(DRAFT)

TECHNICAL EVALUATION REPORT

WIND AND TORNADO LOADINGS (SEP, III-2)

COMMONWEALTH EDISON COMPANY
DRESDEN NUCLEAR POWER STATION UNIT 2

NRC DOCKET NO. 50-237

NRC TAC NO. 41607

NRC CONTRACT NO. NRC-03-79-118

FRC PROJECT C5257

FRC ASSIGNMENT 14

FRC TASK 401

Prepared by

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

Prepared for

Nuclear Regulatory Commission
Washington, D.C. 20555

Author: R. Agarwal
D. J. Barrett

FRC Group Leader: D. J. Barrett

Lead NRC Engineer: D. Persinko

June 21, 1982

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

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FOREWORD

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

1. INTRODUCTION

1.1 PURPOSE OF REVIEW

In the Systematic Evaluation Program (SEP), licensees are required to establish the ability of Class I structures to safely withstand a high wind or tornado strike. After conducting an appropriate investigation, licensees report their analyses and conclusions in a safety analysis report (SAR). The purpose of the present review is to provide a technical evaluation of the SAR prepared by Commonwealth Edison Company (CECO) for the Dresden Nuclear Power Station, Unit 2 [1].

1.2 GENERIC ISSUE BACKGROUND

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

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

1.3 PLANT-SPECIFIC BACKGROUND

Evaluation of the Dresden Unit 2 SAR was begun in May 1982. Prior to that time, CECO responded to NRC requests for information by providing architectural-engineering structural drawings. Additional sources of information were a CECO letter on the SEP structural topics [3] and the Dresden Unit 2 final safety analysis report (FSAR) [4]. The conclusions stated by CECO in the SAR are summarized in Table 1. Based on a visit to the Dresden Unit 2 site [5] and consultation with the NRC Lead Engineer, the reactor building, the high pressure coolant injection pump building, the diesel engine room, and the crib house were established as the primary structures for review.

All structures at Dresden Unit 2 were designed to Uniform Building Code criteria [25] and constructed to withstand the maximum potential loading that results from a wind velocity of 110 mph [4]. The Dresden Unit 2 SAR states that the current NRC requirement for a DBT of 360 mph has been replaced by a DBT of 337 mph for the purpose of that review [1].

Table 1. Summary of Conclusions from Dresden Unit 2 SEP Topic III-2 SAR

Class I Structures	Capacity
1. Reactor Building	
a. Reinforced concrete shear walls below refueling floor (Elev. 613 ft 0 in)	Tornado winds of up to 500 mph
b. Structural steel superstructure above refueling floor (Elev. 613 ft 0 in)	Tornado winds of up to 250 mph*
c. Blow-off metal siding panels on structural steel superstructure	Wind differential pressure of 70 psf (117 mph)
2. Cribhouse (reinforced concrete slab 2 ft thick at grade elev. 517 ft)	Tornado pressure drop of 3 psi
3. Off-Gas Filter Building	Not required for safe shutdown of the reactor
4. Turbine Building	
a. Reinforced concrete shear walls	Tornado winds of 500 mph
b. Steel superstructure above the main floor	Tornado winds of 250 mph
**5. Ventilation Chimney	Tornado winds of 300 mph

* It should be noted that the review of design of steel structures was conducted for windspeeds of 300 mph, as is stated in the Final Safety Analysis Report. However, the state-of-the-art design did not consider the specific form of pressure coefficients for exposed structures as has been recommended since by the Standard Review Plan. When coefficients of ASCE Paper No. 3269 [11] are used, the steel rating is determined to be 250 mph using the same standard criteria in the original design.

**The ventilation chimney has not been identified as a Class I structure in the Dresden Unit 2 SAR [1]. But in Reference 3, it is mentioned that the ventilation chimney shell remains intact at a wind velocity of 300 mph.

2. REVIEW CRITERIA

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

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

Regulatory Guide 1.117 [8] assists in the identification of structures and systems that should be protected from the effects of a DBT. This regulatory position is elaborated in the Standard Review Plan (SRP), Section 3.3.2 (NUREG-0800) [9]. The analysis presented in this report is of a representative sample of safety-related structural systems at Dresden Unit 2.

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

An additional tornado load is the impact of wind-borne missiles against structures. The potential missiles are identified in the missile spectrum of the SRP, Section 3.5.1.4 [13], while the particular missiles to be included

in this study were identified by the NRC as part of the SEP assignment [7]. References 14 and 15 assist in the determination of the structural effects of missile impact, while the guidelines of the SRP, Section 3.3.2 indicate acceptable combinations of impact effects with the loads resulting from wind and differential pressures.

Since the DBT is considered an extreme environmental event, tornado-induced loads are part of the loading combinations to be used in extreme environmental design (see Article CC-3000 in the ASME Boiler and Pressure Vessel Code [16] and the SRP, Section 3.8.4 [17]). The structural effects of these loading combinations are determined by analysis; stresses are calculated either by a working stress or ultimate strength method, whichever is appropriate for the structure under consideration. The ASME Code specifications for an extreme environmental event permit the application of reserve strength factors to allowable working stress design limits, and also permit local strength capacities to be exceeded by missile loadings (concentrated loads) provided that this causes no loss of function in any safety-related systems.

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

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

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

NUREG-0800, Standard Review Plan

Section 3.3.2, "Tornado Loadings" [9]
Section 3.5.1.4, "Missiles Generated by Natural Phenomena" [13]
Section 3.5.3, "Barrier Design Procedures" [18]
Section 3.8.1, "Concrete Containment" [19]
Section 3.8.4, "Other Seismic Category I Structures" [17]
Section 3.8.5, "Foundations" [20]

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

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

ASME Boiler and Pressure Vessel Code, Section III, Division 2 (ACI-359), "Standard Code for Concrete Reactor Vessels and Containments" [16]

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

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

3. TECHNICAL EVALUATION

3.1 GENERAL INFORMATION

The structures evaluated in this review are the reactor building, the high pressure coolant injection pump building, the diesel engine room, and the crib house. These structures are seismically classified as Category I, Nuclear Safety-Related. The site plan for Dresden Unit 2 is shown in Figure 1.

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

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

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

The design basis missiles are C and F from SRP Section 3.5.1.4 Missile Spectrum:

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

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

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

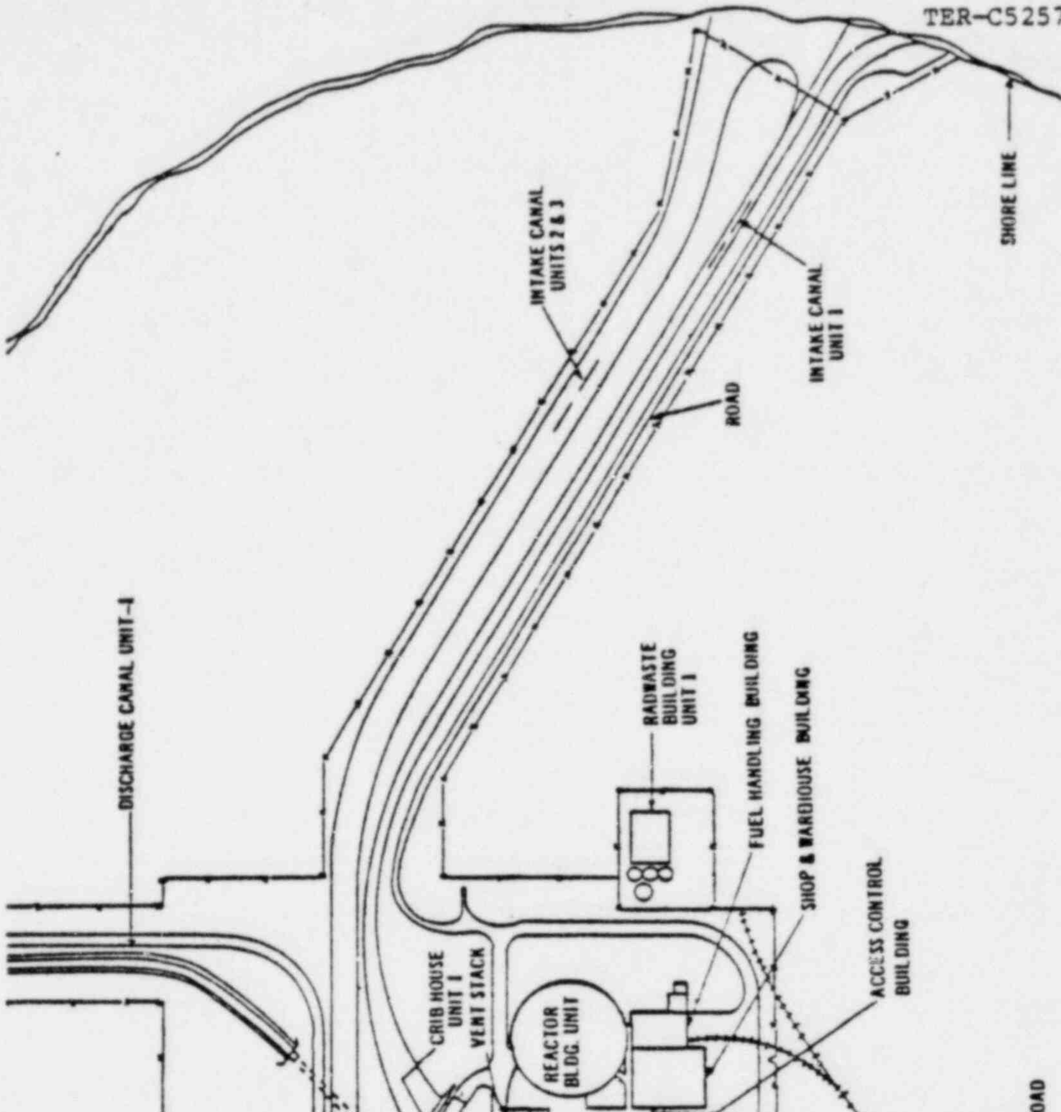

Figure 1. Site Plan

differential pressure. However, even though the roof deck failure provides venting, the tornado loads are still assumed to exist so that the strength of other, stronger structural elements can be analyzed.

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

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

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

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

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

3.2 REACTOR BUILDING

3.2.1 Evaluation

The reactor building is primarily a reinforced concrete structure with the exception of a precast concrete roof decking, which is supported by structural steel framing. The high point of this building is at elevation

658 ft 6-1/2 in, while adjacent grade is at elevation 517 ft. The reactor building can be divided into two sections: the main section, which is below the refueling floor level (elevation 613 ft), and the refueling floor enclosure (above elevation 613 ft).

The exterior of the main section consists of thick reinforced concrete walls framing into the concrete floor slabs, beams, and columns. These elements were examined for the critical load case of differential pressure loads. Each panel is assumed to transfer loads in the horizontal direction to the nearest columns and in the vertical direction to the adjacent thickened floor slabs. The analysis of the panels can be found on pages A-4 through A-6 of Appendix A.

The concrete columns of the main section were checked for stability and capacity. The critical members were chosen on the basis of the smallest section and least reinforcement. The column dead and live loads were reported by Sargent and Lundy Engineers [5]. The wind loads acting on the wall panels adjacent to the columns are transferred to the columns as reactions, producing moments in the columns. The columns are assumed to be supported between successive beam and floor slabs. The column analysis and calculations are given on pages A-7 through A-11 of Appendix A.

