y = 40\,ksi$ STEEL.

THE STACK IS BETWEEN ELEV −3'0" AND ELEV 391'0"

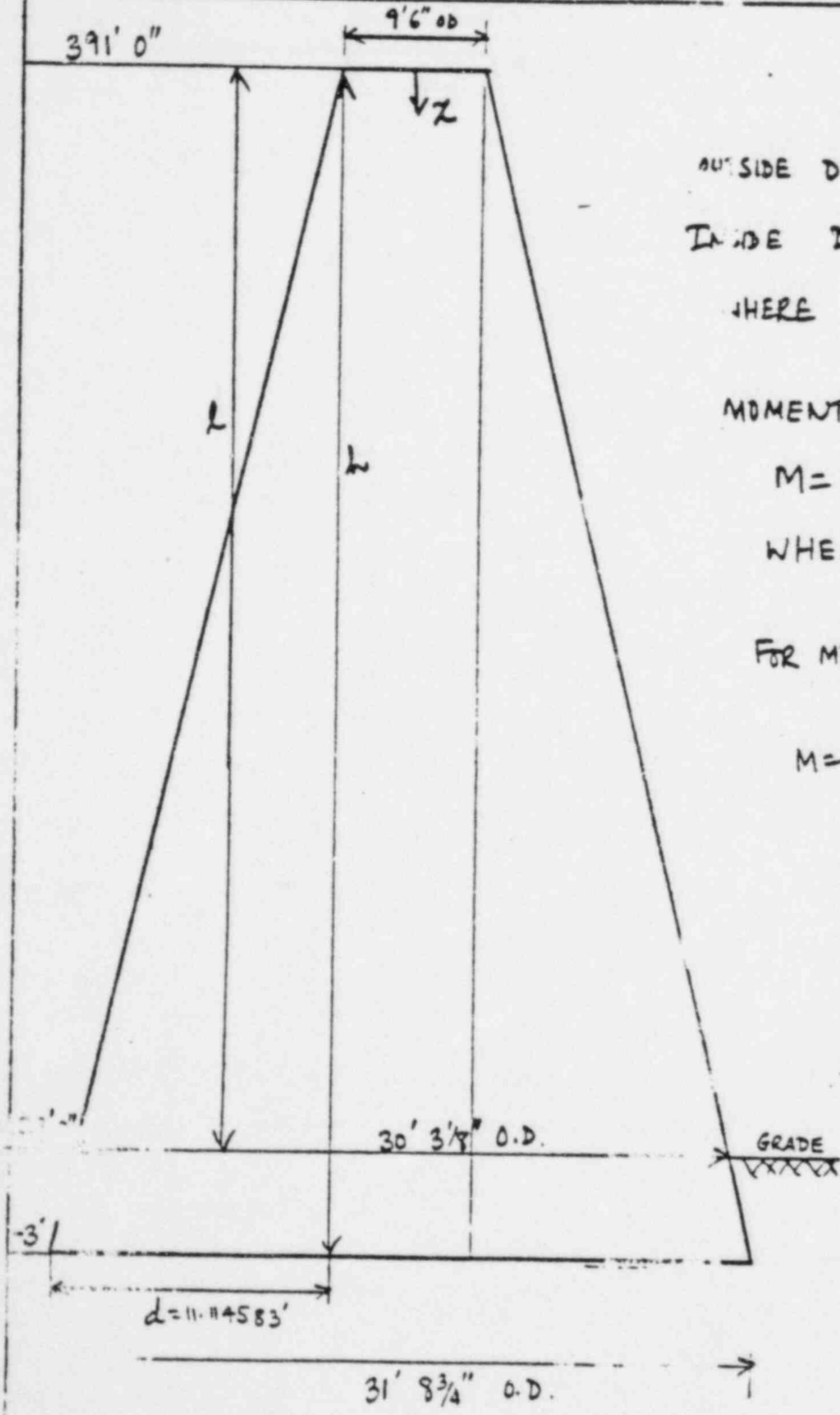
THE GRADE IS AT ELEV 23'0"

THE TOP HAS OUTER DIAMETER OF 9'6" AND HAS 6" THICK CONCRETE.

