Attachment A

S&A Calculation No. 91C2672-C007, Revision 1

Pilgrim IPEEE Building Structures Fragility Analysis

EDG Building

Client: BECo

Calculation No. 91C2672-C007

Title: PILGRIM IPEEE, BUILDING STRUCTURES FRAGILITY ANALYSIS

Project: BECo IPEEE & A-46

Method: HAND CALCULATION

Acceptance Criteria: AISC, 8-th ED., ACI 318-89, FSAR, EPRI NP-6041

Remarks:

REVISIONS

No.	Description	By	Date	Chk.	Date	App.	Date
0	Initial Issue	KC/S.C.	6/30/93	S.C./KC	6/30/93	[illegible]	7-2-93
1	Comments by RPK	KC/S.C.	8-5-93	S.C./KC	8-5-93	[illegible]	9-10-93

S&A Stevenson and Associates

CALCULATION COVER SHEET

CONTRACT NO. 91C2672

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Objective

The objective of this calculation is to determine the Conservative Deterministic Failure Margin (CDFM) and the Median Fragility for Class I structures of the Reactor Building, Turbine Building, Radwaste Building, Intake Structure, and Diesel Generator Building at BECO Pilgrim Nuclear Power Station.

The CDFM and Median Fragility values are calculated implementing the methodology presented in References (3), (4), (9), (10), (11) and the numerical data from References (1), (2), (5) through (8).

The list of Class I structures chosen for this calculation has been made with the assumption that all essential safety related structural components, except for Diesel Generator Building, were properly selected and analyzed by others (see list of References).

List of essential structures and related analytical information for the Reactor Building has been taken from FSAR (Ref. 2), App. C, Tables C.2-1 through C.2-7. Seismic analyses of shear walls in Turbine and Radwaste Buildings prepared by Bechtel have been used for evaluation of these two buildings . Excerpts from these calculations (Ref. 7) are included as Attachments. CYGNA calculation (Ref. 5) has been used for evaluation of the Intake Structure. In the Diesel Generator Building the evaluation of critical structures has been based upon new seismic analysis included into this calculation.

A list of evaluated Class I structures along with corresponding CDFM and Median Fragility values is presented in the "Summary of Results".

References:

1. Stevenson & Associates, "Floor Response Spectra for Pilgrim Nuclear Power Station, IPEEE", Report No. S &A 91C2672, R-001, Rev. 0, February 1993. SUDDS/RF 93-180

2. Boston Edison, "Final Safety Analysis Report for the Pilgrim Station", 1969

3. EPRI (Electric Power Research Institute), NP-6041 SL "A Methodology for Assessment of Nuclear Power Plant Seismic Margin", Rev. 1, Final Report, Aug. 1991.

4. Building Code Requirements for Reinforced Concrete, ACI 318-89.

5. CYGNA, "Calculation of Seismic Margins for the Pilgrim Nuclear Power Station Intake Structure", BECo Calc No. C15.0.2218, Rev. 0, Nov. 1987.

6. Stevenson & Associates, "Finite Element Model for Dynamic Analysis of the Diesel Generator Building at Pilgrim Station", Contract No. 92C2749, Rev. 0, Jan. 1993. SUDDS/RF 93-187

7. Bechtel Eng. Corporation, Boston Edison Co. Pilgrim Station # 600 Calculations.

8. Bechtel Eng. Corporation, "Boston Edison Co., Pilgrim Station # 600". Structural (C), Architectural (A), and Mechanical (M) Drawings.

9. Stevenson & Associates, "Containment Fragility ", Job No. 91C2672, Calc. No. C-008, Rev. 0, 1993.

10. AISC Steel Construction Manual, 9-Th Edition.

11. Review Comments on Pilgrim CDFM Capacity Analyses, R.P. Kennedy, July 20, 1993.

ANALYTICAL APPROACH

I. For two identical dynamic models used for the SSE and RLE analyses a relationship between corresponding stress components in a shear wall at a specific elevation can be approximated as following:

$$\frac{\sum_{k}^{n}(W_A \cdot S_{A,SSE})}{\sigma_{C,SSE}} = \frac{\sum_{k}^{n}(W_A \cdot S_{A,RLE})}{\sigma_{C,RLE}} \qquad \text{where}$$

k and n are numbers of nodes related to the highest model elevation and the elevation where stress component $\sigma_{C,SSE}$ was calculated in Bechtel calculations.