The refueling floor enclosure consists of insulated metal wall panels and a precast concrete slab roof supported by a framework of columns and trusses. To analyze the capacity of the roof steel, the precast concrete roof slab is assumed to remain intact. Also, when examining the capacity of the columns, it is assumed that the metal panels do not fail. The analysis of the structural steel elements of the refueling floor enclosure are presented on pages A-12 through A-24 of Appendix A. The Dresden 2 SAR [1] states that the metal wall blow-off panels will fail when a wind differential pressure of 70 psf is reached. The capacity of the metal wall panels was not reviewed.

3.2.2 Conclusion

The reinforced concrete elements of the main section can safely withstand the tornado loadings. The limiting elements of the roof steel are the 14WF30 beams, which have limit ratings of 2.08 psi (342 mph) for tornado dynamic pressure, and the roof bracing system, which has a limit rating of 294 psf (339 mph) for tornado dynamic pressure. The limiting elements of the roof supports are the east side steel columns, which have a limit rating of 65.8 psf (160 mph) for tornado dynamic pressures. The steel columns on the south side have a limit rating of 199 psf (279 mph) for tornado dynamic pressure.

3.3 CRIB HOUSE

3.3.1 Evaluation

The crib house is located north of the turbine building, and serves as an intake and discharge water supply structure. The underground portions of the structure are below the grade elevation of 517 ft. The above grade portions of the structure have a precast concrete roof with a high-side elevation of 541 ft and a low side elevation of 530 ft 6 in.

The safety-related components of the crib house are located 8 ft below ground level in an area surrounded by reinforced concrete walls 3 ft thick and a reinforced concrete grade slab 2 ft thick [1]. The structure above grade is a concrete block wall enclosure with a precast concrete slab roof supported by structural steel framing. This structure is assumed to fail during a tornado strike so that the reinforced concrete grade slab is subjected to the tornado loadings as well as to any additional loading caused by failure of the superstructure. It is assumed that the concrete block wall fails onto the concrete grade slab. The analysis of the grade slab is given on pages B-1 to B-5 of Appendix B.

3.3.2 Conclusion

The reinforced concrete grade slab at elevation 517 ft can withstand the tornado loadings as well as any additional loadings resulting from failure of the concrete block wall.

3.4 HIGH PRESSURE COOLANT INJECTION PUMP BUILDING

3.4.1 Evaluation

The high pressure coolant injection (HPCI) pump building is located south of the Dresden Unit 3 reactor building. This building houses the standby diesel generators and other safety-related equipment for Dresden Units 2 and 3. The top of the reinforced concrete roof slab is at elevation 526 ft 6 in, while adjacent grade is at elevation 517 ft. This building has two subgrade levels: the lower floor is at elevation 476 ft 6 in, and the upper floor is at elevation 504 ft 6 in.

The exterior walls of the building are made of reinforced concrete 3 ft thick. The floor slab at elevation 476 ft 6 in is a reinforced concrete slab 4 ft thick and the floor slab at elevation 504 ft 6 inches is a reinforced concrete slab 3 ft thick. The reinforced concrete roof slab is 1 ft thick and supported by reinforced concrete beams and columns.

The capacity of the roof slab and the walls have been examined to determine whether they can withstand the pressure drop caused by a tornado. The wall section was assumed to be fixed at elevation 504 ft 6 in and modeled as a cantilevered beam. The analysis of the roof slab and the wall section is given on pages C-1 to C-10 of Appendix C.

3.4.2 Conclusion

The reinforced concrete roof slab of the HPCI pump building can withstand the uplift load due to a differential pressure loading. The reinforced concrete exterior walls can safely withstand all tornado loadings.

3.5 DIESEL ENGINE ROOM

3.5.1 Evaluation

The standby diesel engine room is located in the southeast area of the turbine building. The south wall of the diesel engine room is the exterior wall of the turbine building. The other three walls are composed of reinforced concrete and are located inside the turbine building. The roof slab of the

diesel engine room is at elevation 538 ft, while the adjacent grade is at elevation 517 ft. The diesel engine room is adjacent to a rolling steel door which opens onto the interior of the turbine building. If the rolling steel door fails during a tornado, the interior walls will be exposed to differential pressures.

In determining the capacity of reinforced concrete walls, all axial loads are assumed to be transferred to the columns of the turbine building. Since the concrete walls are reinforced both vertically and horizontally, the walls have been modeled as two-way slabs. The analysis of the diesel engine room is given on pages D-1 to D-7 of Appendix D.

3.5.2 Conclusion

The reinforced concrete wall on the south side of the diesel engine room can withstand all tornado loadings. The east side wall has a limit rating of 1.15 psi (180 mph) for differential pressure.

4. CONCLUSIONS

The results of the tornado structural analysis for the reactor building, crib house, HPCI pump building, and diesel engine room are summarized below in Table 2.

Table 2. Strength Summary of the Structural Components Analyzed

Structure	Element*	Cause of Failure**	Wind Speed (mph)
Reactor Building			
a. Above refueling floor elevation 613 ft	14W30 roof beams	1	342
	Roof bracing system	1	339***
	East side steel columns	1	160
	South side steel columns	1	279
b. Below refueling floor elev 613 ft.	Reinforced concrete walls and columns	-	-
Crib House	Reinforced concrete roof slab	-	-
HPCI Pump Building	Reinforced concrete roof slab and walls	-	-
Diesel Engine Room	South side reinforced concrete wall	-	-
	East side reinforced concrete wall	2	180

*The first element identified for each structure is the limiting element. Additional elements found to be inadequate are subsequently listed. Note that this table does not imply that all inadequate elements have been identified or that entries are listed with respect to the most critical loading combination. Structural details not included in this review are windows, doors, and roof decks.

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

***The strength listing is based on the integrated capacity and action of all components in the roof bracing system.

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NUREG-0800

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TER-C5257-400

APPENDIX A

REACTOR BUILDING DESIGN REVIEW CALCULATIONS

Franklin Research Center
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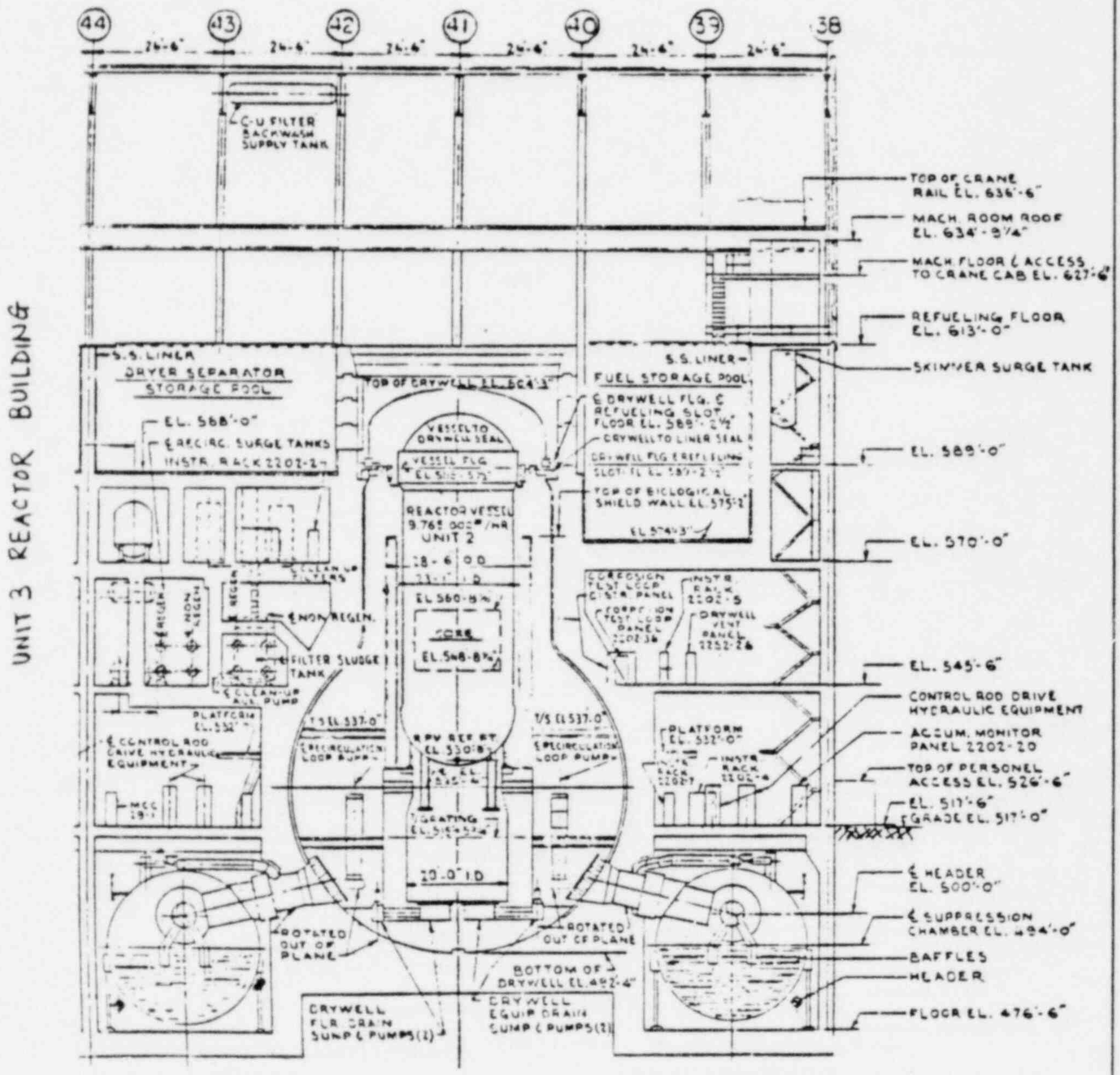
Project	Page
02G-C5257-01	A-1

By	Date	Ch'k'd	Date	Rev.	Date
RA	MAY '82	SJ	5-27		

Title: REACTOR BUILDING - DRESDEN UNIT 2 CROSS SECTION EAST-WEST



ALL STEEL $f_y = 60$ KSI
ALL CONCRETE $f'_c = 4$ KSI

GRADE ELEV. 517'-0"

HEIGHT BETWEEN FLOORS

IST FLOOR	517'-6"	
		28 FT.
II	545'-6"	
		24.5 FT.
III	570'-0"	
		19 FT.
IV	589'-0"	
		24 FT.
V (REFUELING FLOOR LEVEL)	613'-0"	

THE CRITICAL CASE WILL BE THE PRESSURE DROP CASE. A PRESSURE OF 432 PSF WILL ACT ON WALL COLUMNS. SOUTH SIDE AND EAST SIDE WALLS ARE DIRECTLY EXPOSED. COLUMN M38 IS ANALYZED AS IT IS THE WEAKEST SECTION.

CALCULATION OF ACTUAL MOMENT FOR WALLS

USE $M = \frac{w\ell^2}{8}$, WHERE $w = 3$ PSI $= 432$ PSF.

TAKE $\ell = 24'-6"$

FOR 9" WIDE STRIP $M = \frac{9}{12} \times \frac{432}{1000} \times \frac{(24.5)^2}{8}$

$M = 24.310125$ K-FT.

TAKE $\ell = 25'-9"$

$M = \frac{9}{12} \times 0.432 \frac{(25.75)^2}{8}$

$M = 26.854031$ K-FT.