AT GRADE O.D IS 30' 3 1/8" AND AT BASE
AT ELEV−3' O.D. IS 31' 8 3/4", 18" THICK CONCRETE

WE ASSUME THE OUTER SURFACE OF STACK GRADUALLY AND SMOOTHLY CHANGES

Title: CONCRETE STACK — OYSTER CREEK



$b = 9.5\ ft \qquad L = 3$[illegible]

OUTSIDE DIA. $D_o = b + (2d/L)x$

INSIDE DIA. $D_I = D_o - 2T$

WHERE T = THICKNESS

MOMENT FOR ABOVE GRADE

$$M = P\left[b\frac{x^2}{2} + \frac{L}{3}\frac{dx^3}{L}\right]$$

WHERE P = DYNAMIC PRESSURE.

FOR MOMENT BELOW GRADE

$$M = P\left[b\ell\left(x - \frac{\ell}{2}\right) + \frac{d\ell^2}{L}\left(x - \frac{2}{3}\ell\right)\right]$$

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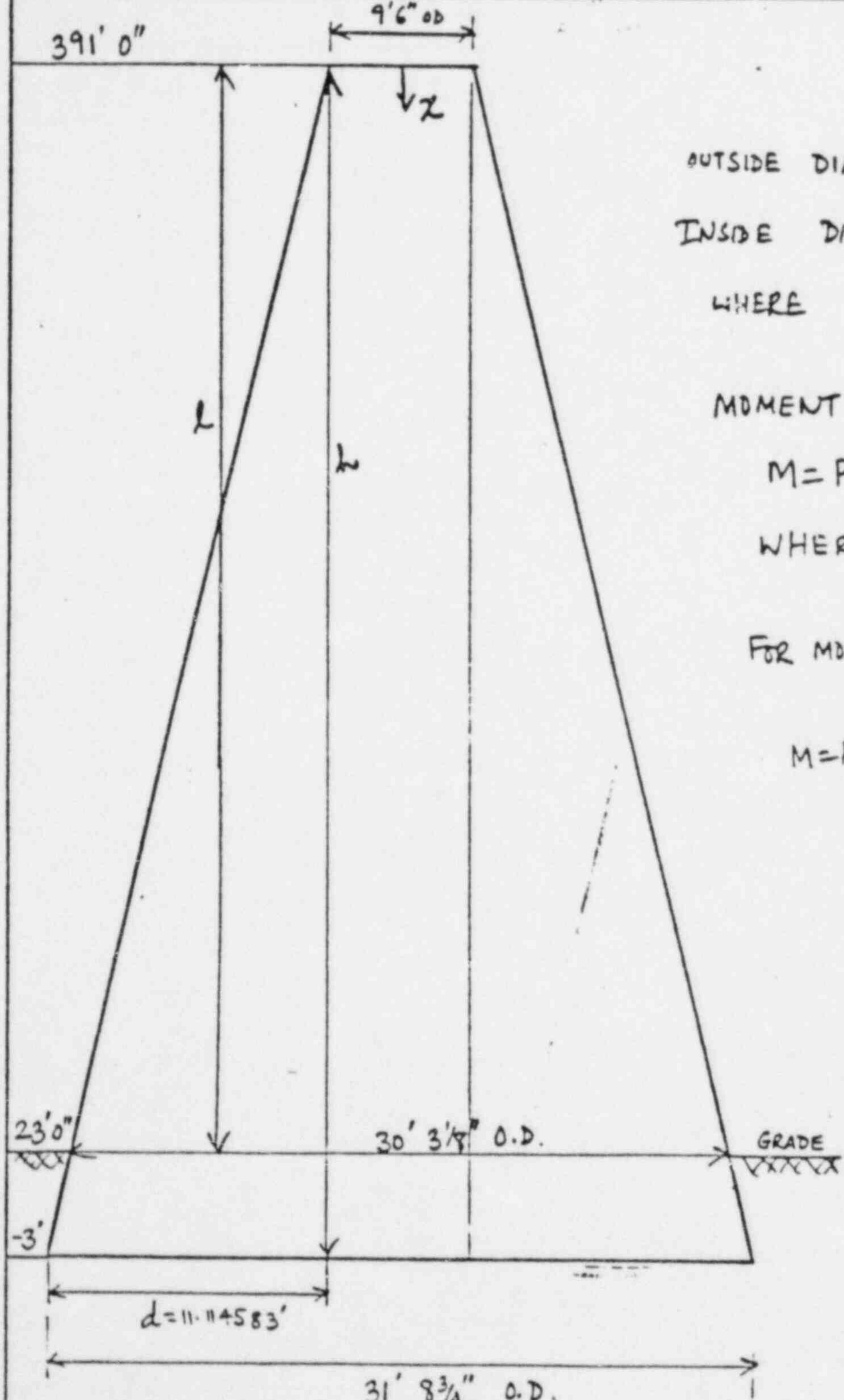
Title: CONCRETE STACK — OYSTER CREEK



