ATTACHMENT 1

TECHNICAL EVALUATION REPORT

REVIEW OF WIND AND TORNADO LOADING RESPONSES

JERSEY CENTRAL POWER AND LIGHT COMPANY
OYSTER CREEK NUCLEAR GENERATING STATION

NRC DOCKET NO. 50-219

NRC TAC NO. 49392

NRC CONTRACT NO. NRC-03-81-130

FRC PROJECT C5506

FRC ASSIGNMENT 17

FRC TASK 428

Prepared by

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

Prepared for

Nuclear Regulatory Commission
Washington, D.C. 20555

Author: D. J. Barrett

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Lead NRC Engineer: D. Persinko

October 31, 1984

FRANKLIN RESEARCH CENTER
DIVISION OF ARVIN/CALSPAN

20th and Race Streets, Phila., Pa. 19103 (215) 448-1000

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Date: Oct. 31, 1984	Date: 10/31/84	Date: 10-31-84

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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

A study of the Oyster Creek Nuclear Generating Station was undertaken by Jersey Central Power and Light Company (JCP&L) to examine the ability of the plant's civil engineering structures to resist a windstorm or a tornado strike. The purpose of this review is to provide a technical evaluation of the approach, analysis, and conclusions of the JCP&L study.

1.2 GENERIC ISSUE BACKGROUND

The current design criteria for nuclear power plant structures contain provisions for protection against windstorms and tornadoes. These requirements were not in effect at the time that some of the older nuclear plants were designed and licensed. Due to concerns regarding the extent to which these older plants can satisfy the current wind loading licensing criteria, the Nuclear Regulatory Commission (NRC), as part of the Systematic Evaluation Program (SEP), initiated Topic III-2, "Wind and Tornado Loadings," to investigate and assess the structural safety of existing designs.

The SEP encompasses a broad range of safety-related issues, many of which are concerned with the integrity of plant structures. The Franklin Research Center (FRC) provided technical assistance to the NRC in the review of several SEP topics and was responsible for technical evaluations for Topic III-2 under Assignment 17 of NRC Contract No. NRC-03-81-130.

1.3 PLANT-SPECIFIC BACKGROUND

1.3.1 Oyster Creek Structural Review

In a previous Technical Evaluation Report (TER) [1], the FRC staff examined a sample of the structures at the Oyster Creek plant for resistance to high wind and tornado loadings. This effort determined that, although most of the designs had adequate strength, some of the structural components were not designed to meet the provisions of current licensing criteria. The NRC included the findings of the FRC study in the Integrated Plant Safety Assessment [2] and also reported the safety-related concerns directly to JCP&L

in an evaluation letter [3] of a previously issued SEP Topic III-2 Safety Analysis Report (SAR) [4].

In response to the safety issues raised by the NRC, JCP&L issued a letter and support documents, herein referred to as the Oyster Creek Structural Review (OCSR) [5], establishing the basis of the position taken in the SAR concerning the adequacy of the structural systems. These documents included engineering calculations that were used to establish the windspeed strength ratings of structures. A subsequent letter [6] included information and calculations on a tornado analysis of the diesel generator building.

FRC was then charged with making a technical evaluation of the OCSR documents. The approach to this task was threefold:

1. review the procedures, criteria, and conclusions of the OCSR.
2. audit the engineering calculations.
3. seek to resolve outstanding safety issues through independent analysis (subject to the limits of the resources assigned to this task).

The particular review items are identified and discussed in Section 3 of this report; the conclusions are summarized in Section 4.

1.3.2 Turbine Building Structural Review

A draft TER [7] reporting the review of the OCSR documents was issued on November 30, 1983. Subsequent to that date, the NRC requested FRC to examine an additional tornado analysis [8] performed by JCP&L for the Oyster Creek turbine building. The purpose of the Turbine Building Tornado Evaluation (TBTE) was identical to that of the OCSR, and the findings of the review of the TBTE documents have been included in the final version of this report.

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 [9] 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.

The Nuclear Regulatory Guide 1.76 [10] defines the design basis tornado (DBT) in terms of six descriptive parameters: the maximum wind speed, the rotational speed, the translational speed, the maximum atmospheric pressure drop, the rate of pressure drop, and the core radius. The specified magnitudes of these regional parameters (listed with respect to geographical location) are the acceptable regulation levels; however, where appropriate, additional meteorological analysis may be performed to justify the selection of a less conservative DBT. In Reference 11, the NRC established the tornado parameters to be used in the SEP study of the Oyster Creek plant.

Regulatory Guide 1.117 [12] identifies the structures and systems that should be protected from the effects of a DBT. This information is elaborated on in Branch Technical Position AAB 3-2 found in the Standard Review Plan (SRP), Section 3.5.1.4 (NUREG-0800) [13]. The OCSR analysis reviewed in this report included most of the safety-related structural systems of the Oyster Creek plant.

A velocity pressure model of a windstorm can be constructed from the pressure and air flow assumptions stated in Section 3.3.1 of the SRP [14] and the American National Standards Institute (ANSI) design loading guide [15]. A velocity pressure model of a tornado strike can be constructed from the DBT characteristics based on the guidance of Section 3.3.2 of the SRP [16] and the engineering literature [17, 18]. The actual loads acting on a structure are calculated from these models through the use of experimentally determined

pressure coefficients [15, 19]. The loads act on the structural surfaces as positive and negative pressures induced by the change in momentum of the wind and in the atmospheric pressure.

An additional tornado load is the impact of windborne missiles against structures. The potential missiles are listed in the missile spectrum of Section 3.5.1.4 of the SRP [13], and the particular missiles to be included in this study were identified by the NRC as part of the SEP assignment [11]. References 20 and 21 assist in the determination of the structural effects of missile impact, whereas the guidelines of the SRP [16] 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 [22] and Section 3.8.4 of the SRP [23]). The structural effects of these loading combinations are determined by analysis; stresses are calculated either by a working stress or an 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. The specifications 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" [10]

NRC Regulatory Guide 1.117, "Tornado Design Classification" [12]

NUREG-0800, Standard Review Plan

Section 3.3.1, "Wind Loadings" [14]
Section 3.3.2, "Tornado Loadings" [16]
Section 3.5.1.4, "Missiles Generated by Natural Phenomena" [13]

Section 3.5.3, "Barrier Design Procedures" [24]
Section 3.8.1, "Concrete Containment" [25]
Section 3.8.4, "Other Seismic Category I Structures" [23]
Section 3.8.5, "Foundations" [26]

AISC Specification for Design, Fabrication, and Erection of Structural Steel for Buildings, Eighth Edition [27]

ACI-318-77, "Building Code Requirements for Reinforced Concrete" [28]

ASME Boiler and Pressure Vessel Code, Section III, Division 2 (ACI-359), "Standard Code for Concrete Reactor Vessels and Containments" [22]

NRC/SEB, "Criteria for Safety-Related Masonry Wall Evaluation," Structural Engineering Branch (1981) [29]

ACI-307-79, "Specification for the Design and Construction of Reinforced Concrete Chimneys" [30].

3. TECHNICAL EVALUATION

3.1 GENERAL INFORMATION

Based on a meteorological study of the Oyster Creek region, the NRC established the following site-specific DBT characteristics:

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 correspond to a tornado with a probability of occurrence of 10^{-7} per year (as required by current licensing criteria). The characteristics were used to calculate the structural loadings of the initial review [1, 11] and are used as the basis of computations in the present review.

The following subsections review the approaches, analysis, and conclusions of the OCSR that are used by JCP&L to support the conclusions of the Oyster Creek SAR. Important steps of the analysis are examined, and an evaluation of the adequacy of the steps are made. Audit calculations are performed to check the analysis. Where necessary, calculations supporting, refuting, or correcting the JCP&L position were made and are included in the appendices of this report.

The major structures of the Oyster Creek plant are the reactor building, the control room, the intake structure, the diesel generator building, the radwaste building, the turbine building, the ventilation stack, and various exposed mechanical components. All of these structures, except for the radwaste and turbine buildings, have been included in the OCSR. The TBTE specifically addresses the turbine building structure alone. As an aid in interpreting the structural review, a site plot plan is shown in Figure 1.

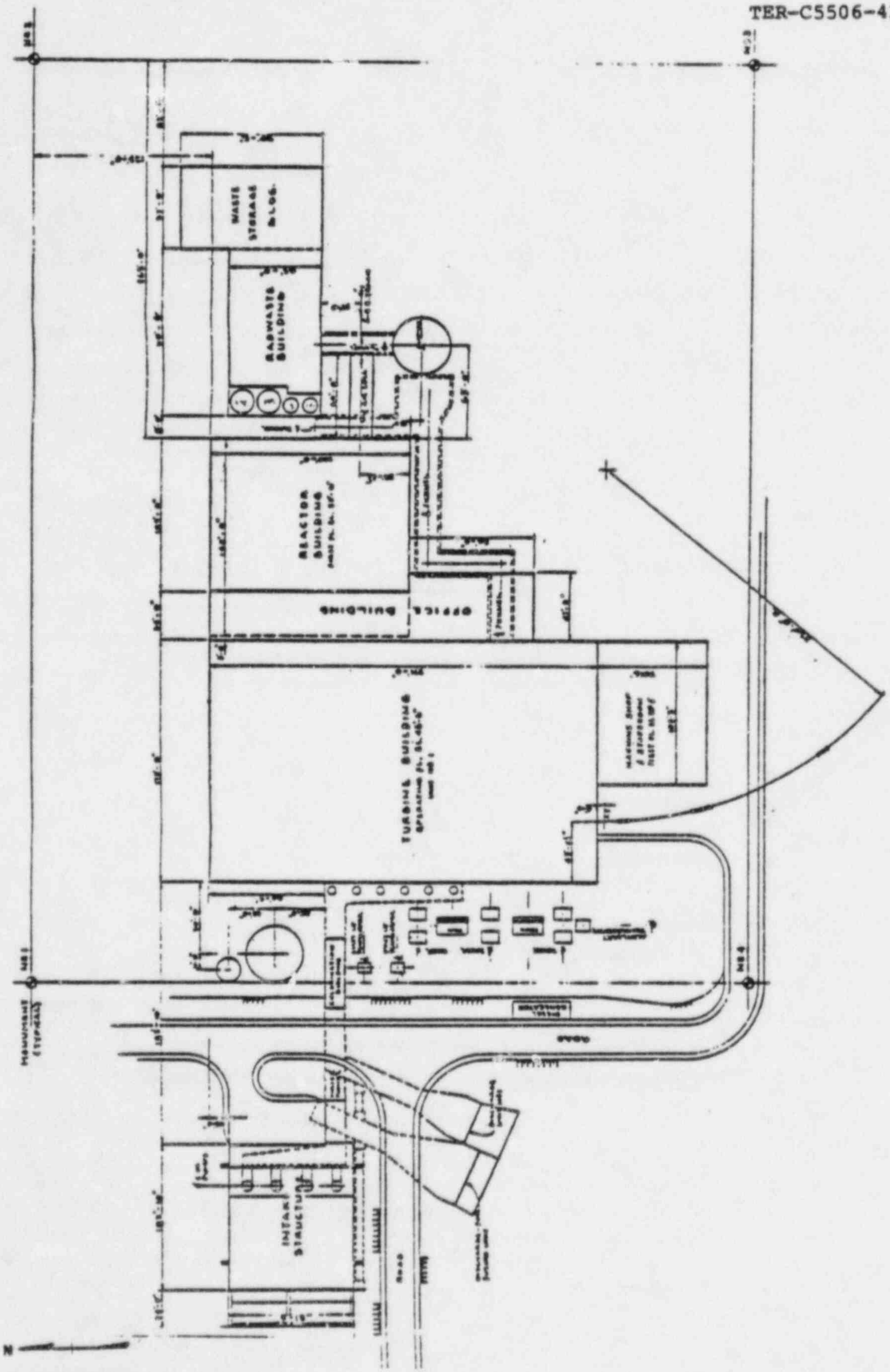

Figure 1. Site Plot Plan

3.2 EFFECTIVE TORNADO LOADINGS

3.2.1 Atmospheric Pressure Change

Evaluation

Given the translational core speed and the windspeed distribution of a tornado, there is a well-defined procedure for calculating the magnitude of the atmospheric pressure change (APC) that occurs at any given point in a tornado [18]. The maximum APC occurs in the center of the tornado core. Since the intent of the calculations of the OCSR was to support the load resistance ratings of the SAR, the wind speeds corresponding to the maximum APC were never calculated.

Conclusion

For structures that resist the full structural loadings resulting from a tornado strike, it is unnecessary to find the limiting windspeed rating. For those structural components that are being qualified for reduced loadings, the limiting windspeed rating corresponding to the APC will be calculated in Appendix A of this report.*

3.2.2 Wind Velocity Pressure

Evaluation

The methods used in the OCSR to calculate wind velocity pressures for windstorms and tornado strikes follow the procedures delineated in Sections 3.3.1 and 3.3.2 of the SRP [14, 16] and ANSI A58.1-1982 [15].

Conclusion

The wind velocity pressures were calculated in accordance with the applicable criteria.

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* Note that this calculation merely expresses the APC windspeed rating of the components based on the strength reported in the OCSR. The adequacy of the reported APC resistance is subject to the validity of the structural review criteria (see Section 3.5).

3.2.3 Windborne Missiles

Evaluation

In evaluating the structural effects of missile impacts, the OCSR bases its conclusions on equations and procedures provided in the engineering literature [20, 31]. These procedures are consistent with the intent of Section 3.5.3 of the SRP [24], which requires missile impacts to be modeled as concentrated loads acting in combination with other applied loadings. The overall response of the structure to such loadings was then examined.

The missiles used in analysis were the steel rod, telephone pole, and automobile of the missile spectrum [13]. Therefore, the missile specifications of Reference 11 were satisfied.

Conclusion

The global structural effects of missile impact were examined in accordance with the applicable criteria (the components examined were limited to those which satisfy the requirements of the other tornado loadings).

3.3 STRUCTURAL LOADINGS

3.3.1 Differential Pressure Load

Evaluation

The atmospheric pressure change of a tornado leads to a lowering of the ambient pressure outside of a structure. For an unvented structure, this change results in differential pressure loadings acting outwardly on the building surfaces. In the tornado strike analysis, the OCSR included the differential pressure as a basic loading condition. For structural components being qualified to resist the full effects of a tornado, the correct maximum value of the differential pressure loading was examined. For those components with limiting strength, loadings were examined that were more conservative than the differential pressure loads.

Conclusion

The differential pressure load was applied to the structures in accordance with the applicable criteria [16].

3.3.2 Effective Structural Pressures

Evaluation

The wind velocity pressure is converted to actual structural loadings through the use of pressure coefficients and gust factors. These coefficients vary for the direction of wind flow and for the section of the structure under consideration. The OCSR analysis draws from the appropriate reference sources [14, 15, 16, 18] for the values of these coefficients.

The applicable criteria [16, 17] recognize that the peak wind velocity pressure occurs in a limited region and that this pressure rapidly decays away from this region. The overall structural pressure acting on a building surface is therefore greatly reduced from a uniform pressure based on the peak wind velocity. However, individual components on the building exterior must still be qualified for the local application of the peak pressure. In the OCSR, all of the components were examined for the peak pressure value.

Conclusion

The calculations for converting wind velocity pressures to effective structural pressures are in accordance with the applicable criteria. The local effect of the application of the peak pressure to exterior structural components was examined.

3.3.3 Design Loads

Evaluation

The design loads that are to be considered acting in combination with tornado loads and wind loads are dead, live, thermal, and pipe reaction loads [23]. The analysis in the OCSR has included these additional loadings where applicable. The magnitude of these loads while acting in combination with the

differential pressure load was not always computed correctly; however, the combinations formed were usually conservative.

Conclusion

The design loads were identified and included in the analysis in accordance with the applicable criteria. Cases where the magnitude of the design loads were incorrectly and unconservatively computed will be identified in Section 3.6, Structural Systems, of this report.

3.3.4 Shielding

Evaluation

The term "shielding" refers to the reduction or elimination of wind loads on a structure from the blockage of wind flow by an upstream obstruction (another structure, physical formation, etc.). The provisions of the applicable standard prohibit the reliance on shielding for the reduction of wind loads. The OCSR does not rely on shielding for the qualification of structural components and, where applicable, accounts for the transmission of lateral forces through structures.

Conclusion

Shielding and the transmission of lateral forces were treated in accordance with the applicable criteria and good engineering judgment.

3.3.5 Load Combinations

Evaluation

The correct load combinations for severe and extreme environmental events are specified in various sections of the SRP [23, 25]. Specific load combinations of the basic tornado-related loadings are given in Section 3.3.2 of the SRP [16]. In establishing the capacity of structural components, the

OCSR formed load combinations that are equal to or more conservative* than those specified for combinations involving effective structural pressures and differential pressures. Although missile loads were considered in proper load combinations for the reactor building, the combination was not properly considered for the control room, ventilation stack, and the diesel generator building.

Conclusion

For some structural components, the missile load combinations were not formed in accordance with the applicable criteria.

3.4 STRUCTURAL ANALYSIS AND MODELING

The review items included in this section (main structural frame, sequence of failure, secondary members, roof decking, etc.) are either not applicable to the OCSR or are addressed elsewhere in this report.

3.5 STRUCTURAL ACCEPTANCE CRITERIA

3.5.1 Steel Components

Evaluation

The extreme environmental structural acceptance criteria specified by the SRP [23] recognized the severity of the loadings presented in the rare occurrence of a tornado and, as such, permitted stress levels in steel components to approach yield conditions. This allowance may result in large deflections but will still guard against instabilities so that a failure mechanism will not occur. In the review of steel components, the OCSR acceptance criterion is based on the tensile strength of steel.** Such

*In many cases, the load combination formed in the qualification of components whose strength was found to be limiting was excessively conservative. The chief conservative measure was combining dynamic pressures and differential pressures whose magnitude did not correspond to the same wind speed

**That is, for components subjected to bending loads, the stress levels were compared with the tensile strength of steel. Compression members (columns) were not examined in the OCSR but were reviewed in an earlier study [1].

a criterion results in larger deflections than are permitted in the SRP and does not ensure structural stability.

Conclusion

The structural acceptance criterion for steel components employed in the OCSR is not in conformance with the applicable criteria. It is also not recommended on the basis of accepted engineering practice.

3.5.2 Concrete Components

Evaluation

As with steel components, the SRP recognizes a tornado strike as a special event and permits levels of stress to occur that are above the established code allowables. The strength of concrete components is therefore based on the ultimate capacity of sections, which allows the stress in the reinforcing bars to reach yield values and the stress in the concrete to approach the ultimate compressive strength. In conducting the review of concrete components,* the analysis of the OCSR adhered to these increased upper limits.

Conclusion

The structural acceptance criterion for concrete components employed in the OCSR is in conformance with the applicable criteria.

3.5.3 Masonry Block Walls

Evaluation

The NRC/SEB has established masonry block wall structural acceptance criteria [23, 29] for stresses due to extreme environmental events. These

*In the ventilation stack analysis, the stresses in the steel rebar were found to be slightly above yield values for the stated windspeed capacity. This is considered acceptable in light of the analysis presented in Section 3.6.9 of this report.

criteria permit overload factors for all basic stress levels including tensile masonry stresses for unreinforced blocks. In Reference 32, NRC asked if any of the Oyster Creek masonry walls will be affected by tornado loads. JCP&L responded that there are no masonry walls in the Category I areas of the plant's structures (Reference 5, page 9, of "Responses to NRC Questions").

Conclusion

This review item is not applicable for the structures at the Oyster Creek plant.