CALCULATION OF ACTUAL MOMENT FOR COLUMNS

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

FOR PRESSURE DROP, PRESSURE ACTIVE P = 432 PSF
LATERAL WIDTH SUPPORTED BY M 38 = 25.75 FT.

$$w = 25.75 \times \frac{432}{1000}$$

$$w = 11.124 \text{ K/FT}$$

HEIGHT ℓ = 24 FT.

$$M = 11.124 \frac{(24)^2}{8}$$

$$M = 800.928 \text{ K-FT}$$

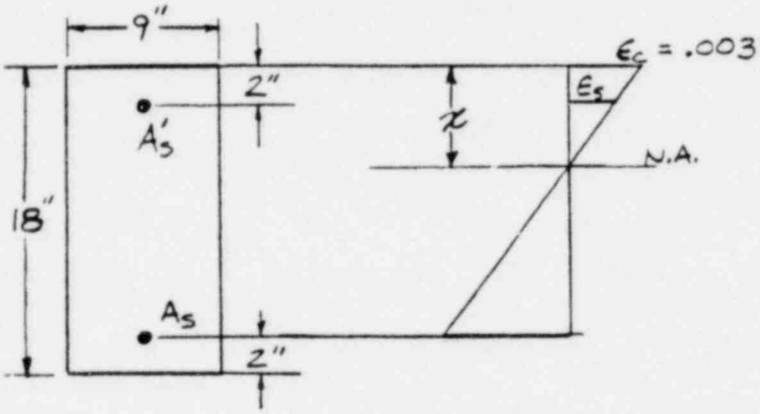
NOW FOR ℓ = 28 FT.

$$M = \frac{11.124\ (28)^2}{8}$$

$$M = 1090.152 \text{ K-FT.}$$

Title REACTOR BUILDING-LATERAL SECT. OF WALL ON SOUTH AND EAST SIDE

WALL SECTION IS 1'-6" THICK.
THE REINFORCEMENT IS #9 @ 9" IN EACH FACE OF WALL.
CONSIDER A 9" STRIP OF WALL SECTION. THIS WILL BE ANALYZED AS A BEAM SECTION.



STEEL $f_y = 60$ KSI $\qquad \epsilon_y = 2.06896 \times 10^{-3}$
CONCRETE $f_c' = 4$ KSI $\qquad \epsilon_c = 0.003$

AREA OF STEEL $\quad A_s = A_s' = 1.00 \text{ IN}^2$

ASSUME NEUTRAL AXIS AT x INCHES.

TENSION
$$T = A_s f_y$$
$$= 1.0 \times 60$$
$$T = 60 \text{ KIPS}$$

COMPRESSION CONCRETE
$$C_c = .85 f_c'(.85x)\,b$$
$$= .85 \times 4\,(.85x)\,9$$
$$C_c = 26.01x \text{ KIPS}$$

FROM STRAIN GEOMETRY
$$\frac{\epsilon_s}{(x-2)} = \frac{\epsilon_c}{x} \Rightarrow \epsilon_s = \frac{.003\,(x-2)}{x}$$

$$\therefore f_s = 29{,}000\,\epsilon_s = \frac{87\,(x-2)}{x}$$

FORM CS-FRC-81

COMPRESSION STEEL $C_S = A'_s(f_s - 0.85 f'_c)$

$$= 0.44\left(\frac{87(x-2)}{x} - 0.85\times 4\right)$$

$$= 0.44\left(\frac{87(x-2)}{x} - 3.4\right)$$

$$C_s = 38.28\frac{(x-2)}{x} - 1.496$$

FOR EQUILIBRIUM $T = C_c + C_s$

$$60 = 26.01x + 38.28\frac{(x-2)}{x} - 1.496$$

$$60x = 26.01x^2 + 38.28(x-2) - 1.496x$$

$$26.01x^2 - 23.216x - 76.56 = 0$$

$$x = \frac{23.216 \pm \sqrt{(23.216)^2 + 4\times 26.01\times 76.56}}{2\times 26.01}$$

$$x = \frac{23.216 \pm 92.218681}{2\times 26.01}$$

$$x = 2.2190442''$$

$$\therefore C_c = 26.01x$$
$$= 26.01\times 2.2190442$$
$$C_c = 57.71734 \text{ KIPS}$$

$$C_s = 38.28\frac{(x-2)}{x} - 1.496$$
$$= 38.28\frac{(2.2190442-2)}{2.2190442} - 1.496$$
$$C_s = 2.2826597 \text{ KIPS}$$

$\therefore$ MOMENT CAPACITY OF SECTION

$a = 0.85x$
$= 1.8861876$

$$M = C_c(d - a/2) + C_s(14)$$

$$M = 57.71734(16 - 0.9430933) + 2.2826597(14)$$
$$= 869.04457 + 31.957235$$
$$M = 901.00181 \text{ K-IN.}$$

$$\therefore M_{ALLOW.} = 75.083484 \text{ K-FT.}$$

FOR WALL $M_{ALLOW} > M_{ACTUAL} = 26.854$ k-ft.

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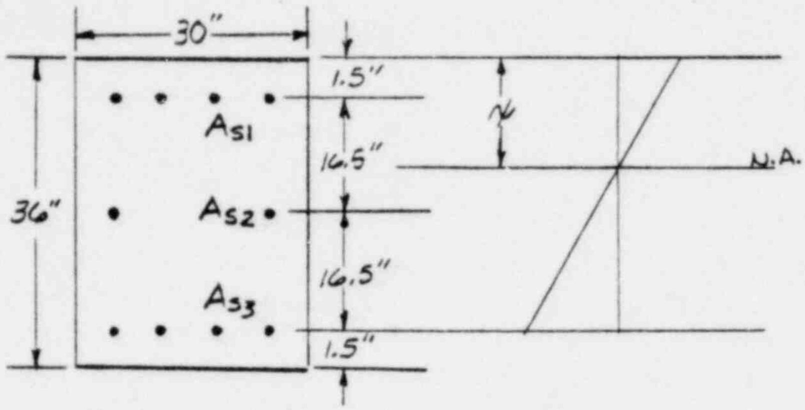
Title REACTOR BUILDING — ANALYSIS OF COLUMN (M38)

BETEEN ELEV. 613' & ELEV. 339'

NO. OF BARS 10 of #11



THIS COLUMN DESIGN FOR AN AXIAL LOAD (DL + LL) = 505 KIPS

ASSUME CONCRETE COVER OF 1.5" ON REINFORCEMENT

STEEL $f_y = 60$ KSI
CONCRETE $f'_c = 4$ KSI

FIRST ASSUME THE BALANCED CONDITION

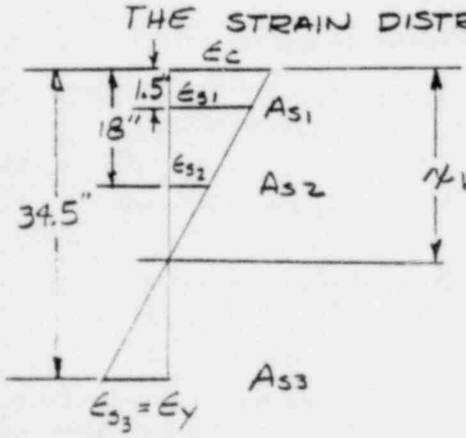
BALANCE CONDITION NEUTRAL AXIS $x_b = \dfrac{\epsilon_c d}{(\epsilon_c + \epsilon_y)}$

$d = 36 - 1.5 = 34.5$

$x_b = \dfrac{0.003 \times 34.5}{(0.003 + 2.06896 \times 10^{-3})}$

$x_b = 20.418367$

THE STRAIN DISTRIBUTION IS THEN:



$\epsilon_{s1} \geq \epsilon_y \Rightarrow f_{s1} = 60$ KSI

$\dfrac{\epsilon_{s2}}{(x_b - 18)} = \dfrac{\epsilon_y}{(34.5 - x_b)}$

$\Rightarrow \epsilon_{s2} = 1.22586 \times 10^{-3} \Rightarrow f_{s2} = 35.55$ KSI

$\epsilon_{s3} = \epsilon_y \Rightarrow f_{s3} = 60$ KSI

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NOW WE FIND THE BALANCE CONDITION ALLOWABLE P_b

COMPRESSION CONCRETE
$$C_c = 0.85 f'_c (0.85\, x_b)\, b$$
$$= 0.85 \times 4\,(0.85 \times 20.418367)\, 30$$
$$C_c = 1770.2724 \text{ KIPS}$$

COMPRESSION STEEL
$$C_s = A_{s1}(f_{s1} - 0.85 f'_c) + A_{s2}(f_{s2} - 0.85 f'_c)$$
$$= 4 \times 1.56(60 - .85 \times 4) + 2 \times 1.56(35.55 - .85 \times 4)$$
$$= 6.24(56.6) + 3.12(32.15)$$
$$= 353.184 + 100.308$$
$$C_s = 453.492 \text{ KIPS}$$

$A_{s1} = 4$ #11 BARS
$A_{s2} = 2$ #11 BARS

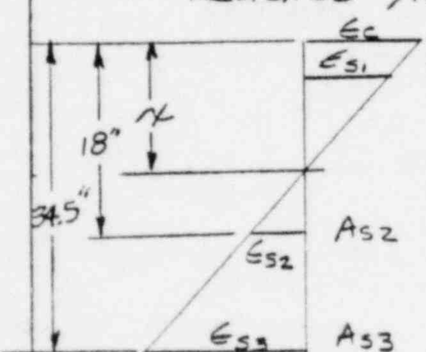
TENSION
$$T = A_{s3} f_{s3}$$
$$= 4 \times 1.56 \times 60$$
$$T = 374.4 \text{ KIPS}$$

FOR EQUILIBRIUM
$$P_b = C_c + C_s - T$$
$$= 1770.2724 + 453.492 - 374.4$$
$$P_b = 1849.3644 \text{ KIPS}$$

BUT ACTUAL DESIGN AXIAL LOAD $P_n = 505$ KIPS.