OUTSIDE DIA. $D_o = b + (2d/L)x$

INSIDE DIA. $D_I = D_o - 2T$

WHERE T = THICKNESS

MOMENT FOR ABOVE GRADE

$$M = P\left[b\frac{x^2}{2} + \frac{L}{3}\frac{dx^3}{L}\right]$$

WHERE P = DYNAMIC PRESSURE.

FOR MOMENT BELOW GRADE

$$M = P\left[bl\left(x - \frac{l}{2}\right) + \frac{dl^2}{L}\left(x - \frac{2}{3}l\right)\right]$$

$b = 9.5$ ft $\quad L = 394'$

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Title CONCRETE STACK - OYSTER CREEK

DISTANCE FROM TOP IN FT.	THICKNESS OF SECTION INCHES	DIAMETERS IN FT. OUTER	INNER	STEEL AREA IN²	MOMENT CAPACITY K-FT.	α	EQUIVALENT PRESSURE Lb/ft²
112.7344	6.00	15.8604	14.8604	40.2	4602.2	62.0°	62.33 ✓
142.7969	7.00	17.5565	16.3898	35.48	5804.21	61.25°	46.72
172.8594	8.00	19.2526	17.9193	32.12	7508.277	61.25°	39.413
202.922	9.00	20.9487	19.4487	30.10	9836.6708	61.75°	35.88
233.0	10.00	22.6448	20.9781	28.52	12853.374	62.5°	34.12 ✓
278.1	12.00	25.1889	23.1889	29.45	19503.219	63.5°	34.25 ✓
315.6563	13.00	27.3091	25.1424	55.12	29028.458	66.0°	37.74 ✓
323.1719	19.00	27.7331	24.5664	63.6	32135.550	60.7°	39.51
353.1719	20.00	29.4257	26.0923	66.0	42717.543	63.25°	42.44
338.1719 Ⓐ	25.00	28.5794	24.4127	79.0	104983.98	88°	115.77
386.50 Ⓑ	18.00	31.3060	28.3060	73.2	105371.54	95.75°	84.484

α & β ARE DEFINED IN ACI 307-79 CODE

Ⓐ THIS SECTION HAS THREE OPENINGS WHICH ARE EQUIVALENT TO OPENING ANGLE $\beta = 49.812°$. THE THREE OPENINGS WERE SUMMED UP TO BE ASSUMED EQUIVALENT TO ONE OPENING.

Ⓑ THIS SECTION IS BELOW GRADE. HAS TWO OPENINGS WHICH ARE EQUIVALENT TO $\beta = 42.0995°$. TWO OPENINGS SUMMED-UP AND ASSUMED EQUIVALENT TO ONE OPENING. IN THIS SECTION ONLY CONCRETE REACHES ALLOWABLE FIRST.

IN ALL OTHER SECTIONS REINFORCING STEEL REACHES ALLOWABLE STRESS BEFORE CONCRETE.

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FOR A SECTION 233' BELOW FROM THE TOP, FOR ...

1) ANGLE $\alpha = 62.5°$ WITH STEEL REACHING ALLOWABLE STRESS OF 12.5 ksi BEFORE CONCRETE ALLOWABLE 1.25 ksi

MOMENT CAPACITY M = 12853 374 k-ft. $\frac{P}{A}$ + . . Y

THIS WHEN CONVERTED BACK PRESSURE P = 34.12 psf.