3.5.4 Connections

Evaluation

Steel connections are permitted the same overstress factor for extreme environmental loadings as steel members. This factor is applied to the allowable stresses as specified in the steel code [27]. The OCSR does not state the criteria used to examine steel connections and does not indicate if these components were examined. However, in typical engineering designs, of which the structures of the Oyster Creek plant are representative, connections are designed to meet the limiting allowable capacity of members. In this case, given the manner in which loads are resisted in the Oyster Creek steel structures, it is concluded that the connections will not be the limiting components of the steel frames if they were sized in accordance with good engineering practice.

Conclusion

This item is not critical for the structures at the Oyster Creek plant.

3.5.5 Roof Decks

Evaluation

Roof decks are typically constructed of light gage steel sheets. Under extreme loadings, the bending compression zones of the decks will be susceptible to local buckling failures. In establishing the capacity of the roof

decks, the OCSR used beam bending formulas without any consideration of reducing stresses for the buckling of local compression elements.

Conclusion

Roof decks were not analyzed in accordance with accepted practice. The limiting failure mechanism of these components was not found.

3.5.6 Architectural Components

Evaluation

Large-scale architectural components such as structural siding and roll-up doors must be reviewed for tornado resistance, and the consequences to the main structure caused by their failure must be examined. No architectural details were included in the analysis of the OCSR.

Conclusion

Verification of the lack of architectural components in critical structures is recommended. If these components are present, they should be included in the wind and tornado loading review.

3.6 STRUCTURAL SYSTEMS

3.6.1 Concrete Components of the Reactor Building

Evaluation

For effective structural pressure and differential pressure loadings, the analysis presented in the OCSR uses a working stress design method for the review of concrete components. Although this procedure is not in accordance with the accepted criteria [23], the conclusions formed are valid since they are in agreement with a previous study [1] and since the members in question inherently have a high capacity against these loadings (Attachment 1 of Reference 5, pp. 6-11).

Missile loadings acting in conjunction with other loads are reviewed through the accepted review procedure, and the conclusions formed are based on the appropriate criteria (Attachment 1 of Reference 5, p. 45).

Conclusion

The SAR windspeed strength ratings for the react[illegible]ilding concrete components are valid and are based on the accepted criteria.

3.6.2 Steel Components of the Reactor Building

Evaluation

The steel components of the reactor building were not reviewed with respect to the standard structural acceptance criteria. Furthermore, not all of the important load-resisting members were included in the review of tornado loadings (see Attachments 1 and 5 of Reference 5). The stated capacities of some elements (girts, purlins, etc.) were based on procedures not consistent with good engineering practice (member capacities based on material tensile strength, see Section 3.5.1).

Conclusion

The SAR windspeed strength ratings for the reactor building steel components are invalid and are based on criteria not in conformance with the accepted criteria.

3.6.3 Metal Siding Systems of the Reactor Building

Evaluation

The analysis used to qualify the insulated metal siding of the reactor building neither accounts for the possibility of buckling of local compression components nor examines the capacity of the underlying connections. Nevertheless, the windspeed rating established for these components is of the level predicted by an experimental procedure performed elsewhere [33]. Based on the experimental results and assuming that the connections of the siding systems are comparable, the capacity of the Oyster Creek siding system has been overestimated by approximately 15 to 20%.

Conclusion

If a safety-related concern arises that involves the metal siding systems, then their stated capacity should be reexamined.

3.6.4 Control Room

Evaluation

The concrete walls and panels of the control room were examined in a manner similar to that used to review the concrete components of the reactor building (Attachment 1 of Reference 5, pp. 19-24). The capacity of the panel connections to resist differential pressure loadings was examined in an appropriate manner.

Conclusion

The windspeed ratings that are listed in the SAR for the control room are valid and are based on the accepted criteria. Note that the OCSR concludes that the control room walls will not be able to resist load combinations involving missile impacts.

3.6.5 Intake Structure

Evaluation

The components of this structure were examined in accordance with the accepted criteria.

Conclusion

The windspeed ratings listed in the SAR for the intake structure are valid and are based on the accepted criteria.

3.6.6 Diesel Generator Building

Evaluation

The concrete walls and roof slab of the diesel generator building were examined in a manner similar to that used to review the concrete components of

the reactor building [6]. Load combinations involving the impact of tornado missiles were not reported.

Conclusion

The windspeed ratings reported in the SAR for the diesel generator building are valid and are based on accepted criteria. This structure's resistance to missile impacts was not examined in the OCSR.

3.6.7 Radwaste Building

Evaluation

The radwaste building is a structure whose failure may have significant safety consequences. This building was not included in the tornado loading review of the OCSR. Justification for excluding this structure from the study was not presented.

Conclusion

The NRC has stated that it does not consider this structure to be an essential review item.

3.6.8 Exposed Mechanical Components

Evaluation

The Integrated Plant Safety Assessment Report identifies the safety-related mechanical components that are not housed in qualified structures. The majority of these components are reported to be enclosed in other structures with adequate protection or are identified as being of insignificant safety-related concern. Some structures (condensate storage tank, condensate storage pumps, and service water pumps) are to be reviewed under SEP Topic III-4.A, "Tornado Missiles." The three remaining components to be reviewed for resistance to a tornado strike are the service water pump, the emergency service water pump, and the start-up transformer.

The calculations used to qualify these components study the capacity of the supports and the structural base to resist lateral loads and overturning moments. In addition, depressurization loads (due to the APC) on the component housing were addressed. The adequacy of the component structure was therefore established. However, no attempt was made to qualify the mechanical components' ability to remain functional after the application of tornado loadings.

Conclusion

The conclusions on the adequacy of the structural support and housing of the mechanical components are valid and based on the accepted criteria. The functional ability of the mechanical components to resist tornado loadings was not examined.

3.6.9 Ventilation Stack

Evaluation

The OCSR examined the ventilation stack through an approach based on maximum stress design (MSD). This technique relies on working stress design formulas [30] but allows the stress in the steel rebar to reach yield stress and the stress in concrete to approach its ultimate compressive strength. Although such a procedure is sound and in conformance with engineering practice [34], the structural acceptance criteria for extreme environmental events permit reinforced concrete structures to be reviewed only through ultimate strength design techniques (USD). However, USD is not an accepted design theory for stack structures (but there is a current movement working towards the acceptability of such a practice). To resolve this dilemma, it is meaningful to examine the ventilation stack through USD to see if the results corroborate the conclusions formed from the MSD study.

The reviewers constructed a USD model of the chimney based on the geometry, service loads, and material property information provided in the OCSR documents. In addition, the following assumptions were included in the model:

1. The longitudinal steel is placed towards the outside face of the stack, and there is a 2-in cover over the circumferential reinforcement.

2. Thermal effects do not influence the strength conclusions for an extreme environmental event. (This assumption is supported by the acceptable stress criteria for working stress designs. See attachment 1 of Reference 5.)

Wind flowing past a stack gives rise to the regular shedding of vortices. These vortices cause a pressure drop across the cylinder which gives rise to lateral forces. If the shedding frequency of the vortices matches a natural frequency of the stack (resonance), then significant dynamic effects can result. Based on the natural frequencies of the vertical stack [35], the wind speeds at which resonance is likely to occur were calculated.

Another dynamic effect due to wind flowing past a stack is ring vibration (in the sectional plane). Wind speeds at which this phenomenon is likely to occur were also estimated.

Conclusion

The results of the USD study corroborated the static strength conclusions of the OCSR. However, the stack was found to have a limiting resonant wind speed of 133 mph. Specific details of the analysis can be found in Appendix B.

3.6.10 Turbine Building

Evaluation

The turbine building is not seismically classified as a Category I type structure. However, the turbine building is adjacent to the control room so that its resistance against collapse during an extreme environmental event is essential. Therefore, the purpose of the TBTE was to examine the ability of the turbine building to remain stable with increasing windspeeds and successive component failure.

The TBTE modeled the turbine building structural systems as two- and three-dimensional frames which were analyzed through the use of a standard computer code. The TBTE document includes the results of this analysis and

the design review of the critical components. It also includes the rationale and procedure for iterating the level of loading vs. component failure to find the windspeed level where the turbine building steel begins to impinge on the control room structure. The conclusions of the TBTE are as follows:

1. At a windspeed of 107 mph, the anchor bolts of some frames will be overstressed. However, the structure will remain stable.

2. At a windspeed of 139 mph, all of the anchor bolts will have yielded. However, stability will still be maintained.

3. At a windspeed of 158 mph, the turbine building frames will have deflected to the point where they will be in contact with the roof of the control room structure.

4. At a windspeed of 212 mph, the anchor bolts will reach ultimate strength and fail. This will result in the collapse of the structure. In the pre-collapse state, the reactions acting on the control room structure from the deflected turbine building frames are less than the level of lateral loading that is expected to occur during an operating basis earthquake (OBE).

The computer program inputs, the tornado loading conditions, and the design review of some structural components were not included in the TBTE documents.

To judge the validity of the turbine building analysis, FRC studied the structure, reviewed the TBTE documents, and performed checks on critical structural members. Because the TBTE documents did not provide a comprehensive report on all of the analytical procedures, FRC's conclusions rely heavily on a subjective study of the structure (backed up by spot-check calculations) and on the assumptions that the procedures found in the OCSR documents are indicative of what was done in the TBTE (i.e., pressure calculations, load combinations, component reviews, etc.)

The structural system of the turbine building was found to be well designed so that this structure would be capable of resisting levels of loading well above the normal operating loads. Some design features that enhance the strength of this structure are as follows:

1. The lateral load-resisting members of the roof steel are structurally separated from the purlins. Thus, the number of components subject to both axial loads and transverse loads is minimized.

2. Horizontal roof trusses provide strong load paths for transmitting the load on intermediate wall columns to the vertically braced lateral column lines.

3. The roof steel sway bracing members are oversized and formed into strong truss subsystems. Thus, the sway bracing can assist in resisting lateral loads by distributing these loads throughout the entire structure.

4. The secondary bracing and axial load resisting members are oversized.

5. The truss arrangements and fastened cross braces reduce the component unbraced lengths.

In the structural review, FRC found components whose strengths were more limiting than those reported in the TBTE. However, the failure of these components would not endanger the control room structure.

Conclusion

The conclusion that a collapse of the turbine building will not occur before windspeeds reach 212 mph is supported by the TBTE documents and a partial study of the structure but is contingent on the following clarifications:

1. On the turbine building design drawings [36], a note indicates that the connections in the structure are to be designed to at least 50% of the capacity of sections. This implies that the structural design is limited by the capacity of the connections and not the capacity of the members. Since the TBTE did not address member connections, an investigation into their adequacy is recommended.

2. In the TBTE, the reactions between the turbine building and the control room structure are qualified by comparison to the OBE loadings. However, these two types of loadings are fundamentally different because the tornado-related load is a highly localized applied loading, whereas the earthquake load is a distributed type load. Although both loadings may lead to the same conclusion on the overall structural strength, it is recommended that the local effects of the contact loading be examined in detail.

4. CONCLUSIONS

The conclusions from the review of the OCSR windstorm and tornado strike analysis are summarized in Tables 1 and 2.

Table 1. OCSR Load and Review Criteria Summary

Review Item	Status[a] 1	2	3	4
Effective Tornado Loadings				
Atmospheric Pressure Change	X			
Wind Velocity Pressure	X			
Windborne Missiles	X			
Structural Loadings				
Differential Pressure Load	X			
Effective Structural Pressure	X			
Design Loads	X			
Shielding	X			
Load Combinations		X[b]		
Structural Acceptance Criteria				
Steel Components		X		
Concrete Components	X			
Masonry Block Walls				X
Connections				X[c]
Roof Decks		X		
Architectural Components			X	

a. The status of each item is defined as 1, 2, 3, or 4, as follows:
   1 = The review item is in conformance with or more conservative than the accepted criteria.
   2 = The review item is not in conformance with the accepted criteria.
   3 = The review item was not addressed in the OCSR and remains an open issue.
   4 = The review item is not applicable.

b. Nonconformance for this item stems from examining the missile loads acting alone and not in conjunction with the wind-related loadings.

c. The connections are assumed to have been sized in accordance with good engineering practice.

Table 2. OCSR Structures and Components Summary

Review Item	Status[a] 1	Status[a] 2	Status[a] 3
Concrete Components of Reactor Building	X		
Steel Components of Reactor Building		X[b]	
Metal Siding System		X[b]	
Control Room	X[c]		
Intake Structure	X		
Diesel Generator Building	X[c]		
Radwaste Building			X
Exposed Mechanical Components	X[d]		
Ventilation Stack	X[c]		

a. The status of each item is defined as 1, 2, or 3, as follows:
   1 = The strength conclusions reported in the SAR [4] are adequately supported by the OCSR documents.
   2 = The strength conclusions reported in the SAR are not adequately supported by the OCSR documents. Where applicable, the strength ratings are limited to the findings of the TER [1].
   3 = The structure or component was not adequately addressed in the SAR and OCSR, and its qualification remains an open issue.

b. If destruction of the steel portions of the reactor building does not cause significant safety-related concerns, then these items can be removed from the OCSR review.

c. Note that the OCSR concludes that this structure will not be able to resist load combintions involving missile impacts. Missile impacts were not examined acting in conjunction with other wind-related loadings. For the diesel generator building, missile impacts were not examined at all.

d. The ability of the mechanical support structure to resist tornado loadings was established. The functional ability of the equipment to survive a tornado strike was not examined.

In addition, the results of the USD study of the ventilation stack are summarized in Table 3; Table 4 lists the results of the structural vibration study.

Table 3. Strength Summary of Ventilation Stack

Stress Type	Review Procedure	Wind Speed (mph)
Longitudinal Stresses	Maximum Stress Design	180
	Ultimate Strength Design	196
Circumferential Stresses[a]	Maximum Stress Design	145
	Ultimate Strength Design	153
Foundation Stresses	Working Stress Design[b]	180
	Ultimate Strength Design	>180
Soil-Bearing Stresses	Working Stress[c]	191

a. The circumferential stress windspeed rating is that level of loading at which longitudinal cracks on the outside surface will occur. Note that this loading does not result in stack collapse.
b. In the OCSR, the foundation and soil stresses are qualified on the basis of comparison to the earthquake loading (see p. 27, Attachment 3 of Reference 5).
c. This result of the OCSR is based on a soil pressure in excess of 18 ksf. It is not clear whether 18 ksf is the allowable or ultimate soil capacity.

Table 4. Wind-Induced Structural Vibration

Vibration Concern	Mode	Resonant Wind Speed (mph)
Vortex Shedding	3	133[a]
Vortex Shedding	4	197
Ovalling	Fundamental	286

a. Because this resonant wind speed is close to the longitudinal strength windspeed rating (see page B-22), it is the recommended limiting windspeed rating for the stack. It is recognized that the third mode may not lead to critical base moments but will cause high bending moments throughout the middle and upper regions or the stack where this particular chimney is weak. Note that if large damping values are allowed then these moments will be substantially reduced.

The conclusions of the TBTE are justified subject to the clarifications of Section 3.6.10 of this report. The results of the turbine building study are summarized in Table 5.

Table 5. Strength Summary of Turbine Building

Component	Loading Type	Wind Speed (mph)
Column 10 Da	Differential Pressure	50
	Dynamic Pressure	76
Edge Lateral Strut W10x33[a] (El. 101 ft 10 in)	Dynamic Pressure	165

a. Although not normally designed as such, the sway bracing members at this elevation are stocky and can help resist lateral loads. Therefore, the actual total structural resistance will be greater than 165 mph.

5. REFERENCES

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Franklin Research Center, Technical Evaluation Report
TER-C5257-402, June 1982

2. Integrated Plant Safety Assessment
Oyster Creek Nuclear Generating Station
Final Report
U.S. Nuclear Regulatory Commission
NUREG-0822, September 1982

3. D. M. Crutchfield (NRC)
Letter to P. B. Fiedler (JCP&L)
Subject: SEP Topic III-2, Wind and Tornado Loadings
September 1, 1982

4. I. R. Finrock (JCP&L)
Letter with Attachments to W. Paulson (NRC)
Subject: Oyster Creek Nuclear Generating Station Systematic Evaluation Program
May 7, 1981

5. P. B. Fiedler (JCP&L)
Letter with Attachments to D. M. Crutchfield (NRC)
Subject: SEP Topic III-2, Wind and Tornado Loadings
Oyster Creek Nuclear Generating Station
Docket No. 50-219
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6. Y. Nagal (JCP&L)
Letter with Attachments to E. McKenna (NRC)
Subject: Tornado Wind Evaluation of Diesel Generator Building
October 25, 1983

7. Barrett, D. J.
"Review of Wind and Tornado Loading Responses, Oyster Creek Nuclear Generating Station"
Franklin Research Center, Technical Evaluation Report
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Letter with Attachment to D. M. Crutchfield (NRC)
Subject: Turbine Building Tornado Evaluation
March 13, 1984

9. Code of Federal Regulations, Title 10, Part 50, Appendix A, "General Design Criteria"

10. Regulatory Guide 1.76
"Design Basis Tornado for Nuclear Power Plants"
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Letter to S. P. Carfagno (FRC)
Subject: Tentative Work Assignment P
April 23, 1981

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NRC, July 1981
NUREG-0800

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NRC, July 1981
NUREG-0800

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NUREG-0800

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NUREG-0800

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Detroit: American Concrete Institute, 1977
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Nuclear Engineering and Design, Vol. 37, pp. 183-203 (1976)

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Letter to I. R. Finfrock (JCP&L)
Subject: SEP Topic III-2, Wind and Tornado Loadings
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Letter to D. M. Crutchfield (NRC)
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NUREG/CR-1981, UCRL-53018

36. "Turbine Building Exterior Wall Framing and Bracing"
W. O. 2299, Drawing #4213-2
Burns and Roe, Inc.