Then $$\sigma_{C,RLE} = \sigma_{C,SSE}\frac{\sum_{k}^{n}(W_A \cdot S_{A,RLE})}{\sum_{k}^{n}(W_A \cdot S_{A,SSE})}$$ and per Ref. 9:

$$CDFM\,PGA = F_\mu \frac{\sigma_U}{\sigma_{C,RLE}} S_{G,RLE} = F_\mu \frac{\sigma_U}{\sigma_{C,SSE}} \frac{\sum_{k}^{n}(W_A \cdot S_{A,SSE})}{\sum_{k}^{n}(W_A \cdot S_{A,RLE})} S_{G,RLE}$$

In these equations :

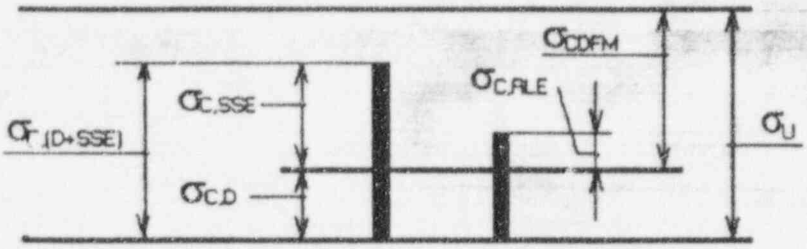
- F_μ - ductility factor
- σ_U - ultimate stress (or ultimate capacity of the structure) calculated per Ref. 3,4,10
- W_A - (DL+LL) responsible for the seismic load and related to one elevation
- $S_{A,SSE}$ - controlling acceleration for the structure due to SSE event
- $S_{A,RLE}$ - same but due to Review Level Earthquake (RLE) event
- $\sigma_{C,SSE}$ - calculated stress due to SSE event
- $\sigma_{C,RLE}$ - same due to RLE event
- $S_{G,RLE}$ - PGA of the IPEEE RLE $\qquad S_{G,RLE} = 0.4$ g

Numerical input data for the $\left[\frac{\sum_{k}^{n}(W_A \cdot S_{A,SSE})}{\sum_{k}^{n}(W_A \cdot S_{A,RLE})}\right]$ part of the equation for CDFM values are shown on pages 8 through 10.

II. When applicable the level of conservatism in the stress scaling procedure shown above can be reduced by dividing the total stress value into two components as follows:

$\sigma_{C,(D+SSE)} = \sigma_{C,D} + \sigma_{C,SSE} = \varphi W(1 + 2/3 \times S_{A,SSE})$

(vertical seismic load component is equal to 2/3 of horizontal seismic load)



$$\sigma_{C,RLE} = \sigma_{C,SSE}\frac{S_{A,RLE}}{S_{A,SSE}}$$

$$CDFM = F_\mu\frac{\sigma_U - \sigma_{C,D}}{\sigma_{C,RLE}} = F_\mu\frac{\sigma_U - \sigma_{C,D}}{\sigma_{C,SSE}}\frac{S_{A,SSE}}{S_{A,RLE}} = \frac{\sigma_U - \sigma_{C,D}}{\sigma_{C,(D+SSE)} - \sigma_{C,D}}\frac{S_{A,SSE}}{S_{A,RLE}}F_\mu$$

$$CDFM\,PGA = F_\mu\frac{\sigma_U - \sigma_{C,D}}{\sigma_{C,(D+SSE)} - \sigma_{C,D}}\frac{S_{A,SSE}}{S_{A,RLE}}0.4g$$

Note: $\sigma_{C,D}$ includes (dead load + live load).