2) BUT IF WE INCREASE ALLOWABLE STRESSES TO:

IN REINFORCEMENT USE YIELD STRESS $f_{SN} = f_y = 40$ ksi

IN CONCRETE USE $0.85 f_c'$ AS ALLOWABLE

THEN ALSO STEEL REACHES YIELD FIRST FOR $\alpha = 47.5°$

MOMENT CAPACITY M = 19342.361 k-ft

EQUIVALENT PRESSURE = 51.34 psf.

FOR SECTION 278.1' BELOW FROM THE TOP FOR

1) ANGLE $\alpha = 63.5°$ WITH STEEL REACHING ALLOWABLE OF 12.5 ksi BEFORE CONCRETE ALLOWABLE 1.25 ksi

MOMENT CAPACITY M = 19503.219 k-ft

EQUIVALENT PRESSURE = 34.25 psf

2) ALLOWABLE IN REINFORCEMENT $f_{SN} = f_y = 40$ ksi AND IN CONCRETE ALLOWABLE = 0.85 ...

$\alpha = 47.375°$

MOMENT CAPACITY M = 27328.402 k-ft

EQUIVALENT PRESSURE = 47.986 psf

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Title CONCRETE STACK — OYSTER CREEK.

FOR A SECTION 233' BELOW FROM THE TOP, FOR

1) ANGLE $\alpha = 62.5°$ WITH STEEL REACHING ALLOWABLE STRESS OF 12.5 ksi BEFORE CONCRETE ALLOWABLE 1.25 ksi

MOMENT CAPACITY M = 12853.374 K-ft.

THIS WHEN CONVERTED BACK PRESSURE P = 34.12 psf.

2) BUT IF WE INCREASE ALLOWABLE STRESSES TO:

IN REINFORCEMENT USE YIELD STRESS $f_{sw} = f_y = 40$ ksi

IN CONCRETE – USE $0.85 f_c'$ AS ALLOWABLE

THEN ALSO STEEL REACHES YIELD FIRST FOR $\alpha = 47.5°$

MOMENT CAPACITY M = 19342.361 K-ft

EQUIVALENT PRESSURE = 51.34 psf.

FOR SECTION 278.1' BELOW FROM THE TOP FOR

1) ANGLE $\alpha = 63.5°$ WITH STEEL REACHING ALLOWABLE OF 12.5 ksi BEFORE CONCRETE ALLOWABLE 1.25 ksi

MOMENT CAPACITY M = 19503.219 K-ft
EQUIVALENT PRESSURE = 34.25 psf

2) ALLOWABLE IN REINFORCEMENT $f_{sw} = f_y = 40$ ksi AND IN CONCRETE ALLOWABLE = $0.85 f_c'$ $\alpha = 47.375°$

MOMENT CAPACITY M = 27328.402 K-ft
EQUIVALENT PRESSURE = 47.986 psf

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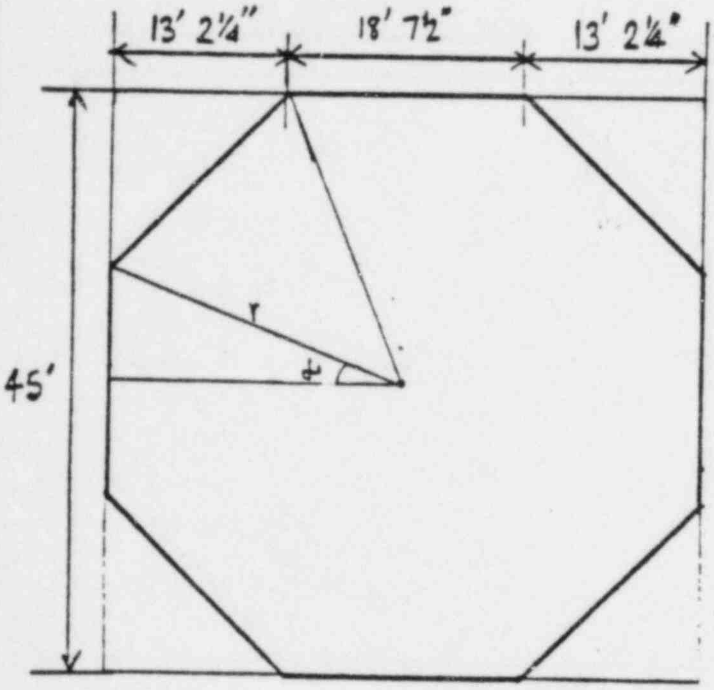
CONCRETE STACK FOUNDATION IS A OCTOGNAL SHAPED 7 FT. THICK REINFORCED CONCRETE SECTION