APPENDIX A

GENERAL DESIGN REVIEW CALCULATIONS

FRANKLIN RESEARCH CENTER
DIVISION OF ARVIN/CALSPAN

20th and Race Streets, Phila., Pa. 19103 (215) 448-1000

Franklin Research Center A Division of The Franklin Institute The Benjamin Franklin Parkway, Phila. Pa. 19103	Project 564-506-001			Page A-1
	By DJB	Date 11/22/83	Ch'k'd Date	Rev. Date

Title: OYSTER CREEK STRUCTURAL REVIEW, CRITICAL LOAD CASES

LOADING TYPES (SEE SRP 3.3.2)

W_p = DIFF. PRESSURE $= -.00512\,(.89\,(V_{MAX}-5))^2 = -.00406\,(V_{MAX}-5)^2$

W_W = WIND LOAD $= C_p\,.00256\,V_{MAX}^2$

LOAD CASES

$W_T^{(1)} = W_W = .00256\,C_p\,V_{MAX}^2$

FOR FRONT	$= (.8)\,(.00256\,V_{MAX}^2)$		$= .00205\,V_{MAX}^2$
SIDE	$= (-.7)$	"	$= -.00179\,V_{MAX}^2$
BACKWALL	$= (-.5)$	"	$= -.00128\,V_{MAX}^2$

$W_T^{(2)} = W_p = -.00406\,(V_{MAX}-5)^2$

FOR ALL SURFACES $= -.00406\,V_{MAX}^2 + .0406\,V_{MAX} - .102$

$W_T^{(3)} = W_T + .5\,W_p$

FOR FRONT $= .00205\,V_{MAX}^2 - .00203\,(V_{MAX}-5)^2$

SIDE $= -.00179\,V_{MAX}^2 - .00203\,(V_{MAX}-5)^2 = -.00382\,V_{MAX}^2 + .0203\,V_{MAX} - .0508$

BACKWALL $= -.00128\,V_{MAX}^2 - .00203\,(V_{MAX}-5)^2$

MAXIMUM LOADINGS

POSITIVE PRESSURE $W_T = W_T^{(1)} = W_W = .00205\,V_{MAX}^2$

NEGATIVE PRESSURE $W_T = W_T^{(3)} = W_p = -.00406\,(V_{MAX}-5)^2$

FORM CS-FRC-81

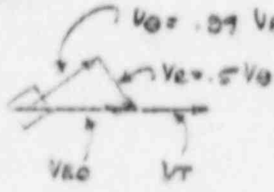
Franklin Research Center A Division of The Franklin Institute The Benjamin Franklin Parkway, Phila. Pa. 19103	Project [illegible]		Page A-2	
	By D.T.B.	Date 11/22/83	Ch'k'd Date	Rev. Date

Title: Oyster Creek Structural Review, Wind speeds corresponding to APC

__Atmospheric "Pressure Change" (APC)__

$+ \longrightarrow V_T$

CENTER OF VORTEX



"WIND FLOW MODEL"

VMAX = MAXIMUM HORIZONTAL ($V_{MAX} = V_{RO} + V_T$)

V_T = TRANSLATIONAL (SEE REG GUIDE 1.76

V_{RO} = ROTATIONAL

V_θ = TANGENTIAL

V_R = RADIAL

NOW $\Delta P = \frac{.00512}{144\ IN^2} V_\theta^2$

$= \frac{.00512}{144\ IN^2} (.89 (V_{MAX} - 5))^2$

$= 2.816 \times 10^{-5} (V_{MAX} - 5)^2 \quad \Rightarrow \quad V_{MAX} = \sqrt{35507 \Delta P} + 5\ MPH$

Summary of Results (Sheet 3(a) of Attachment 1)

REACTOR BUILDING		
METAL SIDING	.53 PSI	142 MPH
ROOF DECKING	.68	160
STEEL (ENCL.)	.68	160
CONTROL ROOM		
NORTH WALL	.53 PSI	142 MPH
REMAINDER	2.0	271 MPH

FORM CS-FRC-61

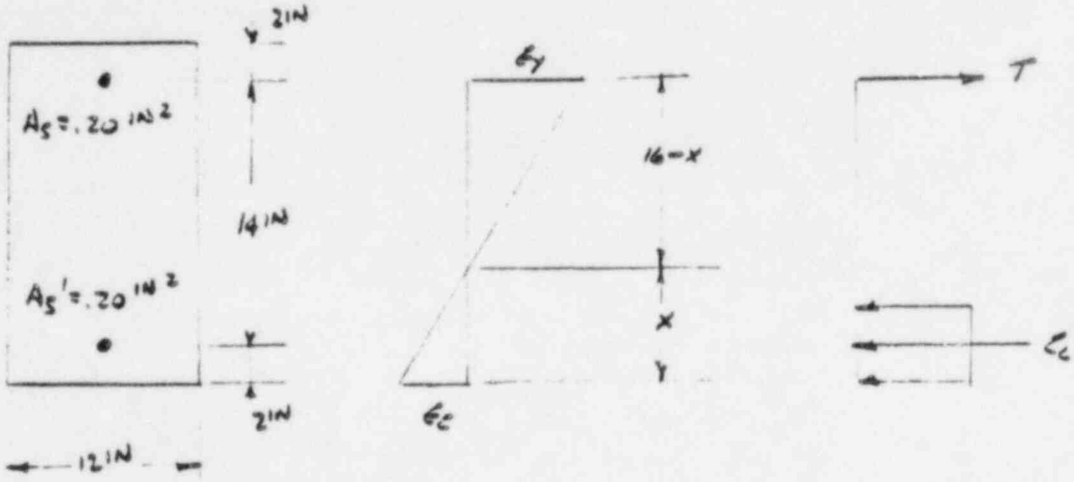
Franklin Research Center A Division of The Franklin Institute The Benjamin Franklin Parkway, Phila., Pa. 19103	Project 504-5506-001		Page A-3
	By DJB Date 11/22/83	Ch'k'd Date	Rev. Date

Title CONTROL ROOM, SOUTH WALL, OYSTER CREEK

SOUTH WALL (SHEET 19 OF ATTACHMENT #1)

DIFFERENTIAL PRESSURE ACTING OF CEILING SLAB MAY RELIEVE AXIAL LOAD IN WALL

EXAMINE FOR NO AXIAL LOAD AND SINGLE YIELD CASE,



$C = (.85)(3^{KSI})(.85X)(12) = 26.01X$

$T = (.20IN^2)(40^{KSI}) = 8$

$\epsilon_S = \frac{(2-X)}{16-X}\epsilon_y \Rightarrow f_S = \frac{40(2-X)}{(16-X)} \Rightarrow T' = A_S' f_S = (.20)(40)\frac{(2-X)}{(16-X)}$

FOR EQUILIBRIUM $T + T' = C$

OR $8 + \frac{8(2-X)}{(16-X)} = 26.01X$

$128 - 8X + 16 - 8X = 416.16X - 26.01X^2$

$26.01X^2 - 432.16X + 144 = 0$

$X = \frac{432.16 \pm \sqrt{186762 - 14982}}{52.02}$

$= 8.31 \pm 7.97 \Rightarrow X = .340\ IN \quad a = .289\ IN$

FORM CS-FRC-81

Franklin Research Center A Division of The Franklin Institute The Benjamin Franklin Parkway, Phila., Pa. 19103	Project 504 5306 - 001				Page A-4	
	By DJ.B	Date 11/22/83	Ch'k'd	Date	Rev.	Date

Title: CONTROL ROOM, SOUTH WALL, OYSTER CREEK

SOUTH WALL (CONT)

$$T' = (8)\frac{(2-.34)}{(16-.34)} = .848^K$$

$$C_c = (26.01)(.34) = 8.84^K$$

$\therefore$ MOMENT CAPACITY $= (8^K)\left(16 - \frac{.289}{2}\right) + (.848^K)\left(2 - \frac{.289}{2}\right)$

$= 126.8 + 1.57$

$= 128.4^{K\text{-}IN}$ OR $10.70^{K\cdot FT}$

ALLOWABLE PRESSURE $\Rightarrow \frac{8M}{\ell^2} = .296\ K/FT \Rightarrow 296^{PSF}$ (2.05^{PSI})

FORM CS-FRC-81

Title: START UP TRANSFORMER, OYSTER CREEK

SLIDING RESISTANCE

NO CONNECTIONS, RELIES ON STATIC FRICTION

TOTAL HORIZONTAL FORCE = 12,240 # (SEE SHEET 12 OF ATTACHMENT 1)

DEAD WEIGHT = 57750 #

FACTOR OF SAFETY = 1.1 (SRP 3.8.5)

$$\therefore \text{REQUIRED } \mu_s = \frac{(1.1)(12240\#)}{57750\#}$$

$= .233$ STEEL ON STONE MIN = .30

APPENDIX B

VENTILATION STACK DESIGN REVIEW CALCULATIONS

FRANKLIN RESEARCH CENTER
DIVISION OF ARVIN/CALSPAN

20th and Race Streets, Phila., Pa. 19103 (215) 448-1000

Title VENT STACK - OYSTER CREEK

CALCULATION OF THE MAXIMUM TORNADO WIND SPEED THAT THE REINFORCED CONCRETE VENT STACK AT OYSTER CREEK CAN WITHSTAND.

THE CALCULATIONS ARE PERFORMED USING TWO CRITERIA:

A) CALCULATIONS BASED ON THE STACK VERTICAL REINFORCEMENT.
(THESE CALC.'S WERE PERFORMED BY BURNS & ROE W.O. NO. 3431-85 CALC. NO. 6.85.05)

B) CALCULATIONS BASED ON THE CIRCUMFERENTIAL REINFORCEMENT.

CALCULATIONS FOR THE LATTER CRITERION ARE DETAILED HEREAFTER

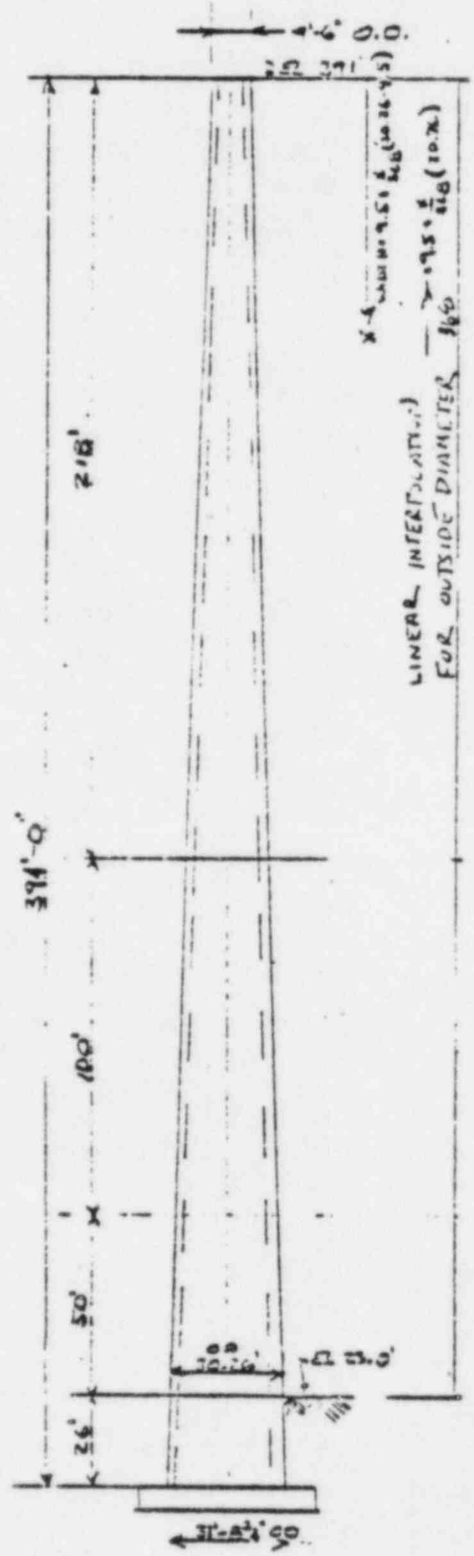
NOTE: THERMAL STRESSES ARE NOT ACCOUNTED FOR IN THESE CALCULATIONS.

Title: REINFORCED CONCRETE VENTILATION STACK, OYSTER CREEK

REFERENCE DOCUMENTS

1. EXTENSION OF WIND LOAD ON STRUCTURE - STACK
   JOB 4009-1 TASK 1-38
   RALPH M. PARSONS CO. JULY 24, 1967
   (ATTACHMENT #6 OF REFERENCE 4)

2. SEP STACK CHECK
   NO 6-85-05 SHEET #1 THRU 27
   BURNS AND ROE INC.
   (ATTACHMENT #3 OF REFERENCE 4)



OYSTER CREEK NUCLEAR GENERATING STATION
SEP TOPIC III-2, 'HIGH WIND AND TORNADO LOADS'
CONCRETE VENTILATION STACK

*** B A S I C G E O M E T R Y T A B L E #1 ***

ELEVATION (FT)	OUTSIDE DIAMETER (FT)	WALL THICKNESS (IN)	CONCRETE AREA (FT**2)	AREA OF INERTIA (FT**4)	REINFORCEMENT #/BAR	#/BAR	STEEL AREA (IN**2)	STEEL RATIO
278.00	15.87	6.0	23.86	713.7	34/ 7	33/ 7	40.27	.01172
248.00	17.57	7.0	31.13	1124.1				
218.00	19.26	8.0	38.72	1685.0	37/ 6	36/ 6	32.27	.00579
188.00	20.95	9.0	47.60	2430.9				
158.00	22.64	10.0	56.89	3398.4	46/ 5	46/ 5	28.24	.00345
113.00	25.18	12.0	75.76	5561.2	48/ 5	47/ 5	29.17	.00267
75.00	27.33	13.0	88.94	7705.2	53/ 7	53/ 6	55.28	.00432
68.00	27.72	19.0	130.01	11142.3				
60.00	28.17	25.0	170.74	14616.2				
23.00	30.26	15.0	113.47	12006.6	54/ 7	54/ 7	64.91	.00397
23.00	30.26	21.0	156.29	15985.4	54/ 7	54/ 7	64.91	.00288
23.00	30.26	21.0	156.29	15985.4	54/ 7	54/ 7	64.91	.00288
-3.00	31.73	18.0	141.23	16313.0	84/11	76/ 7	176.88	.00870

HEIGHT ABOVE BASE 394.00 (FT)
TOP ELEVATION 391.00 (FT)
GRADE ELEVATION 23.00 (FT)

O.D. AT CHIMNEY TOP 9.50 (FT)
I.D. AT CHIMNEY TOP 8.50 (FT)

FLUE OPENINGS

ELEVATION (FT)	#	BETA (DEG)	GAMMA (DEG)
23.00	1	4.94	0.00
23.00	1	4.94	0.00

Franklin Research Center A Division of The Franklin Institute The Benjamin Franklin Parkway, Phila., Pa. 19103	Project 504-5706-001				Page 8-4	
	By DJB	Date 11/21/83	Ch'k'd	Date	Rev.	Date

Title: Longitudinal Stress Analysis, Basic Geometry, V.S. Oyster Creek

OYSTER CREEK NUCLEAR GENERATING STATION
SEP TOPIC III-2, 'HIGH WIND AND TORNADO LOADS'
CONCRETE VENTILATION STACK

*** B A S I C G E O M E T R Y T A B L E #2 ***

ELEVATION (FT)	SECTION VOLUME (FT**2)	EXPOSURE AREA (FT**2)	SECTION ARM (FT)	MOMENT AREA (FT**3)
278.00	2162.84	1433.41	51.77	74209.17
248.00	826.98	501.60	14.75	124607.82
218.00	1048.97	552.45	14.77	190817.97
188.00	1295.95	603.15	14.79	274362.12
158.00	1568.17	653.85	14.81	376761.27
113.00	2984.38	1075.95	22.10	569042.00
75.00	3137.11	997.69	18.74	770914.79
68.00	767.88	192.68	3.49	812314.23
60.00	1203.09	223.56	3.99	861292.23
23.00	5313.38	1080.96	18.28	1111721.70
23.00	0.00	0.00	0.00	1111721.70
23.00	0.00	0.00	0.00	1111721.70
-3.00	3895.42	805.87	12.90	1312312.60

CONCRETE

COMPRESSIVE STRENGTH	5000.	(PSI)
MODULUS OF ELASTICITY	4287.	(KSI)
STRESS BLOCK PARAMETER	.800	
UNIT WEIGHT	150.	(PCF)

STEEL

YIELD STRENGTH	60000.	(PSI)
MODULUS OF ELASTICITY	29000.	(KSI)
YIELD STRAIN	.00207	
MODULAR RATIO	6.76	
STRENGTH RATIO	12.00	

OYSTER CREEK NUCLEAR GENERATING STATION
SEP TOPIC III-2, 'HIGH WIND AND TORNADO LOADS'
CONCRETE VENTILATION STACK

*** B A S I C   L O A D I N G   T A B L E ***

ELEVATION (FT)	DEAD LOAD (KIPS)	SERVICE LOADS (KIPS)	TOTAL LOAD (KIPS)	WIND[1] PRESSURE (PSF)	WIND LOAD (KIPS)	MOMENT (K*FT)	SHEAR FORCE (KIPS)	SHEAR STRESS (PSI)
278.00	324.43	13.00	337.43	82.9	83.22	4308.6	83.22	24.22
248.00	124.05	0.00	461.47	82.9	29.12	7234.8	112.35	25.06
218.00	157.35	8.00	626.82	82.9	32.08	11079.0	144.42	25.90
188.00	194.39	0.00	821.21	82.9	35.02	15929.7	179.44	26.18
158.00	235.23	6.00	1062.44	82.9	37.96	21875.1	217.41	26.54
113.00	447.66	6.00	1516.09	82.9	62.47	33039.0	279.88	25.65
75.00	470.57	5.00	1991.66	82.9	57.93	44759.9	337.80	26.37
68.00	115.18	0.00	2106.84	82.9	11.19	47163.6	348.99	18.64
60.00	180.46	0.00	2287.30	82.9	12.98	50007.3	361.97	14.72
23.00	797.01	77.00	3161.31	82.9	62.76	64547.4	424.73	25.99
23.00	0.00	0.00	3161.31	82.9	0.00	64547.4	424.73	18.87
23.00	0.00	0.00	3161.31	82.9	0.00	64547.4	424.73	18.87
-3.00	584.31	174.00	3919.62	0.0	0.00	75590.5	424.73	20.88

(1) BASED ON 100 mph WINDSPEED

*** S T R E N G T H   T A B L E ***

ELEVATION (FT)	MOMENT CAPACITY (KIP*FT)	NEUTRAL AXIS (DEG)	STEEL STRAIN	CONCRETE STRAIN	TORNADO PRESSURE (PSF)	TORNADO WINDSPEED (MPH)	
278.00	20654.2	27.9	0.04878	0.00300	397.6	394.1	CONCRETE
218.00	23572.1	18.6	0.07000	0.00188	176.5	262.6	STEEL 7%
158.00	29847.8	14.5	0.07000	0.00114	113.2	210.3	STEEL 7%
113.00	39243.0	13.2	0.07000	0.00094	98.5	196.2	STEEL 7%
75.00	68938.6	17.5	0.07000	0.00165	127.7	223.4	STEEL 7%
23.00	99702.3	23.1	0.07000	0.00293	128.1	223.7	STEEL 7%
23.00	99914.1	13.7	0.07000	0.00102	128.4	223.9	STEEL 7%
23.00	98748.7	18.8	0.07000	0.00191	126.9	222.6	STEEL 7%
-3.00	214321.3	26.6	0.05349	0.00300	233.3	301.9	CONCRETE

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

Title: Longitudinal Stress Analysis, Capacity of Section, V.S. Oyster Creek

Project: 504-5536-001

By: DJB Date: 11/21/83

Ch'k'd: Date:

Page: 13-5

Rev.: Date:

B. CALCULATION OF THE MAXIMUM WIND PRESSURE BASED ON THE CIRCUMFERENTIAL STRESSES IN THE STACK

TWO APPROACHES ARE CONSIDERED HEREUNDER :-

I. USING THE WORKING STRESS METHOD GIVEN IN SECTION 4.7 OF THE ACI. 307-79 CODE, WHILE APPLYING LARGER ALLOWABLE STRESSES COMPARABLE TO THE THOSE USED IN THE ULTIMATE STRENGTH METHOD.

II. APPLYING THE ULTIMATE STRENGTH METHOD TO A UNIT HEIGHT OF THE STACK SHELL USING BEAM FORMULA, AS WELL AS THE MOMENT EQUATIONS GIVEN ON PAGE 50 OF THE DERIVATION OF THE EQUATIONS IN THE ACI 307-79 CODE.