NOW FOR ACTUAL AXIAL LOAD CASE

ASSUME THAT THE EAST-WEST FACE (4 BAR SIDES) STEEL REACHES YIELD AND NEUTRAL AXIS LOCATED IN UPPER HALF.



$$\epsilon_{s1} \geq \epsilon_y \Rightarrow f_{s1} = 60 \text{ KSI}$$
$$\epsilon_{s3} \geq \epsilon_y \Rightarrow f_{s3} = 60 \text{ KSI}$$

$$\frac{\epsilon_{s2}}{(18 - x)} = \frac{\epsilon_{s3}}{(34.5 - x)}$$

$$\Rightarrow \epsilon_{s2} = \epsilon_{s3} \frac{(18 - x)}{(34.5 - x)}$$

$$\Rightarrow f_{s2} = 60 \frac{(18 - x)}{(34.5 - x)}$$

COMPRESSION CONCRETE $C_c = 0.85 f_c' (0.85 x) b$
$= 0.85 \times 4 (0.85 x) \times 30$
$C_c = 86.7 x$

COMPRESSION STEEL $C_s = A_{s1}(f_{s1} - 0.85 f_c')$
$= 4 \times 1.56 (60 - 0.85 \times 4)$
$C_s = 353.184$ KIPS

TENSION $T = A_{s2} f_{s2} + A_{s3} f_{s3}$

$$= 2 \times 1.56 \times 60 \frac{(18 - x)}{(34.5 - x)} + 4 \times 1.56 \times 60$$

$$T = 187.2 \frac{(18 - x)}{(34.5 - x)} + 374.4$$

FOR EQUILIBRIUM $P_u = C_c + C_s - T$

$$505 = 86.7x + 353.184 - 187.2 \frac{(18-x)}{(34.5-x)} - 374.4$$

$$86.7x - \frac{187.2(18-x)}{(34.5-x)} - 526.216 = 0$$

$$\Rightarrow 86.7x(34.5-x) - 187.2(18-x) - 526.216(34.5-x) = 0$$
$$2991.15x - 86.7x^2 - 3369.6 + 187.2x - 18154.452 + 526.216x = 0$$

$$86.7x^2 - 3704.566x + 21524.052 = 0$$

$$x = \frac{+3704.566 \pm \sqrt{(3704.566)^2 - 4 \times 86.7 \times 21524.052}}{2 \times 86.7}$$

$$= \frac{3704.566 \pm 2501.8529}{2 \times 86.7}$$

$$x = 6.9360616''$$

$\therefore C_c = 86.7x$
$= 86.7 \times 6.9360616$
$C_c = 601.35654$ KIPS

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Title: REACTOR BUILDING – ANALYSIS OF COLUMN M38

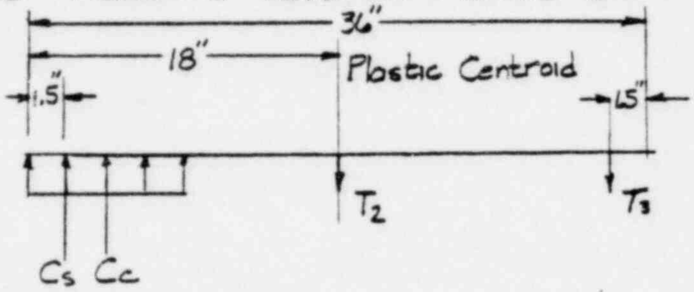
$$T = 187.2\frac{(18-c)}{(34.5-c)} + 374.4$$

$$= 75.140542 + 374.4$$

$$T = 449.54054 \text{ KIPS}$$

NOW FIND THE MOMENT CAPACITY.

THE PLASTIC CENTROID WILL BE AT MID-DEPTH FOR THIS.



TAKE THE MOMENT ABOUT THE PLASTIC CENTROID

$$M = C_c(18 - a/2) + C_s(18-1.5) + T_3(18-1.5)$$

$a = 0.85c$
$= 5.895624$ $M = 601.35654(18-2.9478262) + 353.184(16.5) + 374.4(16.5)$

$$= 9051.7232 + 5827.536 + 6177.6$$

$$M = 21056.859 \text{ K-IN.}$$

$\therefore M_{COL.ALLOW} = \underline{1754.7383 \text{ K-FT}}$

CALCULATION OF ACTUAL MOMENT

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

FOR PRESSURE DROP, PRESSURE ACTIVE $p = 432$ PSF
LATERAL WIDTH SUPPORTED BY M38 = 25.75 FT.

$$w = 25.75 \times \frac{432}{1000} = 11.124 \text{ K/FT}$$

HEIGHT $l = 24$ FT.

$$M = 11.124\frac{(24)^2}{8}$$

$M = 800.928$ K-FT

$M_{ACTUAL} < M_{COL.ALLOW.}$ $\therefore$ ✓O.K.

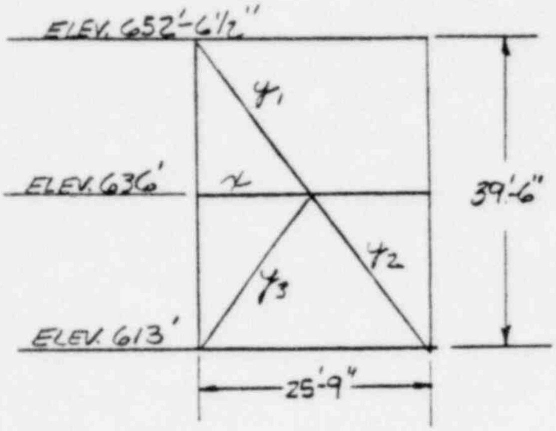
NOW FOR $\ell = 28$ FT

$$M = 11.124 \frac{(28)^2}{8}$$
$$= 1090.152 \text{ K-FT}$$

$M_{ACTUAL} < M_{COL.ALLOW.}$ $\therefore$ O.K

Title REACTOR BUILDING – STEEL ABOVE REFUELING FLOOR ELEV. 613'

12" Ø XXS PIPE



$y_1 + y_2 = 47.152015''$

SIMILAR △'s

$$\frac{16.5}{x} = \frac{39.5}{25.75}$$

$$x = \frac{16.5}{39.5} \times 25.75$$

$$x = 10.756329$$

$$y_1 = \sqrt{x^2 + 16.5^2} = 19.696411$$

$$\therefore y_2 = 27.455604''$$

$$y_3 = \sqrt{x^2 + 23^2} = 25.390916''$$

PIPE PROPERTIES

$d_o = 12.75$ IN.
$d_i = 10.75$ IN.
THICKNESS $t = 1.00$ IN.
WEIGHT = 125.49 lbs/FT.

$I = 641.7$ IN^4
$A = 36.91$ IN^2
$r = 4.17$ IN.

$$\frac{Kl}{r} = \frac{1 \times 12 \times 27.455604}{4.17} = 79.008932$$

$F_y = 36$ ksi FOR A36 STEEL

$$C_c = \sqrt{\frac{2\pi^2 E}{F_y}} = 126.09928$$

$$\frac{Kl/r}{C_c} = .6265613$$

$$F_a = \frac{\left[1 - \frac{1}{2}\left(\frac{Kl/r}{C_c}\right)^2\right] F_y}{\frac{5}{3} + \frac{3}{8}\left(\frac{Kl/r}{C_c}\right) - \frac{1}{8}\left(\frac{Kl/r}{C_c}\right)^3} = \frac{28.933577}{1.8708803}$$

12"φ PIPE

$F_a = 15.465221$ KSI

∴ ALLOWABLE AXIAL COMPRESSION LOAD = $F_a \times A$

$C_{ALLOW.} = 15.465221 \times 36.91 = 570.8213$ KIPS

$T_{ALLOW.} = 0.6 F_y \times A = 0.6 \times 36 \times 36.91$
$= 797.256$ KIPS (FOR WELDED SECTION)

2 L's 7×4×7/8

$$L = \sqrt{(24.5)^2 + (25.75)^2}$$
$$= 35.543108$$

UNBRACED LENGTH = $L/3$ = 11.847703 FT.

AREA = 17.72 IN^2
I_{xx} = 20.4 IN^4
r_{xx} = 1.07 IN
r_{yy} = 3.37 IN

$$\frac{K\ell}{r} = \frac{1 \times 11.847703 \times 12}{1.07} = 132.87143 > C_c = 126.1$$

FOR A36 STEEL

$$F_a = \frac{12\pi^2 E}{23(K\ell/r)^2} = \frac{12\pi^2 \times 29{,}000}{23(132.87143)^2} = 8.4583949 \text{ KSI}$$

(BRACING & SECONDARY MEMBERS) $F_{as} = \dfrac{F_a}{1.6 - \dfrac{\ell}{200r}} = \dfrac{8.4583944}{0.9356428} = 9.0401957$ KSI

$\therefore C_{ALLOW.} = F_{as} \times A$
$= 9.0401957 \times 17.72$
$C_{ALLOW.} = 160.19227$ KIPS

2L's 5×3×7/16

AREA $A = 6.62 \text{ IN}^2$

$r_{xx} = 0.83 \text{ IN}$

UNBRACED LENGTH = 11.847703 FT.

$$\frac{Kl}{r_{xx}} = \frac{1 \times 11.847703 \times 12}{0.83} = 171.29209 > C_c = 126.1$$

FOR A36 STEEL

$$F_a = \frac{12\pi^2 E}{23(Kl/r)^2} = \frac{12\pi^2 \times 29000}{23(171.29209)^2} = 5.0895165 \text{ KSI}$$

(BRACING & SECONDARY MEMBERS) $$F_{as} = \frac{F_a}{1.6 - \frac{l}{200r}} = \frac{5.0895165}{0.7435395} = 6.8449843 \text{ KSI}$$

$\therefore C_{ALLOW.} = F_{as} \times A$

$= 45.313796$ KIPS

2L's 5×3½×¾

AREA $A = 11.62\ \text{IN}^2$
$r_{xx} = 0.98\ \text{IN}$
UNBRACED LENGTH = 11.847703 FT.

$$\frac{Kl}{r_{xx}} = \frac{1 \times 11.847703 \times 12}{0.98} = 145.07391 > C_c = 126.1$$

FOR A36 STEEL

$$F_a = \frac{12\pi^2 E}{23\left(Kl/r\right)^2} = 7.0953289\ \text{KSI}$$

(BRACING & SECONDARY MEMBERS) $$F_{as} = \frac{F_a}{1.6 - \frac{l}{200r}} = \frac{7.0953289}{0.8746304}$$

$$F_{as} = 8.1123734\ \text{KSI}$$

$$\therefore C_{allow.} = F_{as} \times A = 8.1123734 \times 11.62$$

$$C_{allow.} = 94.265779\ \text{KIPS}$$

BEAM 14 WF 30

AREA $A = 8.83 \text{ IN}^2$

$d = 13.86$ IN

$I_{xx} = 290 \text{ IN}^4$ $\quad I_{yy} = 19.5 \text{ IN}^4$

$r_{xx} = 5.74$ IN $\quad r_{yy} = 1.49$ IN $\quad r_T = 1.75$

UNBRACED LENGTH $= 8'\text{-}2'' = 8.1667$ FT.

$$\frac{K\ell}{r_{MIN.}} = \frac{1 \times 8.1667 \times 12}{1.49} = 65.771812$$

TOTAL ALLOWABLE MOMENT FROM AISC BEAM DIAGRAM

$M = 76.8$ K-FT

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

$$w = \frac{8M}{\ell^2} = \frac{8 \times 76.8}{(24.5)^2}$$

$$= 1.023573 \text{ K/FT}$$

$$w = 1023.573 \text{ lbs./FT.}$$

$$\text{PRESSURE} = \frac{1023.573 \text{ lbs/FT}}{(8'\text{-}4\tfrac{1}{4}'')} = 121.9144 \text{ lbs/FT}^2$$