Number of Sides $n = 8$

AREA $A = \dfrac{a^2 n}{4 \tan\alpha}$

$a = 18.625'$
$\alpha = 22.5°$

$\therefore A = \dfrac{(18.625)^2\, 8}{4 \tan 22.5}$

$A = 1674.9361\ ft^2$ ✓

INERTIA ABOUT ANY AXIS $I = \dfrac{A}{24}\left(6\rho_1^2 - a^2\right)$

$\rho_1 = \dfrac{a}{2\sin\alpha} = \dfrac{18.625}{2 \sin 22.5}$

$r = 24.354\ ft.$

$\therefore \rho_1 = 24.3347$ ✓

$I = \dfrac{1674.9361}{24}\left(6\times(24.3347)^2 - (18.625)^2\right)$

$\therefore I = 223756.49\ ft^4$ ✓

DEAD WT OF FOUNDATION $= 0.15 \times 1674.9361 \times 7.0$

$= 1758.6829$ kips

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CONCRETE STACK FOUNDATI OYSTER CREEK.

TOTAL WT. OF CHIM = 3578.6345 kips

∴ T... T $W = 3578.6345 + 1758.6829$

$W = 5337.3175$ kips.

USE $\sigma =$

$y = r = 24.354$ ft

FOR σ USE ARING CAPACITY OF 13 KSF AS ON DWG. # 4049-7

$\therefore\ 13 = \ldots + \frac{M(24.354)}{223756.49}$

$\therefore\ M = 9.\ \ 6$ k-ft.

CONVERT THIS TO PRESS BY FORMULA

$M = p\left[bl(x- \ldots - \frac{dl^2}{h}(x - \frac{2}{3}l)\right]$

$l = 368$ ft

$x = 404$ ft

$b = 9.5$ ft

$d = 11.114583$ ft

$h = 394$ ft

$90163.106 = p \ldots 368(404 - \frac{368}{2}) + \frac{11.114583}{394}(368)^2(404 - \frac{368\times 2}{3})]$

$= \ldots 267.5)$

$\therefore p \ldots .55604$ ksf

$\therefore \ldots 561$ psf

FOR 0.7 SHAPE FACTOR IS IS EQUIVALENT TO 191.33 mph

TOTAL WT. OF CHIMNEY = 3578.6345 kips

∴ TOTAL DEAD WT $W = 3578.6345 + 1758.6829$

$W = 5337.3175$ kips.

USE $\sigma = \frac{P}{A} + \frac{M}{I} \cdot y$

$y = r = 24.354$ ft

FOR σ USE SOIL BEARING CAPACITY OF 13 KSF AS GIVEN ON DWG. # 4049-7

$$\therefore \quad 13 = \frac{5337.3175}{1674.9361} + \frac{M(24.354)}{223756.49}$$

$$\therefore \quad M = 90163.106 \text{ k-ft.}$$

CONVERT THIS TO PRESSURE p BY FORMULA

$$M = p\left[bl\left(x - \frac{l}{2}\right) + \frac{dl^2}{h}\left(x - \frac{2}{3}l\right)\right]$$

$l = 368$ ft
$x = 404$ ft
$b = 9.5$ ft
$d = 11.114583$ ft.
$h = 394$ ft

$$90163.106 = p\left[(9.5)368\left(404 - \frac{368}{2}\right) + \frac{11.114583}{394}(368)^2\left(404 - \frac{368 \times 2}{3}\right)\right]$$

$$= p\,(1375267.5)$$

$$\therefore p = 0.065564 \text{ ksf}$$

$$\therefore P = 65.561 \text{ psf}$$

FOR 0.7 SHAPE FACTOR THIS IS EQUIVALENT TO 191.33 mph