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Title VENT STACK - OYSTER CREEK

I - USING THE ACI 307-79 CODE, THE CIRCUMFERENTIAL STRESSES ARE CALCULATED BY APPLYING THE FOLLOWING FOUR EQUATIONS, AS APPLIED TO WIND LOADING :

(1) MAXIMUM CIRCUMFERENTIAL STRESS IN psi IN THE CONCRETE AT THE INNER SURFACE OF THE SHELL (EQ. 147)

$$f''_{cwc} = \frac{Wr^2}{106t^2}\left[\frac{k'}{(k')^3 + 3np'\left[(z'+k'-1)^2c' + (z-k')^2\right]}\right]$$

WHERE

W = WIND PRESSURE, psf

r = MEAN RADIUS OF STACK SHELL AT SECTION UNDER CONSID. in

t = THICKNESS OF STACK SHELL AT THE SECTION, in

$n = \frac{E_s}{E_c}$

p' = RATIO OF CROSS-SEC. AREA OF THE CIRCUMFERENCE OUTSIDE FACE REINF. STEEL PER UNIT HEIGHT TO THE CROSS-SEC. AREA OF THE STACK SHELL PER UNIT HEIGHT

c' = $\frac{\text{INSIDE FACE CIRCUMFERENTIAL REINFORCING STEEL AREA}}{\text{OUTSIDE CIRCUMF. REINF. STEEL AREA}}$

z' = DISTANCE BETWEEN INNER SURFACE OF STACK SHELL & CIRCUMFERENTIAL OUTSIDE FACE REINF. STEEL ÷ TOTAL SHELL THICKNESS

z = DISTANCE BETWEEN INNER SURFACE OF STACK SHELL & OUTER FACE VERTICAL REINF. ÷ TOTAL SHELL THICKNESS

Title
VENT STACK - OYSTER CREEK

$$k' = -p'n(c'+1) + \sqrt{[p'n(c'+1)]^2 + 2p'n[z' + c'(1-z')]}$$

(2) MAXIMUM STRESS, IN psi IN THE EXTERIOR CIRCUMFERENTIAL REINFORCEMENT (EQ. 148)

$$f_{swc} = n\, f''_{cwc}\left[\frac{z'}{k'} - 1\right]$$

(3) MAXIMUM CIRCUMFERENTIAL STRESS, IN psi, IN THE CONCRETE AT THE OUTER SURFACE OF THE SHELL (EQ. 247)

$$f_{cwc} = \frac{Wr^2}{92t^2}\left[\frac{k'_o}{(k'_o)^3 + 3np'\left[(z' + k'_o - 1)^2 + c'(z' - k'_o)^2\right]}\right]$$

WHERE $k'_o = -p'n(c'+1) + \sqrt{[p'n(c'+1)]^2 + 2p'n[c'z' + 1 - z']}$

(4) MAXIMUM STRESS, IN psi IN THE INTERIOR CIRCUMFERENTIAL REINFORCEMENT (EQ. 248)

$$f''_{swc} = n\, f_{cwc}\left[\frac{z'}{k'_o} - 1\right]$$

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AT ELEVATION 113 ft. (SECTION 4)

$t = 12''$

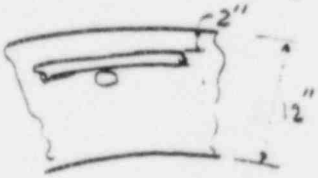
$r = \frac{OD - t}{2} = \frac{25.18 \times 12 - 12}{2} = 145.08$ in

$n = \frac{29,000}{4030} = 7.2$

$p' = \frac{0.2}{12 \times 6} = .002778$ (#4 @ 6")

$c' = 0$ (NO CIRCUMFERENTIAL REINF. ON INNER SURFACE)

$z' = \frac{12-2}{12} = 0.833$ (2" COVER CONSIDERED)



CALCULATION OF k' TO BE USED IN EQ'NS 147 & 148

$$k' = -.002778 \times 7.2(0+1) + \sqrt{[.002778 \times 7.2]^2 + 2 \times .002778 \times 7.2[.833]} = .1636$$

CALCULATION OF k_o' TO BE USED IN EQN'S 247 & 248

$$k_o' = -.002778 \times 7.2 + \sqrt{(.002778 \times 7.2)^2 + 2 \times .002778 \times 7.2(1-.833)} = .06414$$

$$f''_{cwc} = \frac{W \times 145.08^2}{106\,(12)^2}\left[\frac{.1636}{(.1636)^3 + 3 \times 7.2 \times .002778\left[0 + (0.833 - .1636)^2\right]}\right] = \underline{\underline{7.22\,W}} \text{ psi}$$

$$f_{swc} = 7.2 \times 7.22\,W\left[\frac{.833}{.1636} - 1\right] = \underline{\underline{212.7\,W}} \text{ psi}$$

$$f_{cwc} = \frac{W \times 145.08^2}{92 \times (12)^2}\left[\frac{0.06414}{(.06414)^3 + 3 \times 7.2 \times .002778\left[(.833 + .0644 - 1)^2 + 0\right]}\right]$$

$$= \underline{\underline{113.4\,W}} \text{ psi}$$

f''_{swc} IS NOT APPLICABLE, AS NO INTERIOR CIRCUM. REINF. IS USED.

THE ABOVE INDICATES THAT f_{cwc} & f_{swc} ARE TO BE CONSIDERED BECAUSE THEY ARE THE LARGER.

<u>ALLOWABLE STRESSES</u>.

FOR CONCRETE

$$f_{all} = 0.85\, f'_c = 0.85 \times 5000 = 4250 \text{ psi}$$

FOR STEEL

$$F_{all} = F_y = 60 \text{ ksi}$$

CALCULATION OF THE MAXIMUM WIND VELOCITY BASED ON THE CIRCUMFERENTIAL STRESSES IN THE STACK.

THE DYNAMIC WIND PRESSURE $W = C_L \frac{1}{2} \rho V^2$

WHERE

$C_L = 0.7$

$\rho = .00237 \quad lb\,sec^2/ft^4$

$W = 0.7 \times \frac{1}{2} \times .00237\, V^2$

$lb/ft^2 \qquad \frac{lb\,sec^2}{ft^4} \left(ft/sec\right)^2$

$W = 0.7 \times .00237\,(V \times 1.467)^2$ (V in mph)

$\therefore \quad V = \sqrt{\frac{W}{.7 \times .00255}} = 23.67\sqrt{W}$ mph

THE MAXIMUM CIRCUMFERENTIAL STRESS AT THE OUTER SURFACE OF THE CONCRETE

$f_{cwc} = f_{all}$

$113.4\, W = 4250$

$W = \frac{4250}{113.4} = 37.48 \quad lb/ft^2$

$\therefore V = 23.67\sqrt{37.48} = 145$ mph

Title VENT STACK - OYSTER CREEK

FOR THE MAX. STRESS IN THE EXTERIOR CIRCUMFERENTIAL REINFORCEMENT

$$f_{swc} = 52\ W = f_y = 60{,}000 \text{ psi}$$

$$\therefore W = \frac{60{,}000}{212.7} = 282 \ \text{lb/ft}^2$$

i.e. $V = 23.67\sqrt{282} = 397$ mph

$\therefore$ CONCRETE STRESS CONTROLS.

$\therefore$ MAX WIND VELOCITY @ 113 ft ELEV. IS 145 mph WHICH IS CONTROLLED BY THE STRESS IN THE CONCRETE AT THE OUTER SURFACE OF THE SHELL (f_{swc})

Title VENT STACK - OYSTER CREEK

AT ELEVATION 158 ft. (SECTION 3)

$t = 10''$

$r = 130.8$

$n = \frac{29000}{4030} = 7.2$

$p' = \frac{.11}{10 \times 6} = .001833$ (#3 @ 6")

$c' = 0$ (NO CIRCUM. REINF. ON INNER SURFACE)

$z' = \frac{10-2}{10} = 0.8$ (2" Cover Assumed)

$$k' = -.001833 \times 7.2 + \sqrt{(.001833 \times 7.2)^2 + 2 \times .001833 \times 7.2 \times 0.8} = .1327$$

$$k'_o = -.001833 \times 7.2 + \sqrt{(.001833 \times 7.2)^2 + 2 \times .001833 \times 7.2 (1-.8)} = .06065$$

$$f''_{cwc} = \frac{W \times 130.8^2}{106\,(10)^2}\left[\frac{.1327}{(.1327)^3 + 3 \times 7.2 \times .001833\,(.8 - .1327)^2}\right] = \underline{\underline{10.727\,W}} \text{ psi}$$

$$f_{swc} = 7.2 \times 10.727W\left[\frac{.8}{.1327} - 1\right] = \underline{\underline{371.9\ W}} \text{ psi}$$

$$f_{cwc} = \frac{W\,(130.8)^2}{92\,(10)^2}\left[\frac{.06065}{(.06065)^3 + 3 \times 7.2 \times .001833\,(.8 + .06065 - 1)^2}\right] = \underline{\underline{113.7\,W}} \text{ psi}$$

f''_{swc} IS NOT APPLICABLE, AS NO INTERIOR CIRCUM. REINF. IS USED

THE ABOVE INDICATES THAT f_{cwc} & f_{swc} ARE TO BE CONSIDERED BECAUSE THEY ARE THE LARGER.

THE MAX. CIRCUMFERENTIAL STRESS AT THE OUTER SURFACE OF THE CONCRETE

$$f_{cwc} = f_{all.}$$

$$113.7\,W = 4250$$

$$W = \frac{4250}{113.7} = 37.38 \quad lb/ft^2$$

$$\therefore \quad V = 23.67\sqrt{37.38} = \underline{\underline{144.7}} \quad mph$$

THE MAX. STRESS IN THE EXTERIOR CIRCUMFERENTIAL REINF.

$$f_{swc} = 371.9\,W = f_y = 60{,}000 \quad psi$$

$$\therefore \quad W = \frac{60{,}000}{371.9} = 161.3 \quad lb/ft^2$$

$$i.e. \quad V = 23.67\sqrt{161.3} = \underline{\underline{300}} \quad mph$$

∴ CONCRETE STRESS CONTROLS

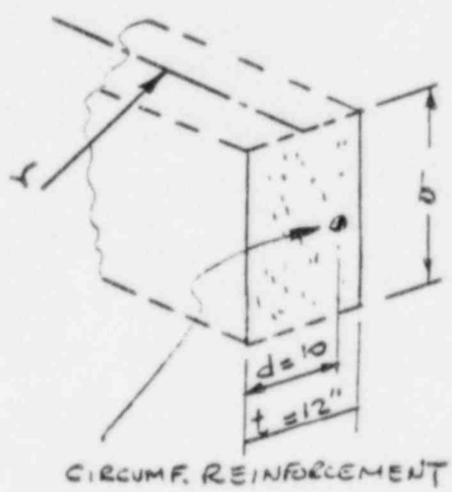
∴ MAX WIND VELOCITY @ 153 ft EL. IS 144.7 mph

II ULTIMATE STRENGTH METHOD APPROACH

USING THE RECTANGULAR BEAM FORMULA

THE ALLOWABLE MOMENT

$$M_u = A_s f_y \left(d - \frac{a}{2} \right) *$$



WHERE

$$a = \frac{A_s f_y}{0.85 f'_c b}$$

AT ELEVATION 113 ft (SECTION 5)

$r = 145.08$ in.

$A_s = .2$ in^2 (# 4 SPIRAL RING @ 6" PITCH)

$f_y = 60$ ksi

$f'_c = 5$ ksi

$b = 6$ in

$t = 12$ in

$$\therefore \; a = \frac{.2 \times 60}{.85 \times 5 \times 6} = 0.4706 \text{ in}$$

* PER STEEL SPACING

Title VENT STACK - OYSTER CREEK

FROM PAGE 50 OF THE ACI 307-79 FORMULAS FOR CIRCUMFERENTIAL MOMENTS DUE TO RADIAL WIND PRESSURE DISTRIBUTION ARE

Compression inside of shell, $M_i = 0.272\ pr^2$

Compression outside of shell, $M_o = 0.314\ pr^2$

p = Velocity wind pressure, psi

r = Mean radius, in.

M = Moment, lb.-in./in.

To use the wind pressures w (psf) in Table 4.2.1, set

$$w = 0.6p\ (144),\ \therefore\ p = \frac{w}{86.4}$$

where 0.6 is the drag coefficient for cylindrical structures and 144 is a conversion factor to convert psi to psf.

$$M_i = .003148wr^2$$

$$M_o = .003634wr^2$$

IN THIS ANALYSIS A DRAG COEFFICIENT OF 0.7 WILL BE USED GIVING:-

$$W = 0.7\ p\ (144)$$

$$\therefore\ p = \frac{W}{100.8}$$

$$M_i = 0.0026984\ Wr^2$$

$$M_o = 0.0031151\ Wr^2$$

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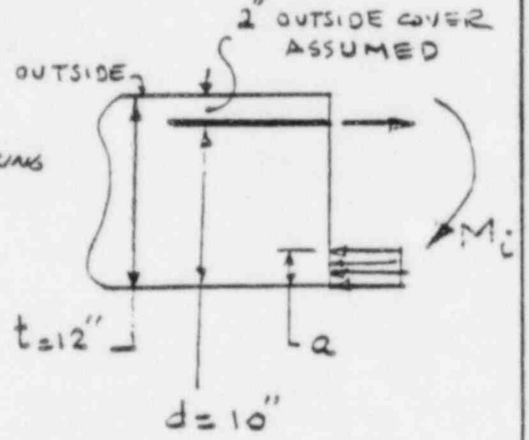
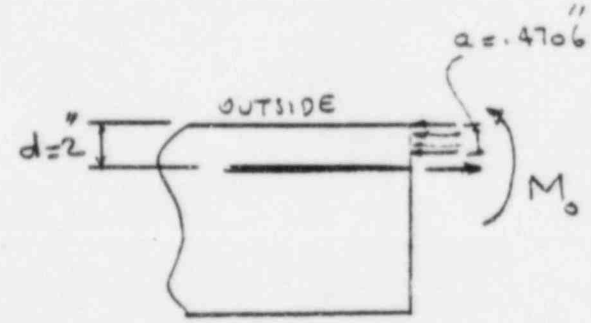
COMPRESSION ON THE INSIDE



$$M_{u_i} = A_s f_y \left(d - \frac{a}{2} \right) \quad \text{PER STEEL SPACING}$$

$$M_{u_i} = .2 \times 60{,}000 \left(10 - \frac{.4706}{2} \right)$$
$$= 117{,}176 \text{ in lb}/6^{\text{IN}}$$

COMPRESSION ON THE OUTSIDE



$$M_{u_o} = .2 \times 60{,}000 \left(2 - \frac{.4706}{2} \right)$$
$$= 21{,}176 \text{ in lb}/6^{\text{IN}}$$

THE LIMITING MOMENT IS M_{u_o}, AS IT IS CONSIDERABLY LESS THAN M_{u_i}

THEREFORE, M_{u_o} ONLY WILL BE CONSIDERED.

BUT $M_o = .314 p r^2$ (p IN psi)

EQUATING M_o & M_{u_o} WE GET

$$p = \frac{M_{u_o}(144)}{.314 r^2} \quad (p \text{ IN psf})$$

$$= \frac{(21176/6)(144)}{(.314)(145.58)^2} = 76.1 \Rightarrow V = \sqrt{\frac{76.1}{.00256}} = 172 \text{ MPH}$$

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Title VENT STACK - OYSTER CREEK

CALCULATION OF THE LIMITING WIND SPEED, USING THE ULTIMATE STRENGTH METHOD APPROACH, BASED ON THE CIRCUMFERENTIAL STRESSES.

EL. ft	SECTION	A_s in^2	b in	d in	a* in	M_{uo}** in-lb/in	r in	V† mph
278	1	.11 (#3 @ 6")	6	2	.2588	2053	92.2	209
218	2	.11 (#3 @ 6")	6	2	.2588	2053	111.6	172
158	3	.11 (#3 @ $5\frac{1}{2}$")	$5\frac{1}{2}$	2	.2824	2231	130.8	$\boxed{153}$
113	4	.2 (#4 @ 6")	6	2	.4706	3529	145.08	173
75	5	.2 (#4 @ 6")	6	2	.4706	3529	157.5	160

(CONTINUED ON PAGE B-28)

$$* \quad a = \frac{A_s f_y}{.85 f'_c b} = \frac{60}{.85 \times 5} \frac{A_s}{b} = 14.118 \frac{A_s}{b}$$

$$** \quad M_{u_o} = A_s f_y \left(d - \frac{a}{2} \right) / b \quad \text{FOR UNITS (\#-IN/IN)}$$

$$\dagger \quad p = \frac{M_{uo}(144)}{.314 R^2} \quad \text{FOR } p \text{ -- psf AND } V = \sqrt{\frac{p}{.00256}} \Rightarrow V = \frac{423.2}{R} \sqrt{M_{uo}}$$

FORM 207-10M-4-74-CP

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Title: OYSTER CREEK VENTILATION STACK, ACROSS WIND VIBRATIONS

VORTEX SHEDDING ANALYSIS

CHIMNEY IS IN RESONANCE WHEN THE FREQUENCY OF VORTEX SHEDDING COINCIDES WITH A NATURAL FREQUENCY OF THE STRUCTURE.

$f = \frac{\omega}{2\pi} = \frac{SV}{D_c}$ S: STROUHAL NUMBER (SEE 8-21)

V: VELOCITY (fps)

D_c: CRITICAL DIAMETER (ft)

TAKE AVG. DIAMETER OF UPPER 1/3

EL 391' OD = 9.56'

EL 270' 15.37'
EL 268' x ⟹ 16.4' O.D.
EL 245' 17.57'

∴ AVG DIAMETER = 13'

WINDSPEEDS CORRESPONDING TO RESONANCE ($V = fD_c/S$)

MODE	FREQUENCY (CPS)	WINDSPEED (fps)	WINDSPEED (mph)
1	.558	29.0	19.3
2	1.706	88.7	60.5
3	3.757	195.5	133.3
4	5.552	288.9	197

FORM CS-FRC-81

5.7.3 Stack

As-computed half-space damping ratios were used for the two stack SSI analyses. Case S1 used a shear modulus of 1500 ksf and a stiffness comprising the half-space stiffness plus the stiffness due to embedment, as calculated by Novak's method. Case S2 used the Fork River shear modulus G and as-computed modal damping ratios.

Computed values of natural periods and associated composite modal damping ratios for Cases S1 and S2 are shown in Table 14.

TABLE 14. Ventilation stack natural frequencies and modal damping values.

Mode number	Case S1 Period (s)		Case S1 Composite modal damping ratio	Case S2 Period (s)	Case S2 Composite modal damping ratio
1	1.791	$f_1 = .558$	0.094	1.791	0.094
2	0.586	⇒ $f_2 = 1.706$	.097	0.586	.097
3	.266	$f_3 = 3.759$	.110	.266	.110
4	.180	$f_4 = 5.556$	.472	.179	.472
5	.155	$f_5 = 6.452$	.150	.155	.150
6	.112	Hz	.204	.112	.204
7	.085		.133	.085	.134
8	.080		.157	.080	.157
9	.063		.106	.062	.106
10	.048		.101	.048	.101
11	.043		.112	.043	.112
12	.038		.100	.038	.100
13	.031		.100	.031	.100
14	0.029		0.105	0.029	0.105

FROM OYSTER CREEK SEISMIC REVIEW NUREG/CR-1981, UCRL-53018

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Title: OCVS, STROUHAL NUMBER

Strouhal Number

The variation of the Strouhal Number with Reynolds Number and surface roughness (1, 2) is shown in figure 1. Depending on surface roughness, regular shedding can occur over virtually all Reynolds Numbers although for smooth cylinders there is a range in which the shedding is aperiodic and its influence very minor. If roughness is to be ignored the Strouhal Number can be approximated as being 0.19 for $Re < 10^5$ and 0.25 for $Re > 10^6$.