ADD SELF WT OF PRECAST CONCRETE SLAB = 15 psf

UPLIFT PRESSURE BEAM CAN TAKE $= \dfrac{121.91 \times 1.6 + 15}{0.7}$

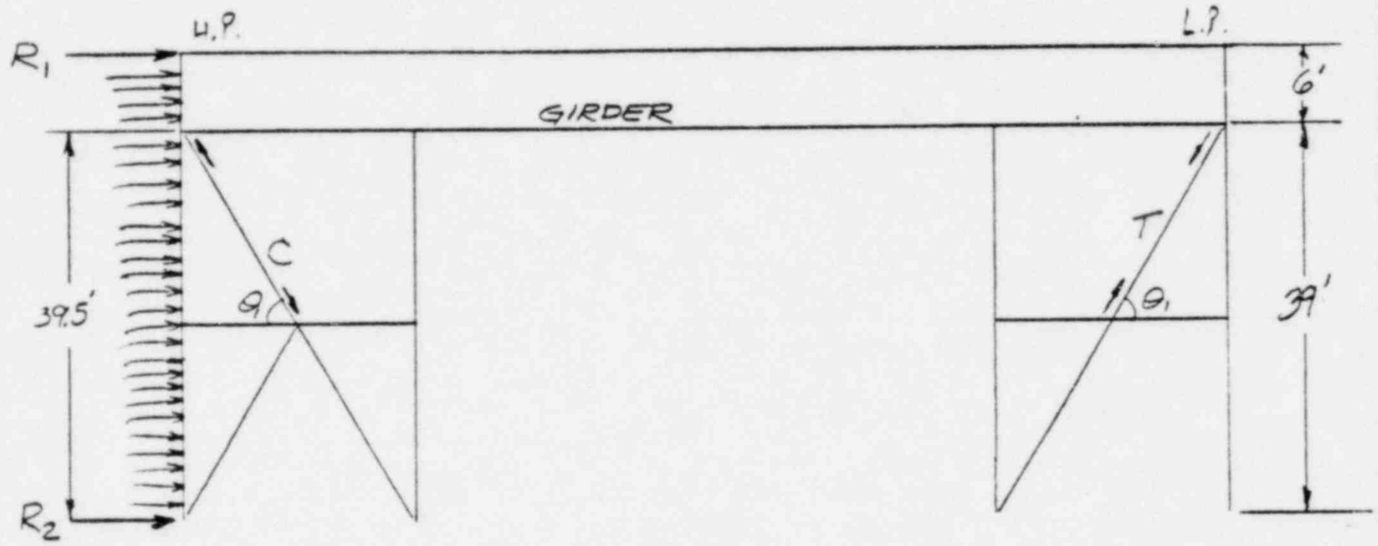
$= 300.08$ psf

EQUIVALENT WIND VELOCITY $= \sqrt{\dfrac{300.08}{0.00256}}$

$= 342.372$ mph.

Title REACTOR BUILDING — STEEL ABOVE REFUELING FLOOR ELEV. 613'

CALCULATION OF EQUIVALENT PRESSURE THAT BRACING ON EAST SIDE CAN WITHSTAND



CAPACITY OF PIPE IN COMPRESSION C = 570.8213 KIPS
" " " IN TENSION T = 797.256 KIPS

$R_1 = C\cos\theta_1 + T\cos\theta_1$

$\cos\theta_1 = \frac{x}{y_1} = \frac{10.756329}{19.696411} = 0.546106$

$\therefore R_1 = (C+T)\cos\theta_1 = (570.8213 + 797.256)\,0.546106$

$R_1 = 1368.0773 \times 0.546106 = 747.11528$ KIPS

FOR UNIFORMLY DISTRIBUTED LOAD

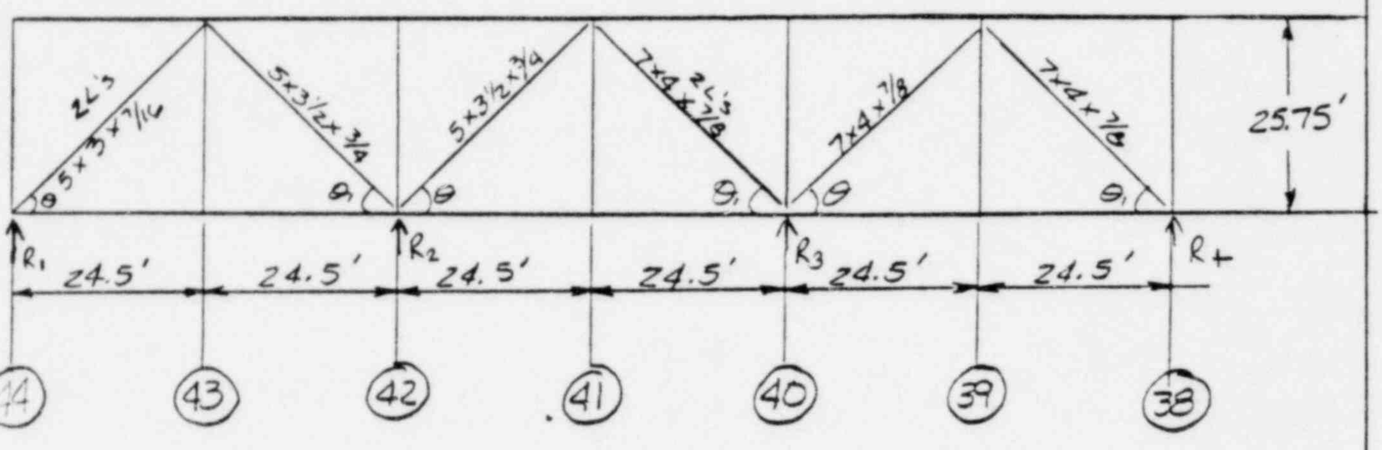
$w = \frac{747.11528}{\frac{45.5}{2}} = 32.840232$ K/FT

TO COVERT THIS TO PRESSURE

$P = \frac{32.840232}{147.5'} = .2226456$ KSF $= 222.6456$ PSF

TORNADO DYNAMIC PRESSURE $= \frac{1.6 \times P}{0.8} = \frac{1.6 \times 222.6456}{0.8}$
$= 445.2912$ psf $\simeq 417.06$ mph WIND SPEED.

Title: REACTOR BUILDING—STEEL ABOVE REFUELING FLOOR ELEV. 613'



$$TAN\theta = \frac{25.75}{24.5} = 1.0510204$$

$$TAN\theta_1 = \frac{25.75}{24.5} = 1.0510204$$

$$COS\theta = \sqrt{1/1+TAN\theta} = 0.6893038$$

R_1	2L's	5x3x7/16	45.313796 COSθ	31.234972
R_2		5x3½x¾	94.265779 COSθ	64.97776
		5x3½x¾	94.265779 COSθ	64.97776
R_3		7x4x7/8	160.19227 COSθ	110.42114
		7x4x7/8	160.19227 COSθ	110.42114
R_4		7x4x7/8	160.19227 COSθ	110.42114
				492.45391 KIPS

THIS IS HALF THE REACTION

$$PRESSURE = \frac{492.45391 \times 2 \times 1000}{(147.5') \times 45.5} = 146.75475 \text{ psF}$$

$$\text{TORNADO DYNAMIC PRESSURE} = \frac{146.75475}{0.8} \times 1.6 = 293.5095 \text{ psF}$$

EQUIVALENT ≃ 338.60323 MPH WIND SPEED

CAPACITY OF COLUMNS ON SOUTH SIDE, COLUMN ROW N
SOUTH SIDE AREA = 24'-6" × 58'-9" = 1439.375 FT²

WT. OF GIRDER

SIZE	LENGTH	WT/FT.	TOTAL WT. (lbs.)
20"×3/4"	58'-9"	51	2996.25
70"×1/2"	58'-9"	119	6991.25
20"×3/4"	58'-9"	51	2996.25
14 WF48	24'-6"	48	1176.0
14 WF30	6×24'-6"	30	4410.0
12 B14	2×58'-9"	14	1645.0
			20214.75 lbs.
			= 20.21475 KIPS

WT. OF CONCRETE SLAB @ 15 psf
= 15 × 24.5 × 58.75 = 21590.625 lbs. = 21.590625 KIPS

WT. OF STEEL + WT. OF CONCRETE = 20.21475 KIPS
+ 21.590625 KIPS
41.805375 KIPS

GIRDER DESIGNED FOR MAXIMUM END REACTION = 100 KIPS

∴ DESIGN THE COLUMNS ALONG ROW N
FOR 50 KIPS AXIAL LOAD.

W 24 × 145 @ 50 KIPS AXIAL LOAD

$$\ell = 270''$$

$$\frac{K\ell}{r_{yy}} = \frac{1 \times 270}{3.32} = 81.325 < C_c = 126.1$$

[AISC 1.5-1]
$$F_a = \frac{\left[1 - \frac{(81.325)^2}{2(126.1)^2}\right] 36}{5/3 + 3/8 \times \frac{81.325}{126.1} - \frac{1}{8}\left(\frac{81.325}{126.1}\right)^3} = 15.207 \text{ KSI}$$

$$f_a = \frac{50 \text{ KIPS}}{42.7 \text{ IN}^2} = 1.170960 2 \text{ KSI}$$

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COLUMN W24×145

$I_{xx} = 4570$ $\quad b_f = 14.043''$ $\quad t_w = 0.608''$ $\quad \frac{d}{A_f} = 1.71$

$d = 24.49''$ $\quad t_f = 1.020''$ $\quad r_T = 3.79$

FROM AISC 1.5.1.4.1

$\frac{b_f}{2t_f} = 6.88 < 8.7$ ✓

$\frac{b_f}{t_f} = 13.76 < 31.7$ ✓

$d/t_w = 40.3 < 63.55 \Leftrightarrow 68.7 - 4.4 f_a$ ✓

$\frac{76.0 b_f}{\sqrt{F_y}} = 12.7 \times 14.043'' = 178.3461 = 14.862175$ FT.

$\ell > 14.862175$ FT. N.G., MUST USE [AISC 1.5-6a] or [AISC 1.5-7]

[AISC 1.5-6a] $\quad F_b = \left[\frac{2}{3} - \frac{F_y(\ell/r_T)^2}{1530 \times 10^3 C_b}\right] F_y$ $\quad$ USE $C_b = 1.0$

$= \left[\frac{2}{3} - 0.1194153\right] F_y$

$F_b = 19.701047$ KSI

[AISC 1.5-7] $\quad F_b = \frac{12 \times 10^3 C_b}{\ell\, d/A_f} = \frac{12 \times 10^3 \times 1}{270 \times 1.71} = 25.9909$ KSI

SINCE 1.5-7 GIVES LARGER VALUE THAN 1.5-6a
AND $25.9909 > 0.6 F_y$ USE $\underline{F_b = 0.6 F_y = 22 \text{ KSI}}$

$\frac{f_a}{F_a} = \frac{1.1709602}{15.207} = 0.0770013$

$\frac{f_a}{F_a} + \frac{f_b}{F_b} \le 1.0 \Rightarrow f_b \le \left(1 - \frac{f_a}{F_a}\right) F_b = 20.305969$ KSI

ALLOW. BENDING STRESS WITH 50 KIP AXIAL LOAD = 20.305969 KSI

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COL. W 24×145

$$M = \frac{f_b I}{d/2} = \frac{20.305969 \times 4570}{24.49/2}$$

$$M = 7578.463 \text{ K-IN.}$$

$$\therefore M_{ALLOW.} = 631.53859 \text{ K-FT.}$$

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

$$l = 45.5'$$

$$w = \frac{8 \times 631.53859 \times 1000}{(45.5)^2} = 2440.4341 \text{ lbs./FT}$$

$$\text{PRESSURE} = \frac{2440.4341 \text{ lbs./FT}}{24.5 \text{ FT}} = 99.609555 \text{ PSF}$$

$$\text{TORNADO DYNAMIC PRESSURE} = \frac{99.609555 \times 1.6}{0.8} = 199.21911 \text{ PSF}$$

$\approx$ 278.9623 MPH WIND SPEED

COLUMNS ON ROW 38 SIDE

COLUMNS R_1 THRU R_4 MUST BE CHECKED FOR BENDING ONLY, WITH NO AXIAL LOAD, FOR TORNADO DYNAMIC PRESSURE, WIND BLOWING EAST-WEST.