III. The Median Fragility values are calculated based upon the CDFM values (see Ref. 9):

$$HCLPF_{50}(e^{1.65(\beta_u+\beta_r)}) = \frac{CDFM}{1.34}(e^{2.3\beta_c})$$ where β_c=0.46 for preliminary screening

IV. Values of $\sigma_{C,SSE}$ for structural components in Reactor Building have been taken from FSAR (Ref. 2). For the other buildings $\sigma_{C,SSE}$ values have been either taken from various Bechtel calculations (Ref. 7) or calculated. All these stresses as they were calculated are working stresses with one exception for shear stresses in concrete walls of Reactor Building. These shear stresses originally were calculated per Working Stress Design Method, and then they were converted with the use of some factors into values compatible with ultimate strength of concrete (these converted stresses are shown in Table C.2-3 of FSAR). For the purpose of this calculation these stresses have been converted back into working stresses with the use of the same factors.

V. In some cases the actual splices of reinforcement bars (in accordance with the drawings) lack to satisfy the criteria of ACI 318-89. The effect of this deficiency has been taken into consideration differently in different cases:

1. Ultimate shear capacity of a shear wall.

If a shear diagonal tension crack in a wall in the vicinity of a short splice crosses only a few bars while most of them remain fully effective, the wall remains ductile as most of the steel is able to reach its yield capacity, and inelastic energy absorption factor F_μ = 1.25 unless friction capacity cannot be considered adequate (page L-6 of Ref. 3). Since reinforcement lacks to satisfy the splice length requirement steel contribution to the wall shear strength should be reduced. The reduction factor of .9 is used in this calculation for shear walls unless otherwise noted.

2. Tension in reinforcement due to bending.

a) When tension in reinforcement is critical for CDFM values (see Diesel Generator Building) the CDFM tensile strength f_{sc} for bars with short splice length is to be calculated in accordance with Ref. 11:

$$f_{sc} = \frac{\phi_B\sqrt{f'_{cc}}}{A_b}\left[6.67 l_d(C_{min} + 0.5 d_b)\left(0.92 + 0.08\frac{C_{max}}{C_{min}}\right) + 300 A_b\right]$$ where

d_b = the bar diameter
A_b = the bar area
l_d = the bar actual splice length
C_{min} = the smaller of C_b (cover) or C_s (1/2 of the clear spacing)
C_{max} = the larger of C_b or C_s
f'_{cc} = the CDFM compressive strength of the concrete
ϕ_B = 0.90 (strength reduction factor)

If f_{sc} < yield stress (60 ksi or 40 ksi in accordance with DWG C-121, Ref. 8), a non-ductile failure mode has been considered with F_μ = 1.0 applied to a CDFM value based on tension in reinforcement.

b) When tension in reinforcement is not critical for the CDFM screening results while lack of splice length is probable, the effect of lacking splice length is conservatively estimated with a ratio of the actual length over the required length which can be taken as 1.4 unless the actual ratio based on the drawing information is found. F_μ = 1.0 similarly to above.

REACTOR BLDG WxSA,SSE/WxSA,RLE RATIO										
NODE NUMBER	ELEVAT'N ft.	MASS k. sec2 ft	WEIGHT W kips	SSE			RLE			RATIO
				SA,SSE	HORIZ'L FORCE kips	SUM OF FORCES kips	SA,RLE	HORIZ'L FORCE kips	SUM OF FORCES kips	WxSA,SSE / WxSA,RLE
8	164.50	40.41	1299.99	1.05	1364.99	1364.99	0.516	670.79	670.79	2.035
7	138.00	46.63	1500.09	0.86	1290.07	2655.06	0.332	498.03	1168.82	2.272
6	117.00	499.30	16062.48	0.45	7228.12	9883.18	0.222	3565.87	4734.69	2.087
5	91.25	352.70	11346.36	0.37	4198.15	14081.33	0.197	2235.23	6969.93	2.020
4	74.25	488.80	15724.70	0.33	5189.15	19270.48	0.182	2861.89	9831.82	1.960
3	51.00	585.30	18829.10	0.30	5648.73	24919.21	0.204	3841.14	13672.96	1.823
2	23.00	765.70	24632.57	0.23	5665.49	30584.70	0.195	4803.35	18476.31	1.655
1	-17.50	1665.00	53563.05	0.17	9105.72	39690.42	0.175	9373.53	27849.84	1.425