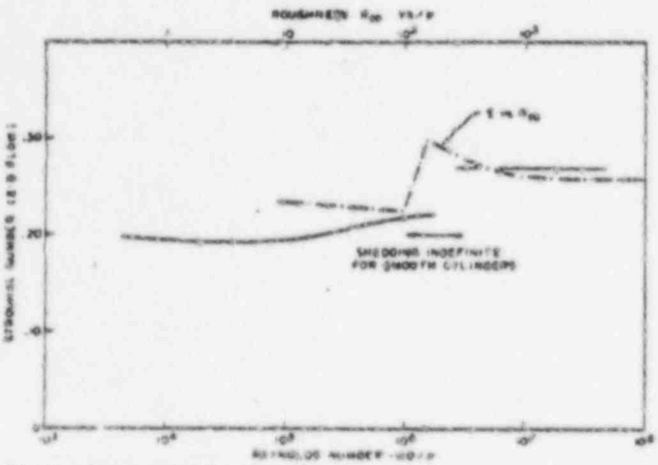

Fig. 1: Variation of Strouhal Number with *Re*

⇒ STROUHAL NUMBER (S)

IS A FUNCTION OF R_e

$$R_e = \frac{Vh}{\nu}$$

WHERE

ν : KINEMATIC VISCOSITY

h : CHARACTERISTIC DIMENSION

FROM REFERENCE

$R_e = 6390\ Vh$ V : IN fps, h : FT

TO FIND THE TRANSITION WINDSPEED TAKE $h \approx 13'$ (AVG. O.D. OF UPPER 1/3)

$\therefore 10^6 = (6390)\,V(13) \Rightarrow V = 17.06$ FPS

$= 8$ MPH

$\therefore$ FOR TORNADO ANALYSIS TAKE $S = .25$

FORM CS-FRC-81

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Title
OCVS, DYNAMIC LOAD FACTOR

MODE	RESONANT VELOCITY (mph)	STATIC APPLICATION * (psf)
3	133.3	45.4
4	197	99.4

CAPACITY OF STACK

OYSTER CREEK STRUCTURAL REVIEW (MSD) = 180 MPH (82.9 PSF)

FRC REVIEW (USD) = 196 MPH (98.5 PSF)

DYNAMIC LOAD FACTOR REQUIRED TO BRING RESONANT CONDITION UP TO STATIC CAPACITY

$$DLF = \frac{82.9}{45.4} = 1.83$$

* DYNAMIC PRESSURE BASED ON THE STATIC APPLICATION OF RESONANT WINDSPEED

FORM CS-FRC-81

OVALLING ANALYSIS

IF THE PERIOD OF A CIRCULAR RING COINCIDES WITH HALF OF THE PERIOD OF THE SHEDDING OF VORTICES, THEN OVALLING OF THE CROSS SECTION COULD OCCUR. THE VELOCITY THAT COULD CAUSE OVALLING VIBRATIONS IN THE FUNDAMENTAL MODE IS GIVEN BY

$$V = 366\,(t/R)$$

$$= 732\,(t/D)$$

WHERE V : IS WIND SPEED IN MPH

t : WALL THICKNESS (IN)

D : DIAMETER OF SECTION (FT)

ELEVATION (FT)	O.D. (FT)	THICKNESS (IN)	M.D. (FT)	VELOCITY TO CAUSE OVALLING (MPH)
341	9.5	6	9.0	488
275	15.87	6	15.37	286
248	17.57	8	16.90	347
218	19.26	8	18.59	315
188	20.95	9	20.2	326
158	22.64	10	21.81	335

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Title: OCVS OVERALL STRUCTURAL EFFECT OF MISSILE IMPACT

EQUIVALENT STATIC LOAD FOR UTILITY POLE IMPACT = 43.2K

(SEE SHEET 31 OF ATTACHMENT #1) *

IMPACT ELEVATION = 30'

∴ EQUIVALENT STATIC MOMENT AT BASE = $43.2^K \times 30' = 1296^{K-FT}$

APPROXIMATE CAPACITY OF SECTION (BASE) = 100000^{K-FT} (SEE PAGE B-5)

∴ REDUCTION FOR RESISTANCE DUE TO MISSILE IMPACT IS NEGLIGIBLE

* CALCULATIONS PREDICT THAT A SECTION OF STACK WILL NOT BE "KNOCKED" OUT BY IMPACT. ALSO, THIS CAPACITY WAS BASED ON CIRCUMFERENTIAL STEEL AREA. (SINCE IMPACT YIELD AREA IS ASSUMED CIRCULAR SOME RESISTANCE IS SOUGHT FROM LONGITUDINAL REBAR).

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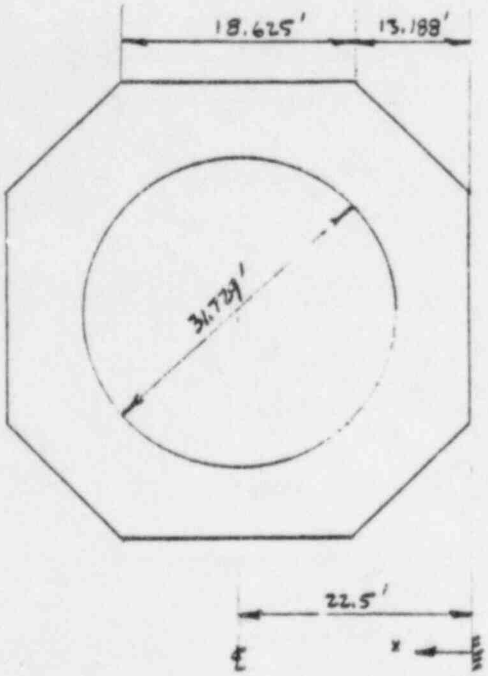
By DJB	Date 2/29/84	Chk'd [illegible]	Date 5/1/84	Rev. Date

Title: CAPACITY OF CHIMNEY FOUNDATION



WIDTH FUNCTION

AT $X=0$ $w=18.625'$

AT $X=13.188'$ $w=45'$

$$\Rightarrow w(x) = 18.625' + \frac{(26.375)}{13.188}x$$

$$= 18.625' + 2x$$

PRESSURE FUNCTION

$$P_{MAX} = \frac{W}{A}K_1 \Rightarrow p(x) = P_{MAX}\left(1 - \frac{x}{RK_2}\right)$$

$$= P_{MAX}\left(1 - \frac{x}{23.475K_2}\right)$$

BURNS AND ROE DRWG # 4016-4
"CHIMNEY FOUNDATION"

SHEAR

$$dV = w(x)p(x)dx = (18.625 + 2x)P_{MAX}\left(1 - \frac{.0427x}{K_2}\right)dx$$

$$\therefore V(x) = \int_0^x P_{MAX}\left(18.625 - \frac{.795x}{K_2} + 2x - \frac{.0854x^2}{K_2}\right)dx$$

$$= P_{MAX}\left(18.625x + \left(1 - \frac{.3975}{K_2}\right)x^2 - \frac{.02847x^3}{K_2}\right)$$

MOMENT

$$dM = V\,dx$$

$$\therefore M(x) = \int_0^x P_{MAX}\left(18.625x - \frac{.3975}{K_2}x^2 + 2x^2 - \frac{.02847}{K_2}x^3\right)dx$$

$$= P_{MAX}\left(9.3125x^2 - \frac{.1325}{K_2}x^3 + \frac{x^3}{3} - \frac{.00712}{K_2}x^4\right)$$

Title: CAPACITY OF CHIMNEY FOUNDATION

EXAMINE CASE (1) FROM PAGE 27, ATTACHMENT 3 OF TEXT REFERENCE #5

(BURNS AND ROE CALCULATION # 6.35.05 SHEET 27)

$K_2 = .84$ AND $P_{MAX} = 18.433$ KSF

$\therefore V(x) = (18.433)(18.625x + .527x^2 - .0339x^3)$ (KIPS)

$M(x) = (18.433)(9.3125x^2 + .176x^3 - .0085x^4)$ (K-FT)

AT BASE OF STACK $x = 22.5' - \frac{31.725'}{2} = 6.64'$

$\therefore W(x) = 31.297$

$V(x) = (18.433)(123.6 + 23.2 - 9.9) = 2523$ K

$M(x) = (18.433)(410.1 + 51.5 - 16.5) = 8205$ K-FT

Title: CAPACITY OF CHIMNEY FOUNDATION

"STACK FOUNDATION SECTION"

TRY QUICK CHECK FOR CONCRETE RESISTANCE ACTING ALONE

$AREA = (31.9')(7') = 223.3 \square'$

$INERTIA = \frac{1}{12}(31.9')(7')^3 = 911.3\ FT^4$

STRESSES:

$$\sigma_V = \frac{2523^K \times 1000^{\#}}{223.3 FT^2 \times 144^{IN^2}} \times 1.5 = 118^{PSI}$$

$$\sigma_T = \frac{(8205^{K\text{-}FT} \times 1000^{\#/K} \times 12^{IN/FT})(3.5' \times 12^{IN})}{(911.3\ FT^4)(12^{IN/FT})^4} = 219^{PSI}$$

ALLOWABLE STRESSES:

FOR $f_c' = 3000^{PSI}$

$$F_V = 2\phi\sqrt{f_c'} = 110^{PSI} \cong 118^{PSI}$$

$$F_T = 5\phi\sqrt{f_c'} = 274^{PSI} > 219^{PSI}$$

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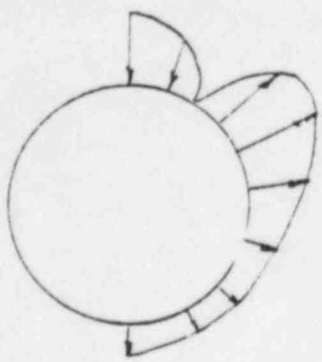
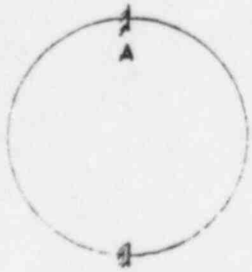
Title: CIRCUMFERENTIAL STRESS ANALYSIS



RADIAL PRESSURE LOADS

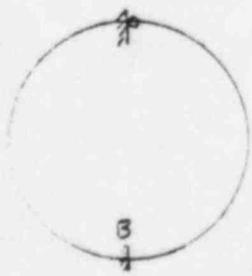
FROM A. ROSHKO, JOURNAL OF FLUID MECHANICS, MAY 1961



INTACT SECTION

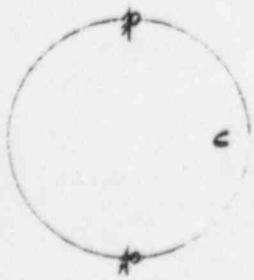
FIXED-FIXED MODEL $\Rightarrow$ $M_A = .357\ pBR^2$ (1)

WHERE "+" INDICATES COMPRESSION ON OUTSIDE SURFACE OF SHELL.



UPSTREAM CRACK

HINGE-FIXED MODEL $\Rightarrow$ $M_B = .633\ pBR^2$



UPSTREAM AND DOWNSTREAM CRACK

HINGE-HINGE MODEL $\Rightarrow$ $M_C = -.91\ pBR^2$

WHERE p = STAGNATION PRESSURE
B = HOOP REIN. SPACING
R = CHIMNEY RADIUS

(1) FROM ERDEI AND GHOSH, CONCRETE, 1967, "THE EFFECTS OF WIND ON LARGE DIAMETER CHIMNEYS AND SHAFTS". ACTUAL EQUATION USED IN REVIEW IS $M_A = .314\ pBR^2$ AS PER ACI 307-79.

NOTE: SUBSEQUENT EQUATIONS BASED ON CONSERVATIVE MODELS THAT ASSUME NO LONGITUDINAL TRANSFER OF SHEAR FORCE.

FORM CS-FRC-81

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By	Date	Ch'k'd	Date	Rev.	Date
DJB	2/24/84	[illegible]	5/1/84		

Title: CIRCUMFERENTIAL CAPACITY FOR HINGE-HINGE MODEL

CIRCUMFERENTIAL CAPACITY TABLE, HINGE-HINGE MODEL

ELEVATION (FT)	A_s (IN^2)	b (IN)	$a^{(1)}$ (IN)	t (IN)	d (IN)	$M_{ui}^{(2)}$ (#-IN)	R (IN)	$M_o^{(3)}$ (#-IN)	$p^{(4)}$ (PSI)
278	.11	6	.259	6.0	4.0	25545	92.2	46415P	.550
218	.11	6	.259	8.0	6.0	38745	111.6	68000P	.570
158	.11	5.5	.282	10.0	8.0	51869	130.8	85629P	.606
113	.20	6	.471	12.0	10.0	117170	145.06	114923P	1.019
75	.20	6	.471	13.0	11.0	129170	157.5	135442P	.954

(1) $a = \frac{A_s f_y}{.85 f_c' b} = 14.118 \frac{A_s}{6}$ WITH $f_y = 60{,}000$ PSI AND $f_c' = 5000$ PSI

(2) $M_{ui} = A_s f_y (d - a/2)$

(3) $M_o = -.91\, p\, B R^2$ WHERE -.91 IS THE MAXIMUM COEFFICENT FROM HINGE-HINGE MODEL

(4) $M_o = M_{ui}$ SOLVED FOR p NOTE: ASSUMES HINGES HAVE FORMED CHECK WITH INTACT SECTION SOLUTION

FOR FIRST FAILURE p B-18

EZ 278 $p = .550$ PSI $= 79.2$ PSF $\Rightarrow V = 176$ MPH — $V = 208$ MPH ∴ CRACK AT C DOES NOT FORM

EZ 218 $p = .570$ PSI $= 82.1$ PSF $\Rightarrow V = 179.1$ MPH — $V = 172$ MPH CRACK FORMS

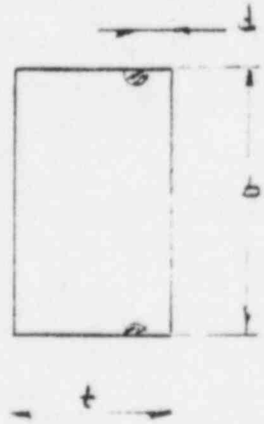
EZ 158 $p = .606$ PSI $= 87.3$ PSF $\Rightarrow V = 185$ MPH — $V = 153$ MPH CRACK FORMS

(LONGITUDINAL STRENGTH: USD = 196 MPH, MSD = 180 MPH)

FORM CS-FP 1-81

Title: SHEAR STRESS CAPACITY OF CRACKED SECTIONS

EXAMINE ABILITY OF CRACKED SECTION TO TRANSMIT SHEAR



"CIRCUMFERENTIAL SECTION"

- ASSUME OUTSIDE COVER IS DESTROYED

ELEVATION (FT)	d (IN)	t (IN)	t' (IN)	b (IN)	As (IN²)	v[1] (PSI)	v[2] (PSI)	v_{ACT}[3] (PSI)
278	2	6	4	6	.11	193	157	26
218	2	8	6	6	.11	128	106	26
158	2	10	8	5.5	.11	105	87	27
113	2	12	10	6	.20	140	115	26
75	2	13	11	6	.20	127	105	26

(1) TRANSMITTABLE SHEAR BASED ON SHEAR FRICTION

$$v = \frac{\mu A_s f_y}{A_G} = \frac{\mu A_s f_y}{bt'} = 42000 \frac{A_s}{bt'}$$ WITH $\mu = .70$, $f_y = 60{,}000$ PSI

(2) TRANSMITTABLE SHEAR BASED ON STEEL SHEAR CAPACITY

$$v = \frac{A_s f_y}{\sqrt{3} A_G} = .577 \frac{f_y A_s}{bt'} = 34641 \frac{A_s}{bt'}$$ WITH $f_y = 60$ KSI

(3) AVERAGE SHEAR STRESS AS PER PAGE B-5 FOR 120 MPH WINDSPEED

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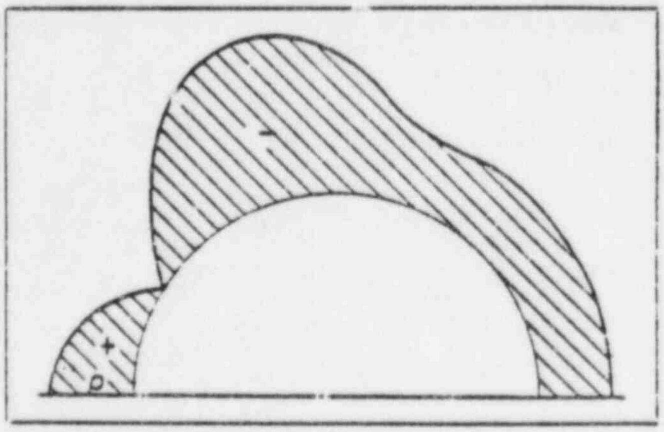
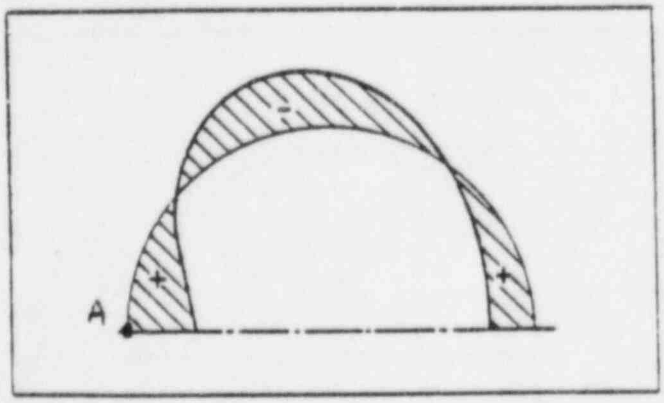
Title CONCRETE CHIMNEY, CIRCUMFERENTIAL STRESS ANALYSIS, PRE-CRACKED

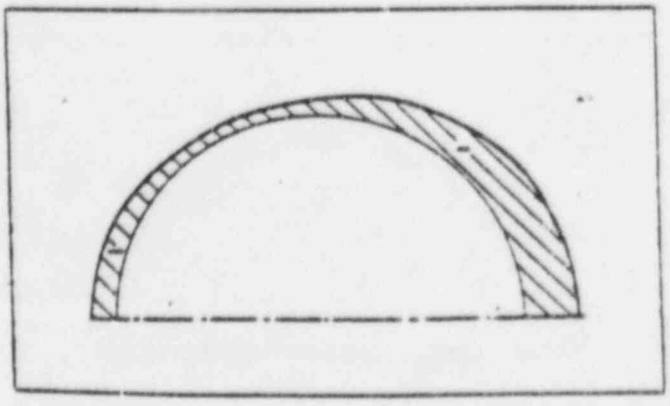

Pressure distribution according to Roshko.

RADIAL PRESSURE LOADS
FROM
A. ROSHKO, JOURNAL OF FLUID MECHANICS
MAY 1961



Bending moment diagram.

Axial force diagram.