COLUMNS ARE 24 WF 76.

UNBRACED HEIGHT = 23 FT.
AREA $A = 22.4\ IN^2$ — $I_{xx} = 2100\ IN^4$ — $I_{yy} = 82.6\ IN^4$
$d = 23.91\ IN.$ — $r_{xx} = 9.69\ IN.$ — $r_{yy} = 1.92\ IN.$
$d/A_f = 3.9$ — $r_T = 2.32\ IN.$

$$\frac{Kl}{r} = \frac{1 \times 23 \times 12}{1.92} = 143.75$$

A 36 STEEL $F_y = 36\,KSI$

$$l = 276\ IN. > \frac{76.0\ b_f}{\sqrt{F_y}} = 113.81''$$

$$l/r_T = 118.96552 \quad \text{USE [AISC 1.5-6a OR 1.5-7]}$$

[AISC 1.5-6a] $$F_b = \left[\frac{2}{3} - \frac{F_y (l/r_T)^2}{1530 \times 10^3 C_b}\right] F_y = .3336597 F_y = 12.01175\ KSI$$

[AISC 1.5-7] $$F_b = \frac{12 \times 10^3\ C_b}{l\ d/A_f} = \frac{12 \times 10^3 \times 1}{276 \times 3.9} = 11.148272\ KSI$$

USE $F_b = 12.01175\ KSI$

$$M = \frac{F_b I}{d/2} = \frac{12.01175 \times 2100}{23.91/2} = 2109.9638\ K\text{-}IN = \underline{175.83073\ K\text{-}FT.}$$

$$M = \frac{w l^2}{8}, \quad w = \frac{8 \times M}{(45.5)^2} = 0.6794569\ K/FT.$$

TAKE $l = 45.5\ FT.$

$$PRESSURE = \frac{679.4569\ \#/FT}{1/2(24'\text{-}8'' + 16'\text{-}8'')} = \frac{679.4569}{20.6667}$$

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COL. 24 WF 76

PRESSURE = 32.876951 psf

$$\text{TORNADO DYNAMIC PRESSURE} = \frac{32.876951 \times 1.6}{0.8} = 65.753903 \text{ psf}$$

$\simeq 160.26577$ MPH WIND SPEED

FORM CS-FRC-81

APPENDIX B

CRIB HOUSE DESIGN REVIEW CALCULATIONS

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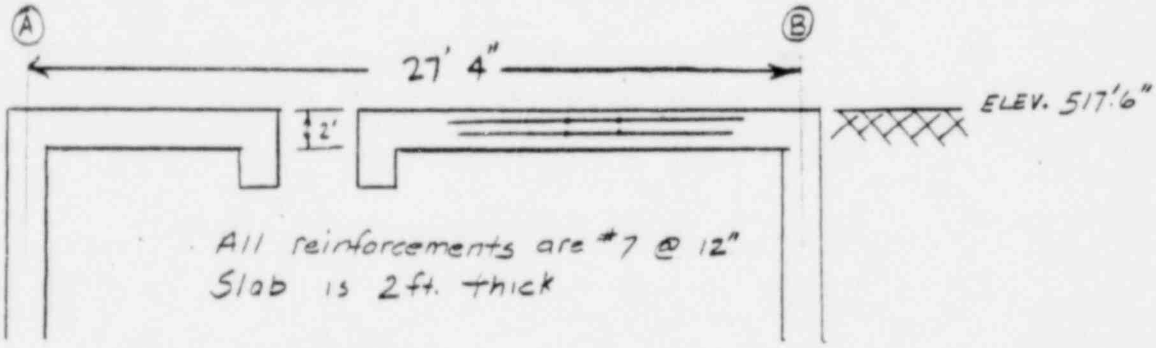
CRIB HOUSE

Concrete $f'_c = 3500$ psi
steel $f_y = 40{,}000$ psi

slab at grade (elev. 517'6") level.

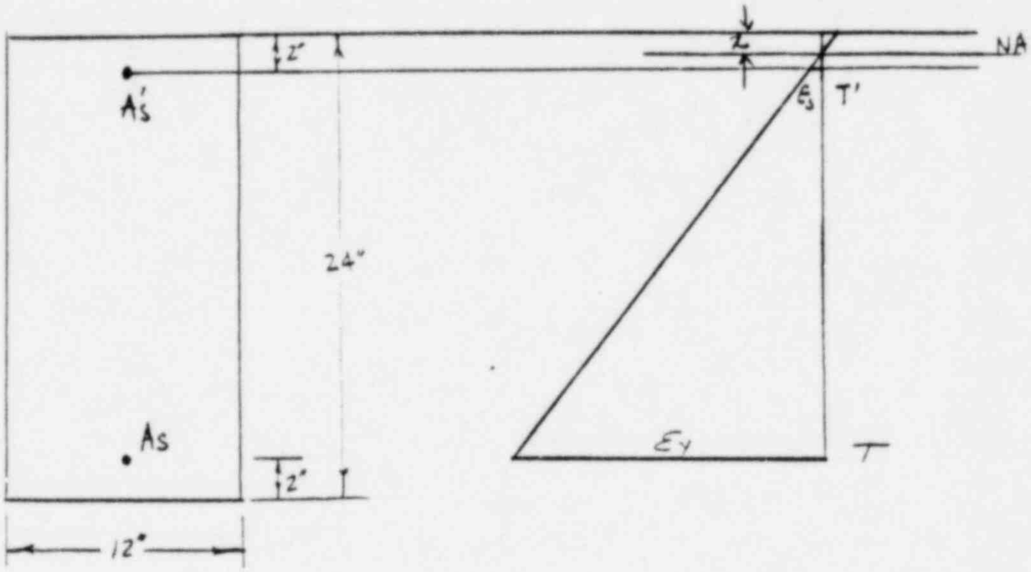
2 ft. thick concrete



Section thru pump room in the Crib House.

WE ASSUME TWO-WAY SLAB ACTION

Title CRIB HOUSE – SLAB AT GRADE LEVEL



Steel $f_y = 40$ ksi
Concrete $f'_c = 3.5$ ksi

Reinforcement Area $A_s = A'_s = 0.60 \text{ in}^2$

Assume both steel are in Tension

Compression

$$C_c = 0.85 f'_c (0.85x) b$$
$$= (0.85)(3.5)(0.85)(x)(12)$$
$$C_c = 30.345x \text{ KIPS}$$

Tension

$$T = A_s f_y$$
$$= (0.6)(40)$$
$$T = 24 \text{ KIPS}$$

$$\varepsilon_s = \frac{\varepsilon_y\,(2-x)}{(22-x)} \Rightarrow f_s = \frac{40\,(2-x)}{(22-x)}$$

$$T' = A'_s f_s$$
$$= (0.6)\left[\frac{40\,(2-x)}{(22-x)}\right]$$

$$\therefore T' = \frac{24\,(2-x)}{(22-x)}$$

For equlibrium $C_c = T + T'$

$$30.345x = 24 + \frac{24(2-x)}{(22-x)}$$

$$30.345x(22-x) = 24(22-x) + 24(2-x)$$
$$667.59x - 30.345x^2 = 576 - 48x$$

$$\Rightarrow 30.345x^2 - 715.59x + 576 = 0$$

$$x = \frac{715.59 \pm \sqrt{(715.59)^2 - 4(30.345)(576)}}{2(30.345)}$$

$$x = \frac{715.59 \pm 664.95}{60.69}$$

$$x = .834$$

$$C_c = 30.345x$$
$$= 30.345(.834)$$
$$C_c = 25.31 \text{ KIPS}$$

$$T' = \frac{24\,(2-x)}{(22-x)}$$

$$= \frac{24\,(2-.834)}{(22-.834)}$$

$$T' = 1.32 \text{ kips}$$

Now the moment capacity

$$a = (.85)x = (.85)(.834) = .709$$

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

$$= 24(22 - .3545) + 1.32(2 - .3545)$$

$$M = 521.66 \text{ k}\cdot\text{inch}$$

$$\therefore M_{allow} = 43.47 \text{ k-ft.}$$

Calculation of allowable load

$$M = \frac{wL^2}{8} \Rightarrow W = \frac{8M}{l^2}$$

$L = 27'4"$

$$= \frac{8\,(43.47)}{(27.333)^2}$$

$$w = 0.46547 \text{ k/ft}$$

$$= 465.47233 \text{ lbs/ft}$$

FOR TWO WAY SLAB, M_{ALLOW} IS THE FACTORED DISTRIBUTED MOMENT. ACCORDING TO ACI 318-77 (13.6.3) FACTORS ARE:

a) Interior negative factored moment $0.75 - \frac{0.10}{1 + 1/\alpha_{ec}}$ Range $0.75 - 0.65$

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Title: CRIB HOUSE - SLAB AT GRADE LEVEL

b) Positive factored moment $0.63 - \frac{0.28}{1 + 1/k_{ec}}$ Range (0.63 - 0.35)

c) Exterior negative factored moment $\frac{0.65}{1 + 1/k_{ec}}$ Range (0.65 - 0.0)

$\therefore$ Allowable load $= \frac{465.47293}{0.75}$

$= 620.63057$ psf.

Self wt of 2 ft. thick slab $= 300.00$

Net reserve load slab can take $= 320.63057$ psf.

Assume the wt. of masonry is 150 lb/ft^3

For 1 ft. wide section, wt. of masonry $=$ 150 lb/ft^2

Additional load on slab due to masonry block wall failure will be 150 lb/ft^2, whereas a 1 ft wide slab, section has reserve capacity of 320.63 psf.

$\therefore$ O.K.

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APPENDIX C

HIGH PRESSURE COOLANT INJECTION
PUMP BUILDING DESIGN REVIEW CALCULATIONS

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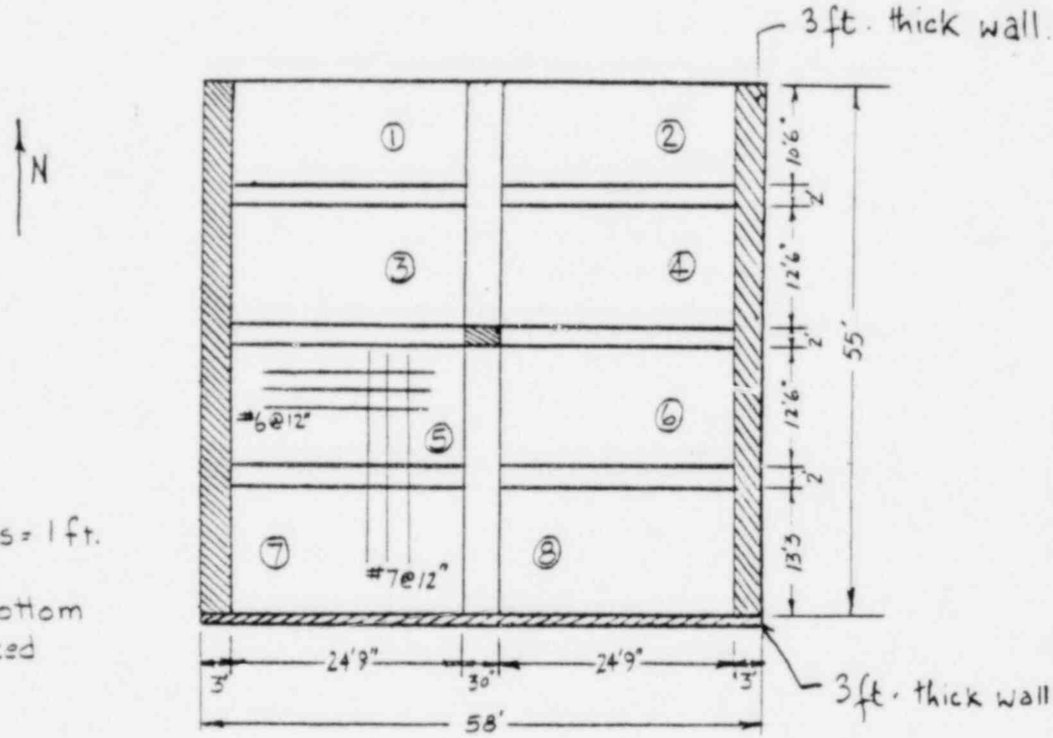
HPCI ROOM

Roof at 526'6"
Plant grade 517'6"



thickness = 1 ft.