TURBINE BLDG WxSA,SSE/WxSA,RLE RATIO

NODE NUMB.	ELEVAT'N ft	MASS k. sec2 ft	WEIGHT W kips	SSE			RLE			RATIO
				S A,SSE	HORIZ'L FORCE kips	SUM OF FORCES kips	S A,RLE	HORIZ'L FORCE kips	SUM OF FORCES kips	WxSA,SSE / WxSA,RLE
7	105.5'	32.6	1048.74	1.71	1793.35	1793.35	0.59	618.76	618.76	2.90
6	90.0'	53.4	1717.88	1.22	2095.81	3889.16	0.45	773.05	1391.80	2.79
5	66.0'	70.4	2264.77	0.73	1653.28	5542.441	0.36	815.32	2207.12	2.51
4	51.0'	309.3	9950.18	0.59	5870.61	11413.05	0.34	3383.06	5590.18	2.04
3	37.0'	156.7	5041.04	0.43	2167.65	13580.69	0.28	1411.49	7001.67	1.94
2	23.0'	288.6	9284.26	0.33	3063.81	16644.5	0.27	2506.75	9508.42	1.75
1	6.0'	1005.3	32340.50	0.17	5497.89	22142.39	0.25	8085.13	17593.55	1.26
		TURBINE PEDESTAL + TURBINE BLDG STRUCTURES								
8	48.0'	481.8	15499.5	0.88	13639.6	13639.57	0.26	4029.87	4029.87	3.38
1	6.0'	-	-	-	22142.4	35781.97	-	17593.6	21623.47	1.65

Note: The value of "Horizontal Force" for node 1 in the shaded part of the table is taken as the "Sum of Forces" for the same node from the unshaded area of the table.

RADWASTE BLDG WxSA,SSE/WxSA,RLE RATIO

NODE NUMBER	ELEVAT'N ft.	MASS k. sec2 ft	WEIGHT W kips	SSE			RLE			RATIO
				SA,SSE	HORIZ'L FORCE kips	SUM OF FORCES kips	SA,RLE	HORIZ'L FORCE kips	SUM OF FORCES kips	WxSA,SSE / WxSA,RLE
6	81.00	38.3	1232.11	0.58	714.62	714.62	0.530	653.02	653.02	1.094
5	51.00	101.2	3255.60	0.33	1074.35	1788.97	0.280	911.57	1564.59	1.143
4	37.00	96.4	3101.19	0.26	806.31	2595.28	0.230	713.27	2277.86	1.139
3	23.00	203.6	6549.81	0.24	1571.95	4167.24	0.210	1375.46	3653.32	1.141
2	7.00	0	0.00	0.15	0.00	4167.24	0.210	0.00	3653.32	1.141
1	-1.00	266.9	8586.17	0.15	1287.93	5455.16	0.240	2060.68	5714.00	0.955

DIESEL GENERATOR BLDG WxSA,SSE/WxSA,RLE RATIO

NODE NUMBER	ELEVAT'N ft.	MASS k. sec2 ft	WEIGHT W kips	SSE			RLE			RATIO
				SA,SSE	HORIZ'L FORCE kips	SUM OF FORCES kips	SA,RLE	HORIZ'L FORCE kips	SUM OF FORCES kips	WxSA,SSE / WxSA,RLE
2	50.00	81.0	2608.00	0.17	443.36	443.36	0.320	834.56	834.56	0.531
1	23.00	30.5	982.00	0.15	147.30	590.66	0.330	324.06	1158.62	0.510