FORCE AND MOMENT DIAGRAM
FROM
E. ERDEI AND J. GHOSH
CONCRETE, SEPT. 1967

"THE EFFECTS OF WIND ON LARGE DIAMETER CHIMNEYS AND SHAFTS"

UPSTREAM

$M = .354\ PR^2$

$F = -.667\ PR$

DOWNSTREAM

$M = .221\ PR^2$

$F = -1.419\ PR$

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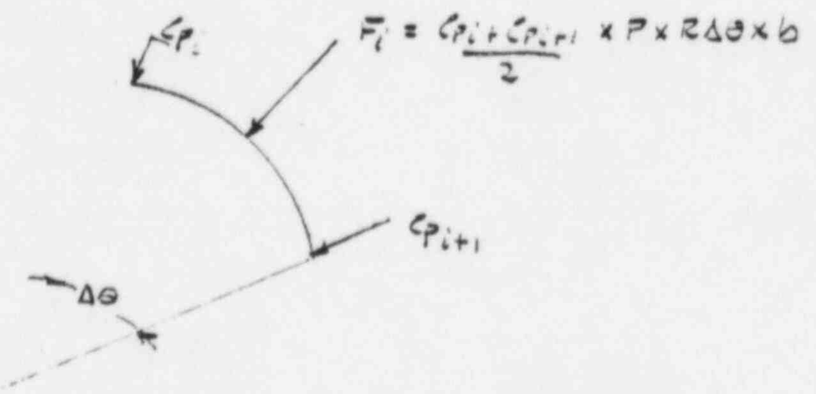
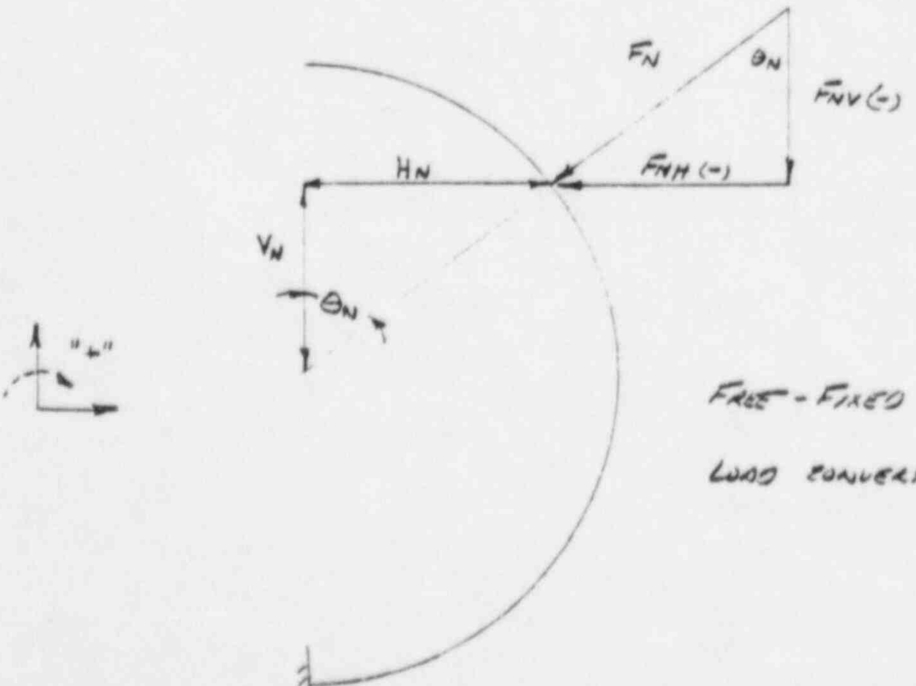
Title: CIRCUMFERENTIAL STRESS ANALYSIS, STATICALLY EQUIVALENT POINT PRESSURE LOADS

CONVERT PRESSURE LOADING TO CONCENTRATED FORCES

θ (DEG)	C_p *
0	1.0
10	.89
20	.55
30	.13
40	-.36
50	-1.07
60	-1.6
70	-2.0
80	-1.96
90	-1.64
100	-1.11
110	-.84
↓	↓
180	-.84



R = RADIUS OF STACK
P = STAGNATION PRESSURE
b = WIDTH



FREE - FIXED MODEL WITH PRESSURE LOAD CONVERTED TO CONCENTRATED FORCES

* FROM ROSHKO OP. CIT.

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	By DJB	Date	Ch'k'd	Date	Rev.	Date

Title: Statically Equivalent Point Pressure Loads, Point of Application and Cut Location

CRACKED RING MODEL OF R.C.CHIMNEYS. USED TO STUDY
THE STRESS RESULTANTS OF THE SECTION DUE TO RADIAL
PRESSURE LOADS. THE PRESSURE DISTRIBUTION IS BASED
ON A HIGH REYNOLD'S NUMBER WIND FLOW (10**7,ROSHKO)

KEY TO VARIABLES:
P= STAGNATION PRESSURE
B= HOOP REIN. SPACING
R= CHIMNEY RADIUS

Force applied at center of 10° arc

PRESSURE LOADING TABLE

Force = F_i

PNT	ANGLE (DEG)	HORIZ (R)	VERT (R)	FORCE (P*B*R)	FH (P*B*R)	FV (P*B*R)
1	5.00	0.0872	0.9962	0.16493	-0.01437	-0.16431
2	15.00	0.2588	0.9659	0.12828	-0.03320	-0.12391
3	25.00	0.4226	0.9063	0.06196	-0.02619	-0.05615
4	35.00	0.5736	0.8192	-0.02007	0.01151	0.01644
5	45.00	0.7071	0.7071	-0.12479	0.08824	0.08824
6	55.00	0.8192	0.5736	-0.23300	0.19086	0.13364
7	65.00	0.9063	0.4226	-0.31416	0.28472	0.13277
8	75.00	0.9659	0.2588	-0.34558	0.33380	0.08944
9	85.00	0.9962	0.0872	-0.31416	0.31296	0.02738
10	95.00	0.9962	-0.0872	-0.23998	0.23907	-0.02092
11	105.00	0.9659	-0.2588	-0.17017	0.16437	-0.04404
12	115.00	0.9063	-0.4226	-0.14661	0.13287	-0.06196
13	125.00	0.8192	-0.5736	-0.14661	0.12009	-0.08409
14	135.00	0.7071	-0.7071	-0.14661	0.10367	-0.10367
15	145.00	0.5736	-0.8192	-0.14661	0.08409	-0.12009
16	155.00	0.4226	-0.9063	-0.14661	0.06196	-0.13287
17	165.00	0.2588	-0.9659	-0.14661	0.03794	-0.14161
18	175.00	0.0872	-0.9962	-0.14661	0.01278	-0.14605

CUT LOCATION TABLE

Cuts located every 10°

CUT	ANGLE (DEG)	HORIZ (R)	VERT (R)
1	10.00	0.1736	0.9848
2	20.00	0.3420	0.9397
3	30.00	0.5000	0.8660
4	40.00	0.6428	0.7660
5	50.00	0.7660	0.6428
6	60.00	0.8660	0.5000
7	70.00	0.9397	0.3420
8	80.00	0.9848	0.1736
9	90.00	1.0000	0.0000
10	100.00	0.9848	-0.1736
11	110.00	0.9397	-0.3420
12	120.00	0.8660	-0.5000
13	130.00	0.7660	-0.6428
14	140.00	0.6428	-0.7660
15	150.00	0.5000	-0.8660
16	160.00	0.3420	-0.9397
17	170.00	0.1736	-0.9848
18	180.00	0.0000	-1.0000

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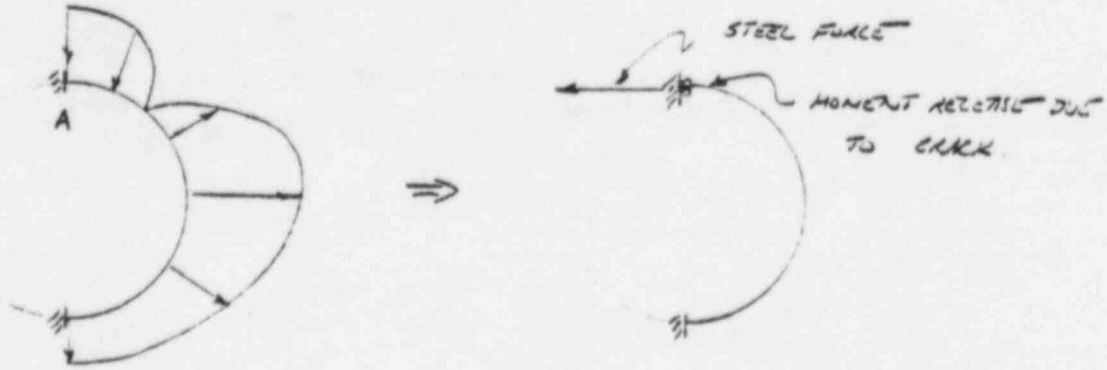
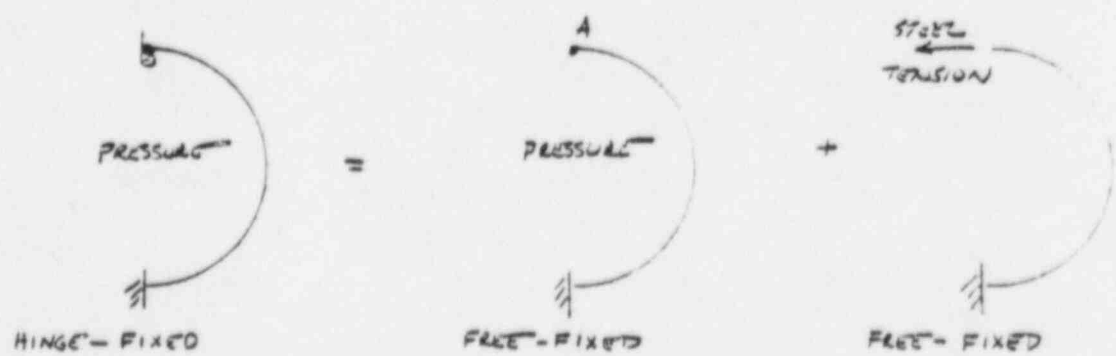
By [illegible] Date	Ch'k'd Date	Rev. Date

Title
CIRCUMFERENTIAL STRESS ANALYSIS, UPSTREAM FACE CRACKED

AS THE WINDSPEED INCREASES THE MOMENT CAPACITY OF THE UPSTREAM SECTION IS EXCEEDED (POINT A). THE OUTSIDE CONCRETE COVER FAILS AND THE SECTION CAN NO LONGER RESIST MOMENT. BECAUSE OF THIS RELEASE THERE IS A REDISTRIBUTION OF MOMENT



SOLVE HINGE-FIXED ARCH PROBLEM THROUGH METHOD OF CONSISTENT DEFORMATIONS



WHERE AT POINT A DEFLECTION DUE TO PRESSURE = DEFLECTION DUE TO TENSION

ASSUMING NEGLIGIBLE ELONGATION IN STEEL BAR.

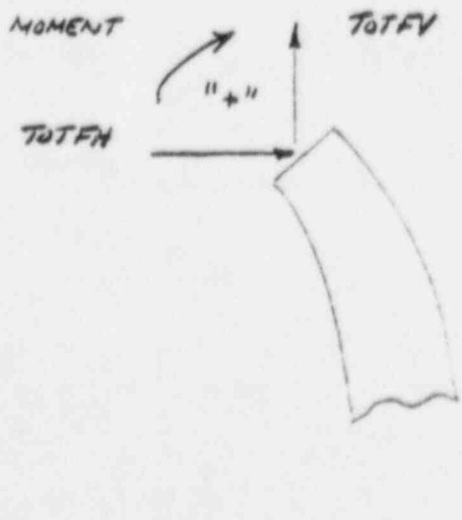
FORM CS-FRC-81

Title
FREE-FIXED MODEL WITH PRESSURE LOADS

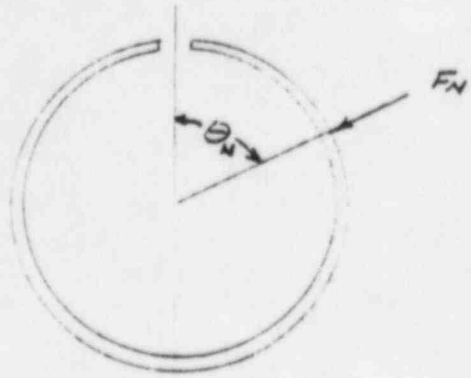
STRESS RESULTANT TABLE
FOR
FREE-FIXED MODEL

ANGULAR LOCATION OF CUT

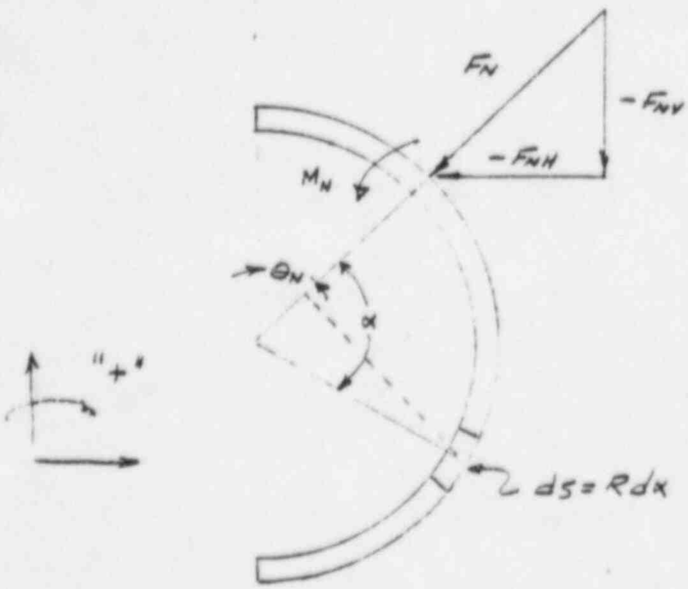
CUT	MOMENT (P*B*R**2)	AXIAL (P*B*R)	SHEAR (P*B*R)	TOTFH (P*B*R)	TOTFV (P*B*R)	ANGLE (DEG)
1	-0.01437	0.00000	-0.16493	-0.01437	-0.16431	10.00000
2	-0.05387	0.02228	-0.29127	-0.04758	-0.28822	20.00000
3	-0.10831	0.04347	-0.34949	-0.07376	-0.34437	30.00000
4	-0.16310	0.03343	-0.33211	-0.06225	-0.32793	40.00000
5	-0.20032	-0.04678	-0.23651	0.02599	-0.23969	50.00000
6	-0.20026	-0.22527	-0.08674	0.21685	-0.10604	60.00000
7	-0.14644	-0.49734	0.07034	0.50158	0.02672	70.00000
8	-0.03066	-0.82208	0.18853	0.83538	0.11617	80.00000
9	0.14355	-1.13146	0.24309	1.14834	0.14355	90.00000
10	0.36169	-1.37145	0.24309	1.38741	0.12263	100.00000
11	0.60459	-1.53903	0.21354	1.55178	0.07859	110.00000
12	0.85673	-1.67680	0.16339	1.68466	0.01663	120.00000
13	1.10839	-1.80376	0.09009	1.80475	-0.06746	130.00000
14	1.35193	-1.91607	-0.00415	1.90842	-0.17113	140.00000
15	1.57995	-2.01031	-0.11646	1.99251	-0.29122	150.00000
16	1.78552	-2.08361	-0.24342	2.05447	-0.42409	160.00000
17	1.96239	-2.13375	-0.38119	2.09241	-0.56571	170.00000
18	2.10519	-2.15921	-0.52557	2.10519	-0.71176	180.00000



Title: DEFLECTION ANALYSIS FOR FREE-FIXED MODEL UNDER PRESSURE LOADS



- ALL FORCES RADIAL
- F_N IS POSITIVE WHEN ACTING INWARD
- CALCULATIONS PERFORMED PER UNIT WIDTH
- ASSUME t/R SMALL WITH LINEAR DISTRIBUTION OF STRAINS THROUGH THICKNESS



$F_N^2 = F_{NV}^2 + F_{NH}^2$

M_N = DUMMY LOADING

U_N = STRAIN ENERGY FOR LOAD F_N

$$= \int_{\alpha=0}^{\alpha=\pi-\theta_N} \frac{M^2}{2EI} R\,d\alpha$$

$$M(\alpha) = -F_N R \sin\alpha - M_N$$

$$M^2(\alpha) = (F_N R \sin\alpha)^2 + 2M_N F_N R \sin\alpha + M_N^2$$

$$U_N = \int_0^{\pi-\theta_N} \left((F_{NH}^2 + F_{NV}^2) R^2 \sin^2\alpha + 2M_N (F_{NV}^2 + F_{NH}^2)^{1/2} R \sin\alpha + M_N^2 \right) \frac{R\,d\alpha}{2EI}$$

TO SOLVE FOR DEFLECTIONS UNDER LOAD USE CASTIGLIANO'S THM.

$$\Delta x_N = \frac{\partial U_N}{\partial F_{NH}} = \int_0^{\pi-\theta_N} \left(2F_{NH} R^2 \sin^2\alpha + \frac{2M_0 F_{NH} R \sin\alpha}{(F_{NV}^2 + F_{NH}^2)^{1/2}} \right) \frac{R\,d\alpha}{2EI}$$

$$= \frac{F_{NH} R^3}{EI} \left(\frac{\alpha}{2} - \frac{\sin 2\alpha}{4} \Big|_0^{\pi-\theta_N} \right) \qquad \text{WITH } M_N = 0$$

$$= \frac{F_{NH} R^3}{EI}\left(\frac{\pi-\theta_N}{2} - \frac{\sin(2\pi-2\theta_N)}{4}\right)$$

$$\boxed{\therefore\ \Delta x_N = \frac{F_{NH} R^3}{EI}\left(\frac{\pi-\theta_N}{2} + \frac{\sin(2\theta_N)}{4}\right)}$$

$$\Delta y_N = \frac{\partial U_N}{\partial F_{NV}} = \int_0^{\pi-\theta_N}\left(2F_{NV}R^2\sin^2\alpha + \frac{2M_N F_{NV} R\sin\alpha}{(F_{NV}^2+F_{NH}^2)^{1/2}}\right)\frac{R\,d\alpha}{2EI}$$

$$= \frac{F_{NV} R^3}{EI}\left(\frac{\alpha}{2} - \frac{\sin 2\alpha}{4}\Big|_0^{\pi-\theta_N}\right)$$

$$\boxed{\Delta y_N = \frac{F_{NV} R^3}{EI}\left(\frac{\pi-\theta_N}{2} + \frac{\sin(2\theta_N)}{4}\right)}$$

$$\Delta\theta_N = \frac{\partial U_N}{\partial M_N} = \int_0^{\pi-\theta_N}\left(2\left(F_{NV}^2+F_{NH}^2\right)^{1/2} R\sin\alpha + 2M_N\right)\frac{R\,d\alpha}{2EI}$$