Top & Bottom Reinforced

Steel: $f_y = 60$ ksi
concrete: $f'_c = 4$ ksi
} General Notes: Drawings B-200, B-626

Ratio of $\frac{\text{larger size}}{\text{smaller size}} = \frac{24.75}{13.25} = 1.87 < 2$

Design the slab as two-way slab supported on Beams & Girders

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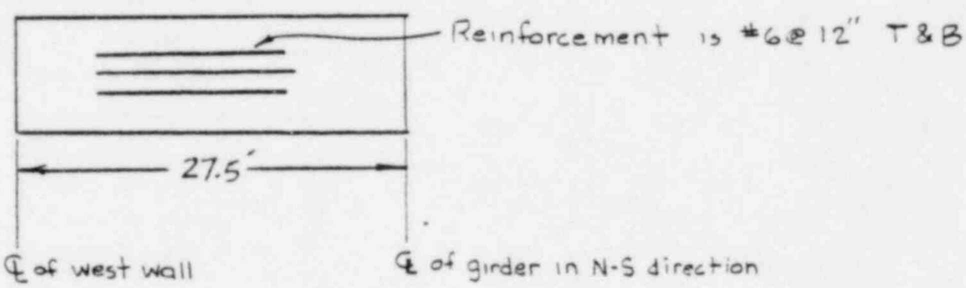
Title: HPCI PUMP BUILDING — ROOF SLAB SECTION

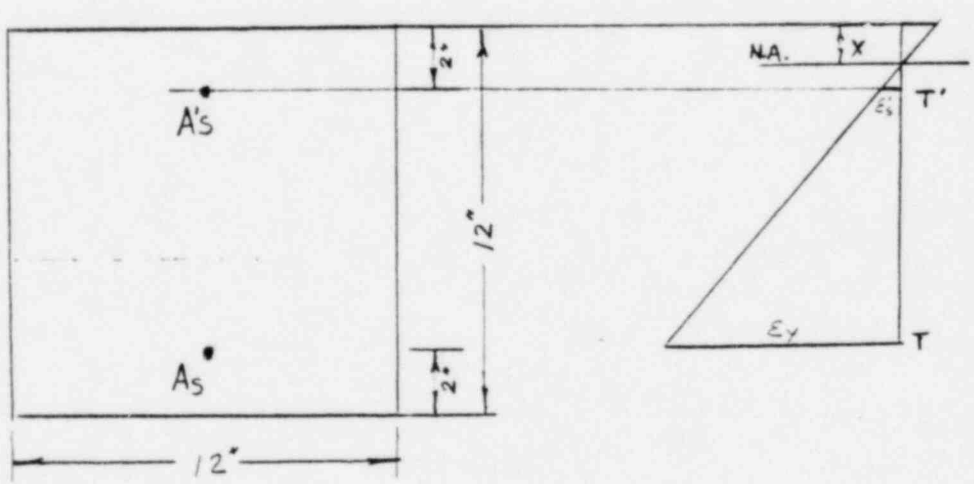
Pressure drop = 3 psi = 432 psf uplift

Dead weight of slab = 150 lbs/ft^2 per ft. width

Consider a 1ft. wide strip in E-W direction.



Net uplift load = 432 - 150 = 282 psf



Area of steel $A_s = A'_s = 0.44\ in^2$

Assume Tension in both bars

$$T = A_s f_y$$
$$= (0.44)(60)$$
$$T = 26.4 \text{ kips}$$

From strain geometry:

Use $\varepsilon_s = \dfrac{0.003(2-x)}{x}$

$$\therefore \quad f_s = 29{,}000\,\varepsilon_s = \frac{87}{x}(2-x)$$

$$T' = A'_s f_s$$
$$= (0.44)\left[\frac{87(2-x)}{x}\right]$$
$$= \frac{38.28(2-x)}{x}$$

Compression:

$$C_c = 0.85 f'_c\,(0.85x)\,b$$
$$= (0.85)(4)(0.85x)(12)$$
$$\therefore C_c = 34.68\,x$$

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

$$34.68\,x = 26.4 + \frac{38.28(2-x)}{x}$$

$$34.68\,x^2 = 26.4\,x + 38.28(2-x)$$

$$0 = 34.68\,x^2 - 11.88\,x + 76.56$$

$$\Rightarrow x = \frac{-11.88 \pm \sqrt{(11.88)^2 + 4(34.68)(76.56)}}{2(34.68)}$$

$$= \frac{-11.88 \pm 103.74}{69.36}$$

$x = 1.32$ inches

$$C_c = 34.68x$$
$$= 34.68(1.32)$$
$$= 45.93 \text{ KIPS}$$

$$T' = \frac{38.28\,(2-x)}{x}$$
$$= \frac{38.28\,(2-1.32)}{1.32}$$
$$T' = 19.72 \text{ KIPS}$$

Now Moment Capacity

$$a = 0.85x = 0.85(1.32) = 1.122$$

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

$$\therefore M = 26.4(10-.561) + 19.72(2-.561)$$
$$= 249.19 + 28.38$$
$$M = 277.57 \text{ K-inch}$$

$$M_{Allow} = 23.13 \text{ K-ft.}$$

$$\text{Actual Moment} = \frac{wl^2}{8} = \frac{282}{1000}\,\frac{(27.5)^2}{8}$$

$$M_{actual} = 26.66 \text{ K-ft.}$$

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Title HPCI PUMP BUILDING – ROOF SLAB SECTION

Distribute the Moment as follows (ACI 318 - 13.6.3)

		Range
a) Interior negative factored moment	$0.75 - \dfrac{0.10}{1+\frac{1}{\alpha_{ec}}}$	(0.75 - 0.65)
b) Positive factored moment	$0.63 - \dfrac{0.28}{1+\frac{1}{\alpha_{ec}}}$	(0.63 - 0.35)
c) Exterior negative factored moment	$\dfrac{0.65}{1+\frac{1}{\alpha_{ec}}}$	(0.65 - 0)

$0.75\, M_{actual} = 0.75 \times 26.66 = 20.0$

$\therefore 0.75\, M_{actual} < M_{allow}$

$\therefore$ ok

Wall Section HPCI Pump Bldg.

3ft. thick wall on all sides

Vertical Reinforcement #8 @ 12" E.F.

Lets assume that loading on 22ft of wall above elevation 504'6"

360 mph → Pressure = $0.00256\, V^2 = 331.776$ psf

Assume fixed at elevation 504'6"

$$M_{actual} = \frac{Wl^2}{2}$$

on 1ft. of section $M_{actual} = \frac{331.776}{1000} \frac{(22)^2}{2}$

$\therefore M_{actual} = 80.3$ k·ft

For diff. presure drop case Pressure = 3 psi
= 432 psf

on 1ft. of section $M_{actual} = \frac{432}{1000} \frac{(22)^2}{2}$

$\therefore M_{actual} = 104.5$ k·ft.

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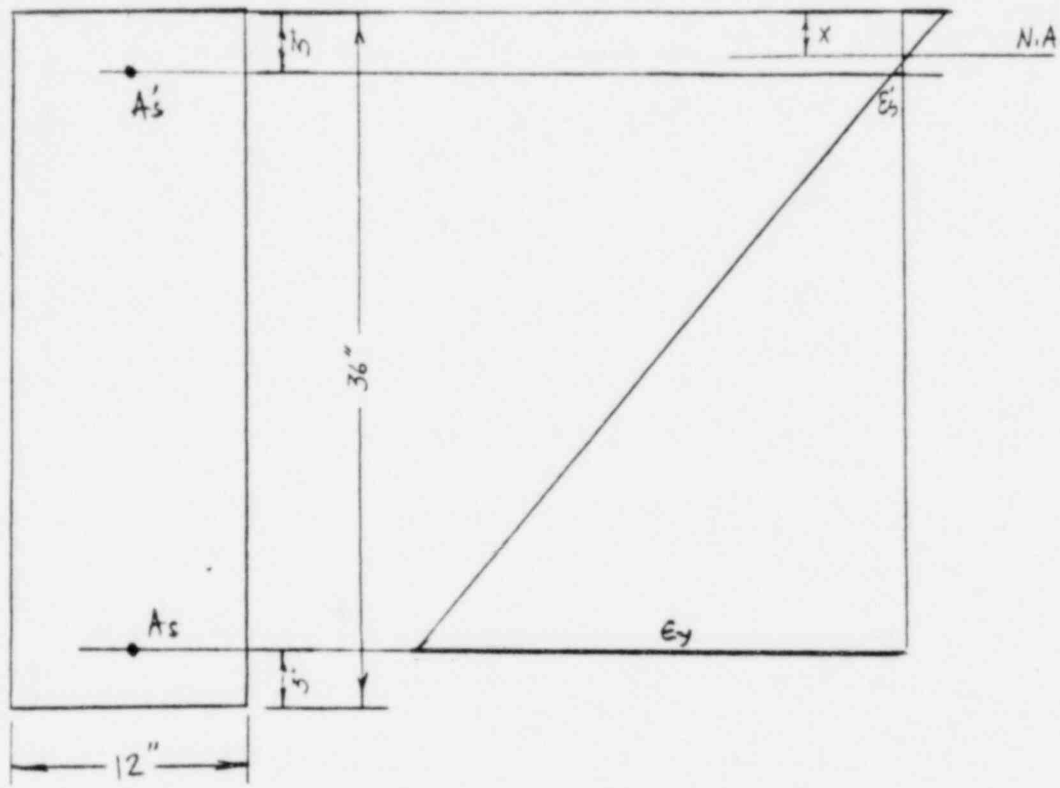
Title HPCI PUMP BUILDING — WALL SECTION



steel $f_y = 60$ KSI
concrete $f'_c = 4$ KSI

all reinforcements are # 8 bars

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

Axial load will only be self weight of 22' height

$$P_a = (22)(3)(1)\left(\frac{150}{1000}\right) = 9.9 \text{ KIPS}$$

Design for 10 KIPS

Assume both steel are in tension

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Title: HPCI PUMP BUILDING — WALL SECTION

Compression:

$$C_c = 0.85 f'_c (0.85x) b$$
$$= (0.85)(4)(0.85x)(12)$$
$$C_c = 34.68x \text{ KIPS}$$

Tension:

$$T = A_s f_y$$
$$= (0.79)(60)$$
$$T = 47.4 \text{ KIPS}$$

$$\varepsilon_s = \frac{\varepsilon_y (3-x)}{(33-x)}$$

$$\Rightarrow f_s = \frac{60(3-x)}{(33-x)}$$

$$T' = A'_s f_s$$
$$= (0.79)\left[\frac{60(3-x)}{(33-x)}\right]$$

$$T' = \frac{47.4(3-x)}{(33-x)}$$

For equlibrium

$$P_a = C_c - (T + T')$$

$$10 = 34.68x - 47.4\left[1 + \frac{(3-x)}{(33-x)}\right]$$

$$10(33-x) = 34.68x(33-x) - 47.4[(33-x) + (3-x)]$$
$$330 - 10x = 1144.44x - 34.68x^2 - 1706.4 + 94.8x$$

$$34.68x^2 - 1249.24x + 2036.4 = 0$$

$$x = \frac{1249.24 \pm \sqrt{(1249.24)^2 - 4(2036.4)(34.68)}}{2(34.68)}$$

$$x = \frac{1249.24 \pm 1130.54}{69.36}$$

$x = 1.71$ inches

$$C_c = 34.68\,x$$
$$= 34.68\,(1.71)$$
$$C_c = 59.3 \text{ kips}$$

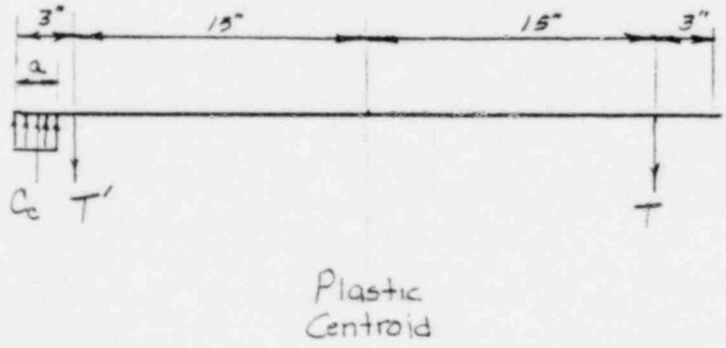
$$T' = \frac{47.4\,(3-x)}{(33-x)}$$

$$= \frac{47.4\,(3-1.71)}{(33-1.71)}$$

$$T' = 1.95 \text{ kips}$$

Now the moment capacity

For similar sections take the plastic centroid at the mid-depth of the section



$a = 0.85x = (0.85)(1.71) = 1.454$

Moment Capacity: $M = T(15) + C_c(18 - a/2) - T'(15)$

$$M = 47.4(15) + 59.3(18 - .727) - 1.95(15)$$
$$= 711 + 1024.29 - 29.25$$

$$M = 1706.04 \text{ k-inch}$$

$$\therefore M_{allow} = 142.17 \text{ k-ft}$$

$\therefore M_{allow} > M_{actual}$ for all cases

APPENDIX D

DIESEL ENGINE ROOM DESIGN REVIEW CALCULATIONS

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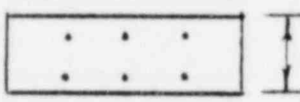
DIESEL ENGINE ROOM

Between Elevations 517'6" & 538'

Height = 538 - 517.5 = 20.5'

The south side of the wall is 19' in span.



1'6" thick

Vertical Reinforcement is #6 @ 12"
Horizontal Reinforcement is #8 @ 12" — Drawings B138, B173

East & West side walls are 1 ft. thick with both reinforcements as #6 @ 12"

East side wall span is 30'
West side wall span is 27'

ASSUME ALL LOAD IS TRANSFERED IN VERTICAL DIRECTION.

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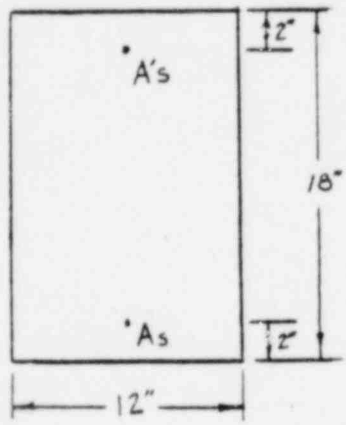
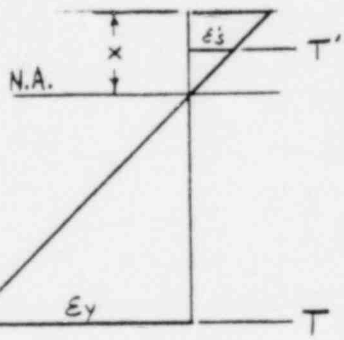
Title: DIESEL ENGINE ROOM IN TURBINE BUILDING — WALL SECTIONS

A 1 ft wide section in the south wall





B-83 Steel: $f_y = 40$ ksi
Concrete: $f'_c = 3$ ksi

Reinforcement area $A_s = A'_s = 0.44 \text{ in}^2$

All axial load taken by steel columns.

Design for flexural load only.

Assume both steel are in tension

Compression:

$$C_c = 0.85 f'_c (0.85x) b$$
$$= (0.85)(3)(0.85x)(12)$$
$$C_c = 26.01x \text{ KIPS}$$

Tension:

$$T = A_s f_y$$
$$= (0.44)(40)$$
$$T = 17.6 \text{ KIPS}$$

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$$\varepsilon_s = \frac{\varepsilon_Y (2-x)}{(16-x)} \Rightarrow f_s = \frac{40(2-x)}{(16-x)}$$

$$T' = A's\, f_s$$
$$= (0.44)\left[\frac{40(2-x)}{(16-x)}\right]$$

$$T' = \frac{17.6(2-x)}{(16-x)}$$

For equlibrium $C_c = T + T'$

$$\therefore\ 26.01x = 17.6 + \frac{17.6(2-x)}{(16-x)}$$

$$26.01x(16-x) = 17.6(16-x) + 17.6(2-x)$$

$$416.16x - 26.01x^2 = 17.6(18-2x)$$

$$\rightarrow\ 0 = 26.01x^2 - 451.36x + 316.8$$

$$x = \frac{451.36 \pm \sqrt{(451.36)^2 - 4(26.01)(316.8)}}{2(26.01)}$$

$$= \frac{451.36 \pm 413.24}{52.02}$$

$$\therefore\ x = 0.73''$$

$$\therefore\ C_c = 26.01x$$
$$= 26.01(0.73)$$
$$= 19.0 \text{ KIPS}$$

$$T' = \frac{17.6(2-x)}{(16-x)}$$
$$= \frac{17.6(2-0.76)}{(16-0.76)}$$
$$T' = 1.43 \text{ KIPS}$$

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Title: DIESEL ENGINE ROOM IN TURBINE BUILDING – WALL SECTIONS

Now the moment capacity of the section

$a = 0.85x = 0.85(0.76) = .646$

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

$= 17.6(16 - .323) + 1.43(2 - .323)$

$= 275.92 + 2.4$

$M = 278.32$ k-inch

$\therefore M = 23.19$ k-ft

$M_{allow} = 23.2$ k-ft

calculation of actual moment, south wall side

$M_{actual} = \frac{WL^2}{8}$

$= \frac{0.432(20.5)^2}{8}$

$= 22.69$ k-ft

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

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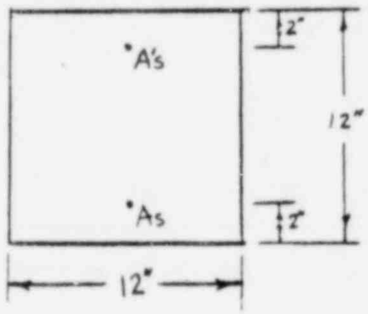
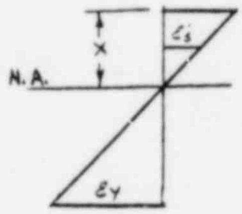
Title: DIESEL ENGINE ROOM IN TURBINE BUILDING — WALL SECTIONS

Incase the rolling steel door fails, The east wall will be exposed

Now the 1ft thick section in the wall (E & W side)





Reinforcement Area $A_s = A'_s = 0.44\ in^2$

Assume both steel are in tension

Compression:

$$C_c = 0.85\, f'_c\, (0.85x)\, b$$
$$= (0.85)(3)(0.85x)(12)$$
$$C_c = 26.01\, x$$

Tension:

$$T = A_s f_y$$
$$= (0.44)(40)$$
$$T = 17.6 \text{ KIPS}$$

$$\varepsilon_s = \frac{\varepsilon_y (2-x)}{(10-x)} \Rightarrow f_s = \frac{40(2-x)}{(10-x)}$$

$$T' = A'_s f_s$$
$$= 0.44 \left[\frac{40(2-x)}{(10-x)}\right]$$

$$= \frac{17.6(2-x)}{(10-x)}$$

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For equlibrium $C_c = T + T'$

$$26.01\,x = 17.6 + \frac{17.6\,(2-x)}{(10-x)}$$

$$26.01\,x\,(10-x) = 17.6\,(10-x) + 17.6\,(2-x)$$

$$260.1\,x - 26.01\,x^2 = 211.2 - 35.2\,x$$

$$\rightarrow \quad 0 = 26.01\,x^2 - 295.3\,x + 211.2$$

$$x = \frac{295.3 \pm \sqrt{(295.3)^2 - 4(26.01)(211.2)}}{2\,(26.01)}$$

$$x = \frac{295.3 \pm 255.4}{52.02}$$

$$x = 0.77 \text{ in.}$$

$$\therefore \; C_c = 26.01\,x$$
$$= 26.01\,(0.77)$$
$$= 20.02 \text{ KIPS}$$

$$T' = \frac{17.6\,(2-x)}{(10-x)}$$

$$= \frac{17.6\,(2-0.77)}{(10-0.77)}$$

$$T' = 2.34 \text{ KIPS}$$

$$a = 0.85\,x = 0.85\,(0.77) = 0.65$$

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

$$= 17.6\,(10 - .65) + 2.34\,(2 - .65)$$

$$= 164.56 + 3.16$$

$$M = 167.72 \text{ k-inch}$$

$$\therefore M = 13.98 \text{ k-ft}$$

M_{allow} = 14.0 k-ft. IN VERTICAL & HORIZONTAL DIRECTION.

ASSUME A TWO-WAY FLAT PLATE ACTION
HORIZONTAL SPAN 30ft.
VERTICAL SPAN 22.5ft.

AS PER ACI 318-77 (13.6.3) TWO-WAY SLABS, M_{ALLOW} IS THE FACTORED DISTRIBUTED MOMENT. DISTRIBUTION FACTORS ARE

a) Interior Negative factored moment $0.75 - \dfrac{0.10}{1 + 1/\alpha_{ec}}$ Range (0.75 – 0.65)

b) Positive factored moment $0.63 - \dfrac{0.28}{1 + 1/\alpha_{ec}}$ Range (0.63 – 0.35)

c) Exterior negative factored moment $\dfrac{0.65}{1 + 1/\alpha_{ec}}$ Range (0.65 – 0.0)

Horizontal span, more critical

$\therefore$ Allowable pressure (For 1ft wide section) $w = \dfrac{8 \times M}{0.75\, l^2} = \dfrac{8 \times 13.98 \times 1000}{0.75\,(30)^2}$

$$w = 165.69 \text{ psf}$$

For Differential pressure, this will be equivalent to wind velocity $= \sqrt{\dfrac{165.69}{0.00511}}$

$$= 180.063 \text{ mph}$$