SIGN OF F_N IS "−"

$$= \frac{F_N R^2}{EI}\cos\alpha\Big|_0^{\pi-\theta_N}$$

$$\boxed{\Delta\theta_N = -\frac{F_N R^2}{EI}(\cos\theta_N + 1)}$$

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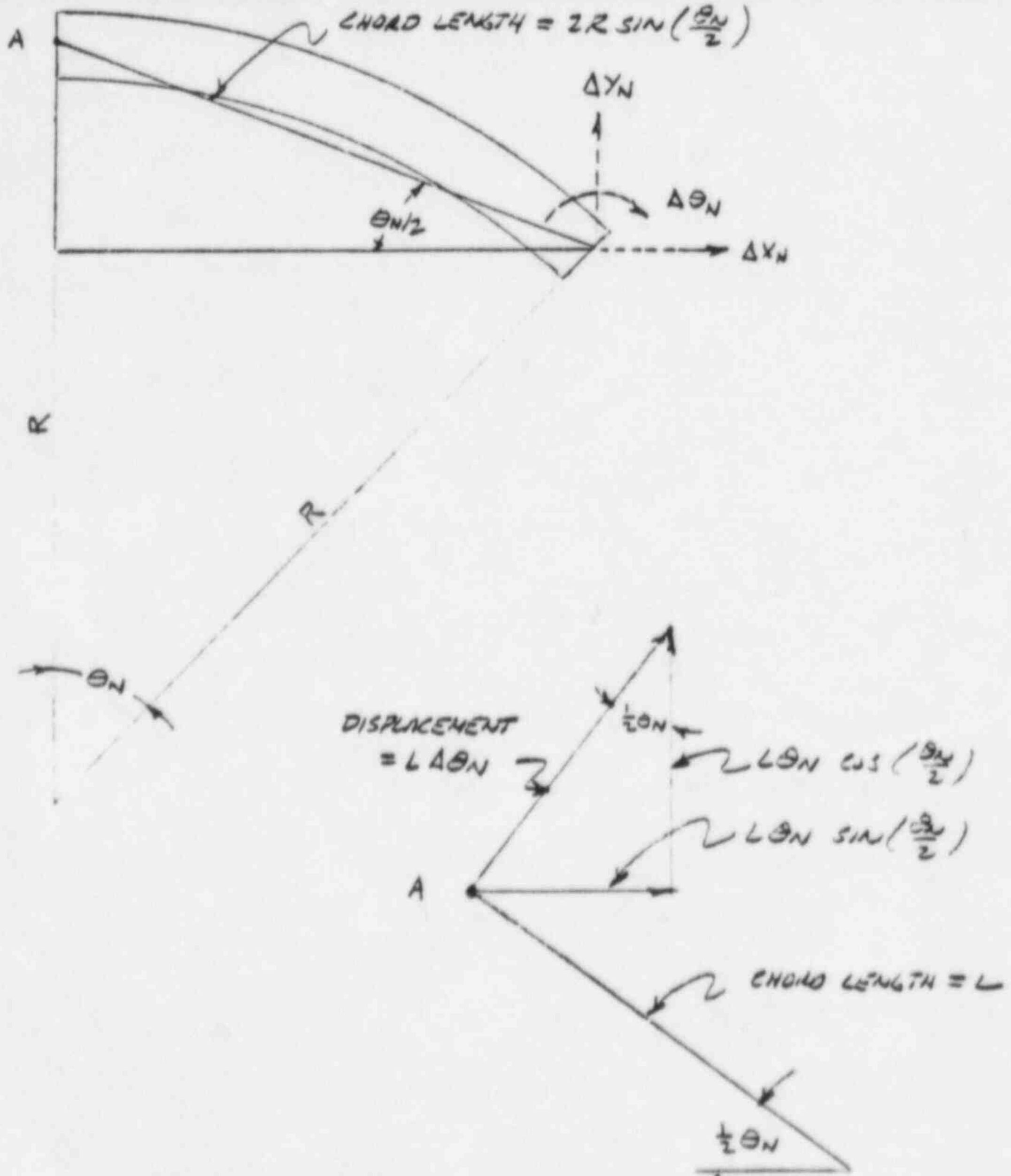
By DJB | Date | Ch'k'd | Date | Rev. | Date

Title DEFLECTION ANALYSIS FOR FREE-FIXED MODEL UNDER PRESSURE LOADS



TOTAL DISPLACEMENT,

HORIZONTAL $= \Delta X_N + L\theta_N \sin(\frac{\theta_N}{2})$

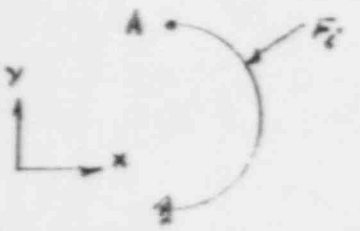
VERTICAL $= \Delta Y_N + L\theta_N \cos(\frac{\theta_N}{2})$

DEFLECTION TABLE
FOR
FREE-FIXED MODEL

PNT	DELXN (K)	DELYN (K)	DLLRA (L)	CHORD (R)	CUTX (K)	CUTY (K)	DEBUG (R4)
1	-0.02258	-0.25805	-0.32924	0.08724	-0.02383	-0.28675	1.57058
2	-0.05196	-0.19391	-0.25219	0.26105	-0.06055	-0.25918	1.56490
3	-0.04043	-0.08671	-0.11811	0.43288	-0.05150	-0.13663	1.54414
4	0.01727	0.02467	0.03651	0.60141	0.02388	0.04561	1.50029
5	0.12602	0.12602	0.21303	0.76537	0.18841	0.27665	1.42810
6	0.25304	0.17718	0.36665	0.92350	0.40938	0.47752	1.32575
7	0.34027	0.15867	0.44693	1.07460	0.59832	0.56372	1.19508
8	0.34759	0.09314	0.43502	1.21752	0.67001	0.51333	1.04130
9	0.27304	0.02389	0.34154	1.35118	0.58482	0.36413	0.87244
10	0.16695	-0.01461	0.21907	1.47455	0.40511	0.20363	0.69835
11	0.08703	-0.02332	0.12613	1.58671	0.24580	0.09851	0.52950
12	0.04992	-0.02328	0.08465	1.68678	0.17035	0.05344	0.37572
13	0.02943	-0.02061	0.06252	1.77402	0.12780	0.03061	0.24504
14	0.01479	-0.01479	0.04294	1.84776	0.08810	0.01557	0.14270
15	0.00593	-0.00847	0.02651	1.90743	0.05416	0.00674	0.07051
16	0.00165	-0.00354	0.01374	1.95259	0.02784	0.00226	0.02666
17	0.00022	-0.00084	0.00500	1.98289	0.01004	0.00046	0.00590
18	0.00000	-0.00003	0.00056	1.99810	0.00112	0.00002	0.00022

TOTAL DEFLECTION AT CUT
X-DIRECTION 3.46926*(K)
Y-DIRECTION 1.96963*(K)

K=P*B*R**4/EI
L=P*B*R**3/EI



HORIZONTAL DISPLACEMENT AT POINT A DUE TO LOAD i = $CUTX_i$

VERTICAL " " " = $CUTY_i$

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Title: DEFLECTION ANALYSIS FOR FREE-FIXED MODEL UNDER RESULTANT LOADS

Project: 504-F08-003

By: DJB | Date | Chk'd | Date | Rev. | Date

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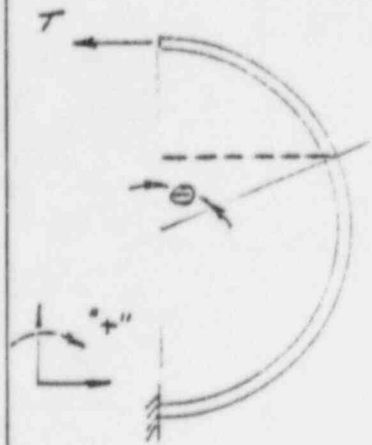
Title: DEFLECTION ANALYSIS FOR FREE-FIXED MODEL UNDER STEEL TENSION LOAD



FREE-FIXED MODEL

T = STEEL TENSION

$$U = \text{STRAIN ENERGY} = \int_{\theta=0}^{\theta=\pi} \frac{M^2}{2EI} R\,d\theta$$

$$M(\theta) = -TR(1-\cos\theta)$$

$$\therefore \quad U = \int_0^{\pi} T^2R^2(1-2\cos\theta+\cos^2\theta)\frac{R\,d\theta}{2EI}$$

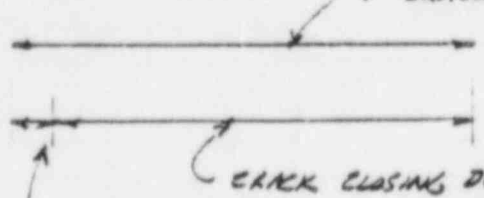
$$= \frac{T^2R^3}{2EI}\left(\theta - 2\sin\theta + \frac{\theta}{2} + \frac{\sin 2\theta}{4}\Big|_0^{\pi}\right)$$

$$= \frac{T^2R^3}{2EI}\left(\pi + \frac{\pi}{2}\right)$$

USING CASTIGLIANO'S THM. $\frac{\partial U}{\partial T} = -\Delta x = \frac{3TR^3\pi}{2EI}$

$$\boxed{\Delta x = \frac{3TR^3\pi}{2EI}}$$

FROM COMPATIBILITY,



$$\Delta_C = C_\Delta \frac{PBR^4}{E_G I_G}$$ WHERE C_Δ = 3.469 AS PER PAGE

$$\Delta_S = \frac{3}{2}\pi \frac{TR^3}{E_G I_G}$$

AND SINCE $\Delta_C = \Delta_S \Rightarrow T = \frac{C_\Delta PBR}{1.5\pi}$

∴ THE NET MOMENT AT A SECTION

$$= C_M PBR^2 - TR(1-\cos\theta)$$

$$= PBR^2\left(C_M - \frac{C_\Delta}{1.5\pi}(1-\cos\theta)\right) \Rightarrow$$

WHERE C_Δ = THE FREE-FIXED MODEL COEFFICIENTS AS GIVEN ON PAGE

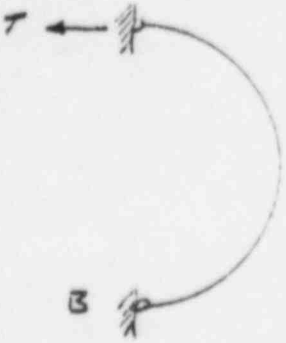
MOMENT RESULTANT TABLE
FOR
HINGE-FIXED MODEL

ANGLE (DEG)	NET MOMENT (P*B*R**2)
10.00000	-0.02556
20.00000	-0.09827
30.00000	-0.20694
40.00000	-0.33534
50.00000	-0.46330
60.00000	-0.56836
70.00000	-0.63064
80.00000	-0.63902
90.00000	-0.59265
100.00000	-0.50235
110.00000	-0.38340
120.00000	-0.24757
130.00000	-0.10103
140.00000	0.05177
150.00000	0.20618
160.00000	0.35752
170.00000	0.50117
180.00000	0.63279

Title

CIRCUMFERENTIAL STRESS ANALYSIS, HINGE – HINGE MODEL

THE HINGE–FIXED MODEL PREDICTS OUTER FACE CRACKING ON THE DOWNSTREAM SIDE. WITH THE CRACK, A NEW HINGE IS FORMED SO THAT THE MOMENTS ARE REDISTRIBUTED AGAIN. TO FIND THE NEW DISTRIBUTION EXAMINE A HINGE–HINGE MODEL.



TO SOLVE, SUPERIMPOSE THE FREE–FIXED PRESSURE LOADED SOLUTION WITH A TENSION SOLUTION WITH T AT A LEVEL WHICH CAUSES ZERO MOMENTS AT B. (THUS SATISFYING EQUILIBRIUM)

MOMENT AT B, HINGE–FIXED MODEL = $C_{M_{180}} PBR^2$
" , STEEL RESISTANCE = $2RT$

$\Rightarrow T = \frac{1}{2} C_{M_{180}} PBR$

WHERE $C_{M\,180} = 2.105$ AS PER PAGE (AT CUT 19)

$\therefore$ NET MOMENT AT SECTION $= C_M PBR^2 - T(R - R\cos\theta)$

$= PBR^2 \left(C_M - \frac{1}{2} C_{M_{180}} (1 - \cos\theta)\right)$

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Title: SOLUTION OF HINGE - HINGE MODEL

THE HINGE-HINGE MODEL PREDICTS NEGATIVE MOMENTS THROUGHOUT THE SECTION SO THAT THE EXTERNAL FACE OF THE CHIMNEY IS NO LONGER IN COMPRESSION (WRT CIRCUMFERENTIAL STRESSES).

MOMENT RESULTANT TABLE
FOR
HINGE-HINGE MODEL

ANGLE (DEG)	NET MOMENT (P*B*R**2)
10.00000	-0.03037
20.00000	-0.11735
30.00000	-0.24933
40.00000	-0.40936
50.00000	-0.57632
60.00000	-0.72656
70.00000	-0.83902
80.00000	-0.90047
90.00000	-0.90905
100.00000	-0.87369
110.00000	-0.80801
120.00000	-0.72216
130.00000	-0.62080
140.00000	-0.50700
150.00000	-0.38422
160.00000	-0.25619
170.00000	-0.12681
180.00000	0.00000

APPENDIX C

TURBINE BUILDING DESIGN REVIEW CALCULATIONS

FRANKLIN RESEARCH CENTER
DIVISION OF ARVIN/CALSPAN

20th and Race Streets. Phila., Pa. 19103 (215) 448-1000

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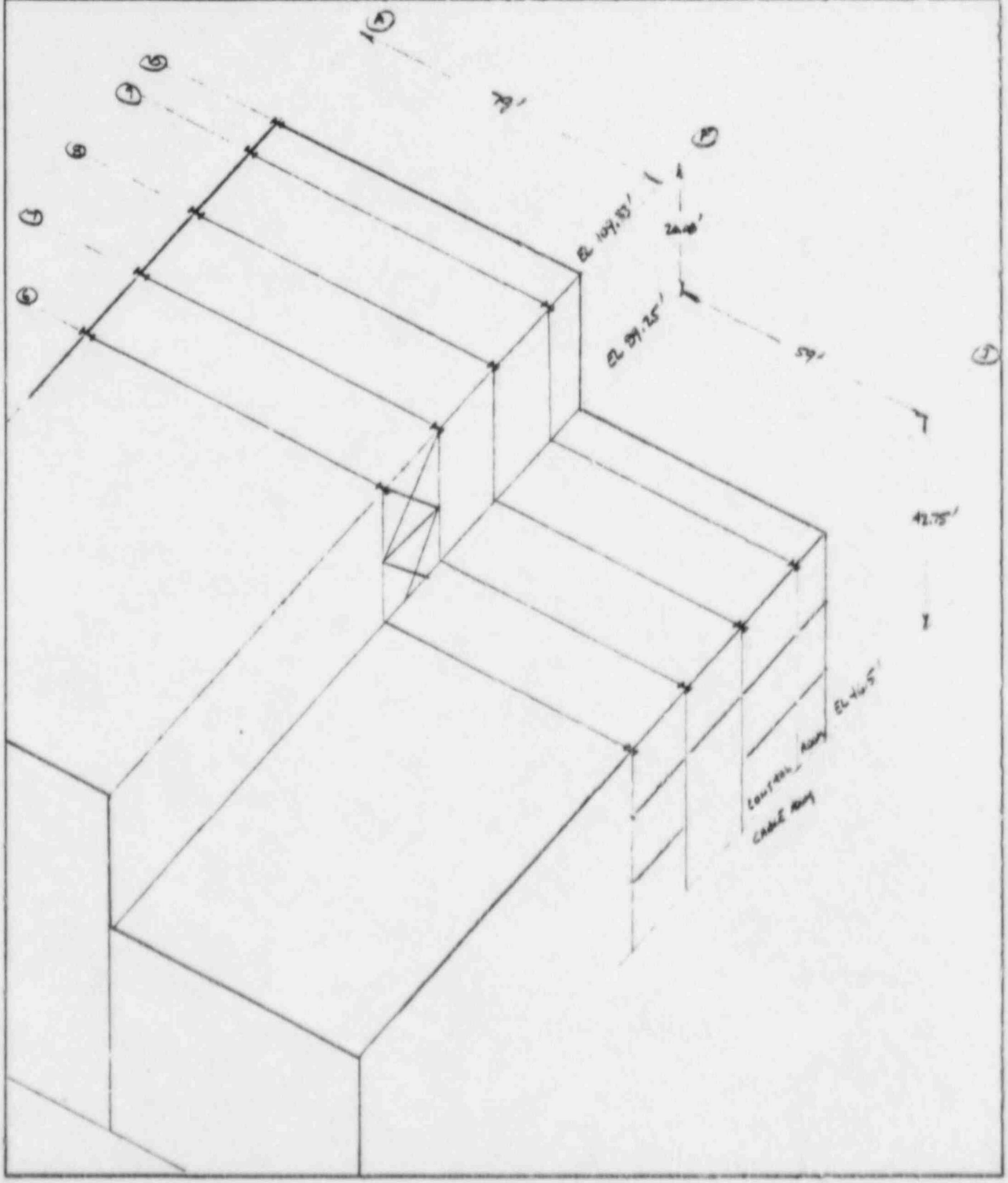
Title
TURBINE BLDG



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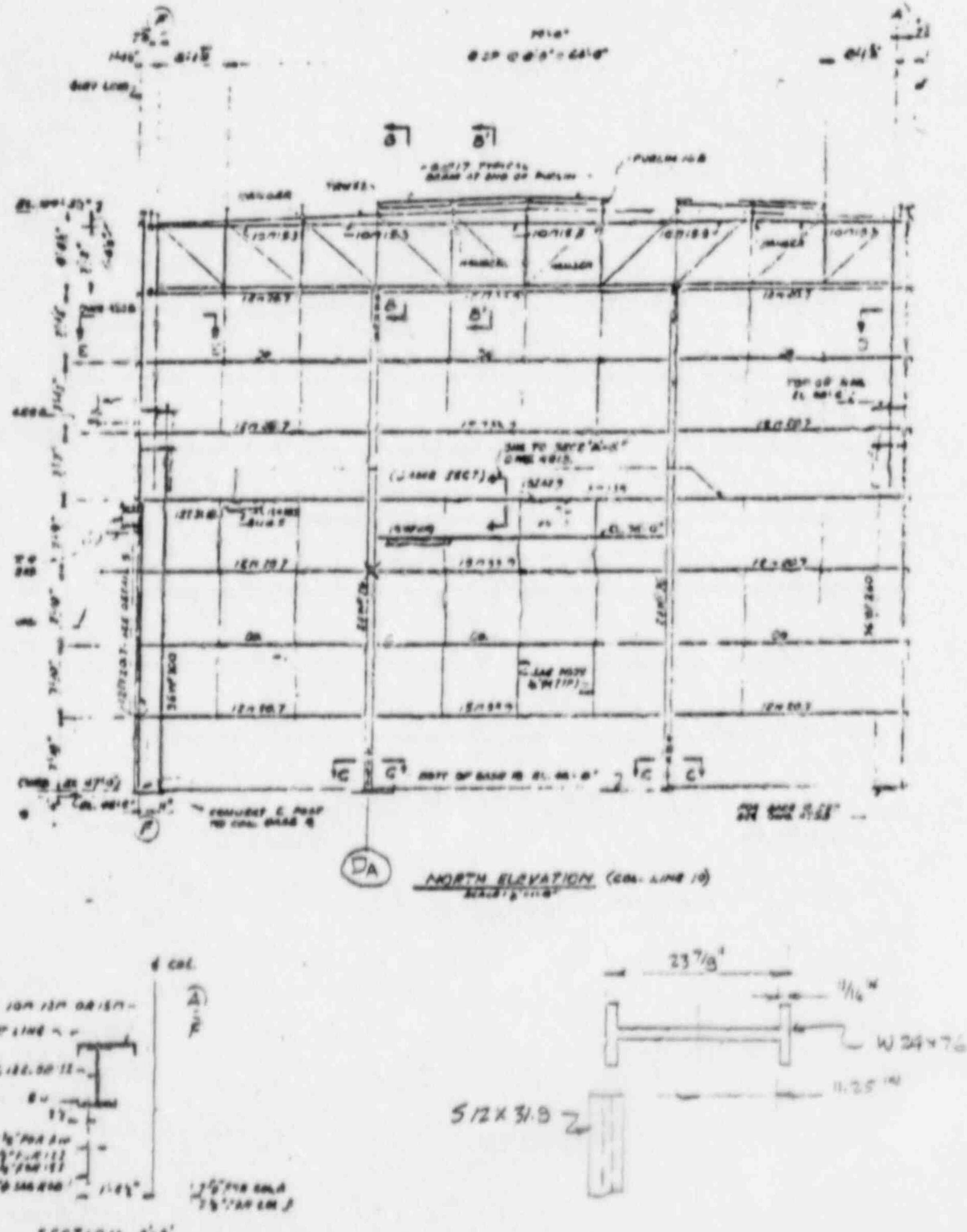
504-5526-003	C-2

By	Date	Chk'd	Date	Rev.	Date
DJB	4/10/84	CB	5/1/84		

Title: TURBINE BUILDING



FORM CS-FRC-81

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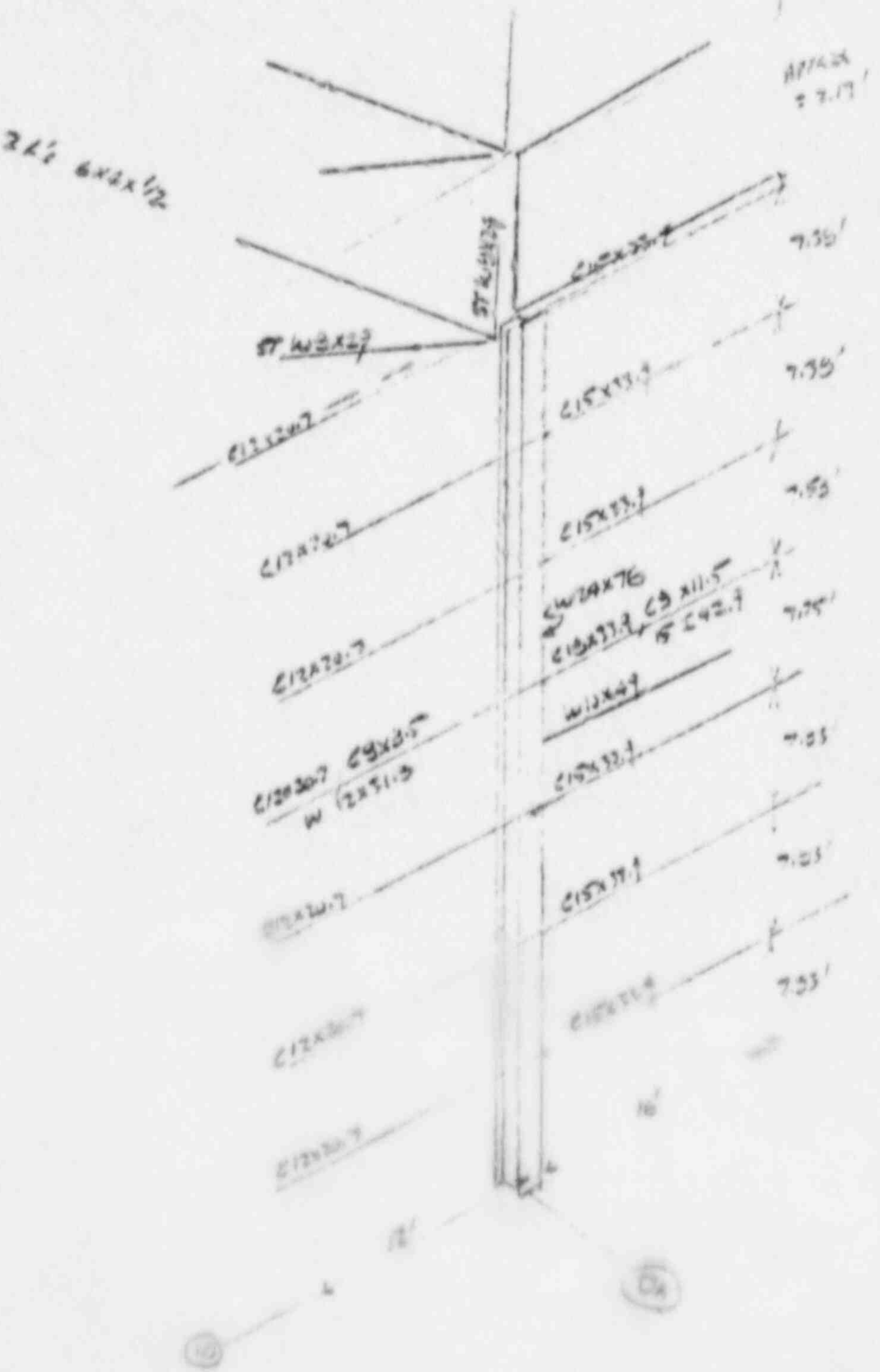
	By DJB	Date 4/27/84	Ch'k'd	Date	Rev.	Date

Title
TURBINE BUILDING, COLUMN ON NORTH ELEVATION



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524-532-003	C-4

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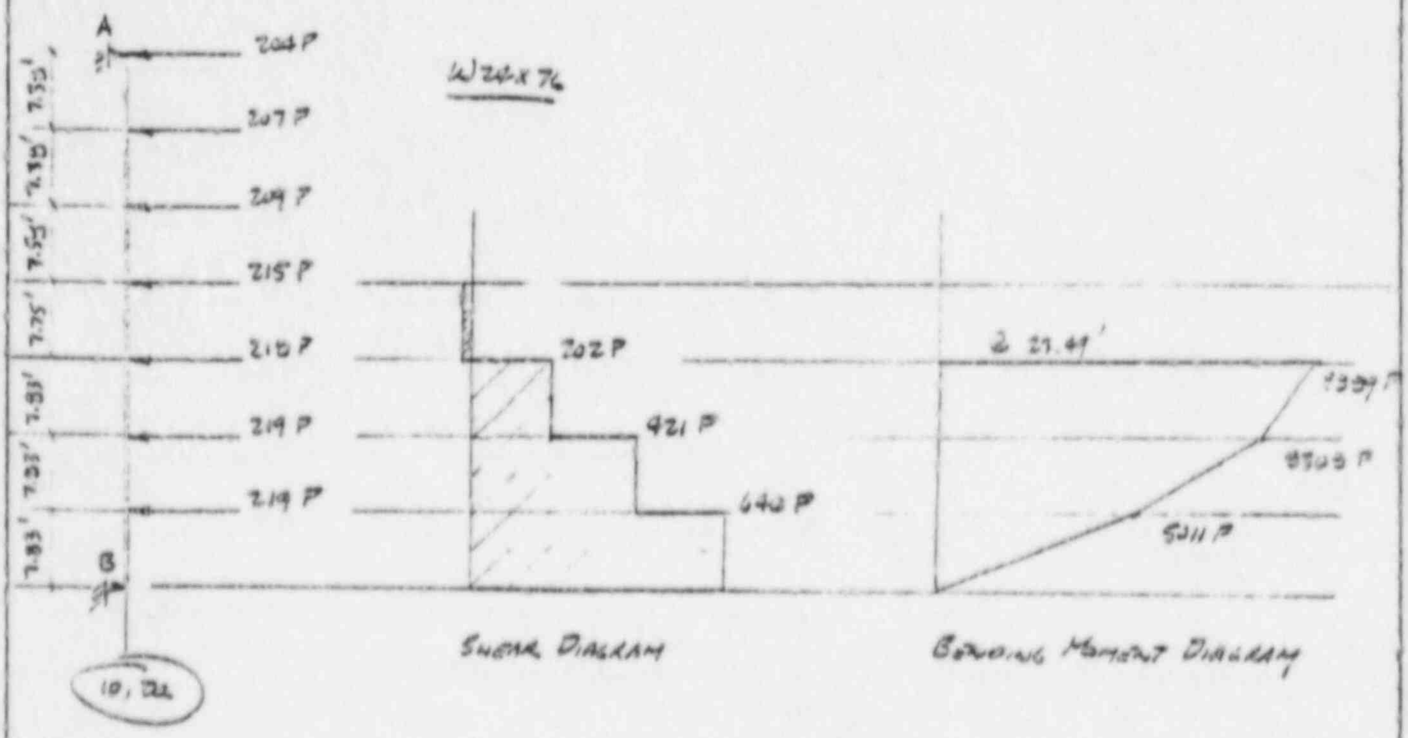
DJB	4/30/84				

Title: Turbine Building, Columns on North Elevation



53.58 R_A = 204 P (53.58') + 207 P (46.2') + 209 P (38.32') + 215 P (31.24')

+ 213 P (23.49') + 219 P (15.66') + 219 P (7.83')

= 45590 P ⇒ R_A = 851 P

R_B = 640 P

Maximum Bending Moment = 9337 P (# - ft.)

where P is in PSF

(C12x20.7)(12')(5) = 1242#

(C8x8.5)(12')(1) = 102#

(W12x31.8)(12')(1) = 382#

(C15x33.9)(16')(5) = 2712#

(C8x11.5)(16')(1) = 184#

(15 I 42.9)(16')(1) = 686#

(W10x49)(16')(1) = 784#

(W24x76)(30.09') = 2287#

(16'+12')(30.09+7.5)(5 PSF)
= 5263# siding

Total D.L. @ 23.49'
= 13.64 K

FORM CS-FRC-81

Title: TURBINE BUILDING, COLUMN ON NORTH ELEVATION

W24x76

$A = 22.4\ IN^2$ $\quad S_{xx} = 176\ IN^3$ $\quad R_y = 1.92\ IN$ $\quad d/A_F = 3.90$

$R_x = 9.69\ IN$ $\quad R_T = 2.32\ IN$ $\quad b_F/2t_F = 6.57$

$$\frac{K\ell}{R}\Big)_x = \frac{(1.0)(53.58 \times 12'')}{9.69\ IN} = 66.4$$

$$\frac{K\ell}{R}\Big)_y = \frac{(1.0)(31.25 \times 12'')}{1.92\ IN} = 195.3$$

$$\Longrightarrow F_A = \frac{12\pi^2 E}{23\ (K\ell/R)^2} = 3.92\ KSI \qquad \text{FOR EEE} = 6.27\ KSI$$

$$f_A = \frac{13.64}{22.4\ IN^2} = .607\ KSI$$

$$f_A/F_A = .15 \qquad = .097$$

$$\frac{\ell}{R_T} = \frac{53.58 \times 12''}{2.32\ IN} = 277.1 \quad >> 119\sqrt{C_B}$$

$$F_B = \frac{170 \times 10^3 C_B}{(\ell/R_T)^2} = 2.21\ KSI$$

$$F_B = \frac{12 \times 10^3 C_B}{\ell d/A_F} = 4.79\ KSI$$

$$\Longrightarrow F_B = 4.79\ KSI$$

$$M_{ALL} = (4.79\ KSI)(176\ IN^3)\left(\frac{1000\#\text{-FT}}{12\ IN\text{-}K}\right) = 70,250\ \#\text{-FT}$$

FOR EEE $\quad M_{ALL} = 112,400\ \#\text{-FT}$

FROM PAGE C-4 $\quad M = 9839\ P$

THEREFORE, INTERACTION EQUATION BECOMES

$$.097 + \frac{9839 P}{112,400} \leq 1 \quad \Longrightarrow \quad P \leq 10.3\ PSF$$

FOR OUTWARDLY ACTING DYNAMIC PRESSURE $\quad P = \frac{10.3}{.70} = 14.7\ PSF, \quad V = 75.9\ MPH$

FOR DIFFERENTIAL PRESSURE $\quad V = \left(35507 \times \frac{P}{144}\right)^{1/2} = 50.4\ MPH$

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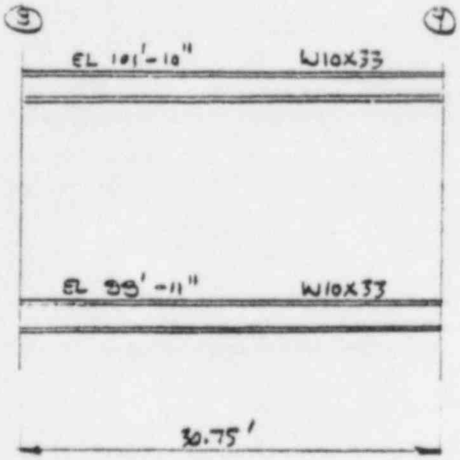
Title: COMPRESSION MEMBERS, EAST ELEVATION OF TURBINE BUILDING



COLUMN LINE (J)

W10x33

$A = 9.71\ IN^2$ $\quad R_T = 2.16^{IN}$ $\quad d/A_F = 2.33$

$S_{xx} = 35^{IN^3}$ $\quad S_{yy} = 9.16^{IN^3}$

$R_x = 4.20^{IN}$ $\quad R_y = 1.94^{IN}$

$$\left.\frac{l}{R}\right)_y = \frac{30.75 \times 12^{IN}}{1.94^{IN}} = 190.2 \implies F_A = \frac{12\pi^2 E}{23\left(\frac{Kl}{R}\right)_y^2} = 4.13^{KSI}$$

$$\frac{l}{R_T} = \frac{30.75 \times 12^{IN}}{2.16^{IN}} = 170.8 >> 119\sqrt{C_B}$$

$$F_B = \frac{170 \times 10^3 C_b}{(l/R_T)^2} = 5.83^{KSI}$$

$$F_B = \frac{12 \times 10^3 \times C_b}{l d/A_F} = 11.49^{KSI}$$

$$\implies F_B = 11.49^{KSI}$$

$$\left.\frac{l}{R}\right)_B = \frac{30.75 \times 12^{IN}}{4.20^{IN}} = 87.9$$

$$F'_e = \frac{12\pi^2 E}{23\left(\frac{Kl}{R}\right)_B^2} = 19.33^{KSI}$$

$$M = \frac{1}{8} w l^2 = \frac{1}{8}(33^{\#/FT})(30.75')^2 = 3900\ ^{\#}\text{-FT}$$

$$f_B = M/S = (3900 \times 12'')/35^{IN^3} = 1337\ PSI$$

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504-5506-003	C-7

By	Date	Ch'k'd	Date	Rev.	Date
JTB	4/30/84				

Title: Compression Members, East Elevation of Turbine Building

INTERACTION EQUATION,

$$\frac{f_A}{F_A} + \frac{C_{mx} f_{bx}}{\left(1-\frac{f_A}{F'_{ex}}\right) F_{bx}} \le 1.0 \Rightarrow \frac{f_A}{4.13^{KSI}} + \frac{(1.0)(1.34^{KSI})}{\left(1-\frac{f_A}{19.33^{KSI}}\right)(11.27^{KSI})} \le 1.0$$

$$(f_A)\left(1-\frac{f_A}{19.33}\right) + .492 \le (4.13)\left(1-\frac{f_A}{19.33}\right)$$

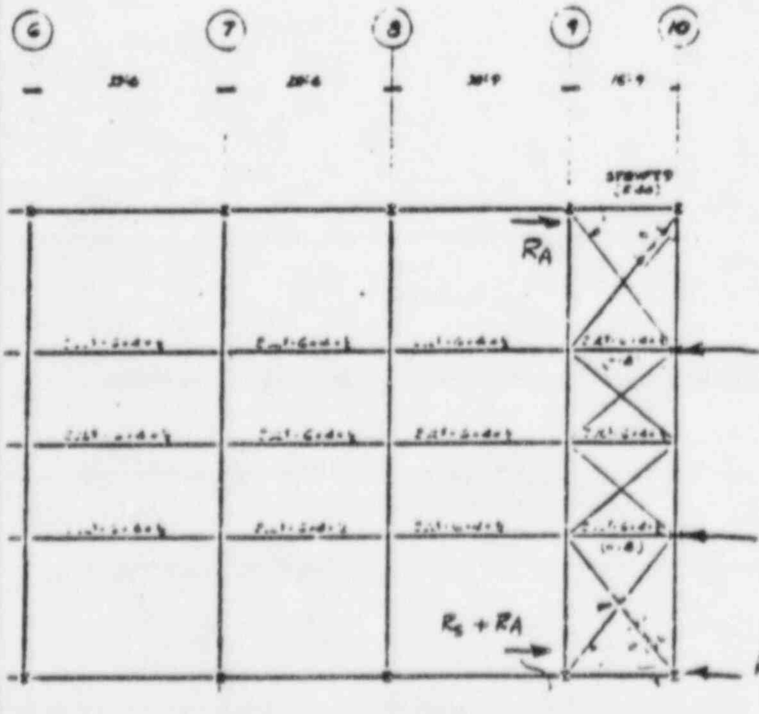
$$19.33 f_A - f_A^2 + 9.52 \le 79.83 - 4.13 f_A$$

$$f_A^2 - 23.46 f_A + 70.31 \le 0$$

$$\therefore f_A = \frac{23.46 \pm \sqrt{(-23.46)^2 - (4)(1)(70.31)}}{2}$$

$$= \underline{3.54^{KSI}},\ 19.92^{KSI}$$

FOR SSE $f_A = (1.6)(3.54) = 5.66^{KSI}$



- ASSUME FRAMES AT EL. 39'-3" TRANSMIT LATERAL FORCES THROUGH ROOF STEEL — RESISTED BY DOWNSTREAM BRACE IN TURBINE BUILDING
- ASSUME CONCRETE WALLS DIRECTLY RESIST FORCES BETWEEN COL LINES (E) & (J)

REACTION R_A SEE PAGE C-4

REACTION R_A = 851 P

$R_S = (12')(10.96')P = 132 P$

= 983 P WHERE P IS IN PSF

FORM CS-FRC-81

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By	Date	Ch'k'd	Date	Rev.	Date
DJB	5/1/84				

Title: COMPRESSION MEMBERS, EAST ELEVATION OF TURBINE BUILDING

EQUATING APPLIED TO ALLOWABLE

$993 P = (5.66 KSI)(1000 \#)(9.71 IN^2)$

$P = 55.9$ PSF

$\therefore$ DYNAMIC PRESSURE $= 55.9/.8 = 69.9$ PSF

$$V = \left(\frac{69.9}{.00256}\right)^{1/2} = 165 \text{ MPH}$$

FORM CS-FRC-81

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	By: DJB Date: 5/1/84	Ch'k'd Date:	Rev. Date

Title: BOTTOM CHORD HORIZONTAL BRACING MEMBERS

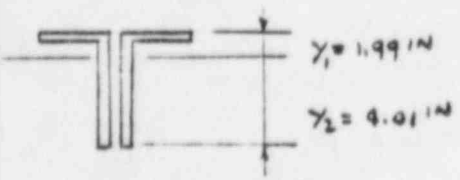
STRUTS (SEE PAGE)

2 L'S 6X4X1/2 LLBB

$w = 32.4$ #/FT $S_{xx} = 2.67$ IN³ $R_y = 1.60$ IN FOR 1/4" SEPERATION

$A = 9.50$ IN² $R_x = 1.91$ IN $R_y = 1.69$ IN FOR 1/2" SEPERATION

$$\frac{Kl}{R}\Big)_y = \frac{(1.0)(16.75 \times 12^{IN})}{1.60^{IN}} = 125.6 \implies F_A = \frac{12\pi^2 E}{23\left(\frac{Kl}{R}\right)_y^2} = 9.46^{KSI}$$



$I_{xx} = 34.8$ IN⁴

$$M = \tfrac{1}{8} w l^2 = \tfrac{1}{8}(32.4\,\#/FT)(16.75')^2 = 1136\ \#\text{-}FT$$

$$f_B = \frac{(1136\,\#\text{-}FT \times 12^{IN})(1.99^{IN})}{34.8^{IN^4}} = .78^{KSI}$$

SWAY BRACES AND TRUSSES

SECTION B-B ON DWG. 4209-2

2 L'S 6X4X1/2 SLBB

$R_{xx} = 1.15$ IN

$R_{yy} = 2.85$" FOR 1/4"

$= 2.94$" FOR 1/2"

$$\frac{Kl}{R}\Big)_x = \frac{(1.0)(\frac{1}{2} \times 30.75 \times 12'')}{1.15^{IN}} = 160.4 \implies F_A = 5.80^{KSI}$$

$$\frac{Kl}{R}\Big)_y = \frac{(1.0)(30.75 \times 12^{IN})}{2.94^{IN}} = 125.5$$