INTAKE STRUCTURE WxSA,SSE/WxSA,RLE RATIO

NODE NUMBER	ELEVAT'N ft.	SSE					RLE					RATIO
		MASS k. sec2 ft	WEIGHT W kips	SA,SSE	HORIZ'L FORCE kips	SUM OF FORCES kips	MASS k.sec2 ft	WEIGHT W kips	SA,RLE	HORIZ'L FORCE kips	SUM OF FORCES kips	WxSA,SSE / WxSA,RLE
3	38.00	29.9	961.9	0.30	288.56	288.56	22.6	727.7	0.187	136.08	136.08	2.120
2	21.50	502.1	16152.6	0.28	4522.72	4811.28	400.3	12889.7	0.172	2217.02	2353.11	2.045
1	-24.00	528.8	17011.5	0.15	2551.72	7363.01	528.8	17027.4	0.181	3081.95	5435.06	1.355

S & A
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JOB NO. 91C2672
SUBJECT PILGRIM IPEEE
BUILDING STRUCTURES FRAGILITY ANALYSIS

Revision 1
By K.C. 8/5/93
Chk S.C. 8-5-93

SUMMARY of RESULTS
CDFM and Median Fragility Values

Abbreviations:
- R.B. for Reactor Building
- T.B. for Turbine Building
- R.W. for Radwaste Building
- I.S. for Intake Structure
- D.G. for Diesel Generator Building

Item No./ BLDG	Structural Component	Elevation	Governing Stress Component	CDFM PGA (g)	Comment	Median Fragility (g)
1. / R.B.	Concrete wall, North	above -(17'-6")	concrete shear	2.36	Note 1	5.07
		above 23'-0"	concrete shear	2.78		5.98
2. / R.B.	Precast panels as a part of shear walls	above 117'-0"	concrete compression	2.85		6.13
			concrete shear	1.38		2.97
			reinforcing tension	1.54		3.31
	Precast panels in out-of-plan bending	above 117'-0"	concrete compression	2.18		4.69
			concrete shear	5.37		11.55
			reinforcing tension	1.72		3.70
3. / R.B.	Truck access floor	23'-0"	concrete compression	6.29	composite action	13.52
			beam bending	2.22		4.77
			concrete compression	4.82	slab between beams	10.36
			concrete shear	1.65		3.55
			reinforcing tension	4.94		10.62
4. / R.B	Internal diagonal concrete wall	between 23'-0" and 51'-0"	concrete compression	2.68		5.76
			concrete shear	1.66		3.57
			reinforcing tension	11.23		24.14

cont. next page

SUMMARY of RESULTS
CDFM and Median Fragility Values (cont'd)

Item No./ BLDG	Structural Component	Elevation	Governing Stress Component	CDFM PGA (g)	Comment	Median Fragility (g)
5. / R.B.	Drywell shielding concrete	about 30 ft about 40 ft about 115 ft	concrete compression reinforcing tension concrete shear	1.39 0.95 2.53	Note 2	2.99 2.04 5.44
6. / R.B.	Internal steel column N-15.5	23'-0"	axial compression	2.81		6.04
7. / R.B.	Steel roof	above 156'-6"	axial or bending stress	0.81	Note 3	1.74
8. / R.B.	Foundation	-(25'-6")	soil bearing stress	0.82	Note 4	1.76
9. / T.B.	Concrete wall @ Line 20	above 6'-0"	concrete shear	1.81		3.89
10. / T.B.	Concrete wall @ Line 3	above 23'-0"	concrete shear	2.53		5.44
11. / R.W.	Concrete wall @ Line 16	below 23'-0" below 37'-0"	concrete shear concrete shear	1.46 1.15		3.14 2.47
12. / R.W.	Concrete wall @ Line E	below 23'-0" below 37'-0"	concrete shear concrete shear	0.9 0.9		1.94 1.92
13. / D.G.	Concrete wall @ Line R.3	above 23'-0"	concrete shear	1.21		2.60
14. / D.G.	Concrete wall (pilons) @ Lines U & T	above 23'-0"	concrete shear reinforcing tension	1 0.4	 Note 5	2.15 0.86

cont. next page

SUMMARY of RESULTS

CDFM and Median Fragility Values (cont'd)

Item No./ BLDG	Structural Component	Elevation	Governing Stress Component	CDFM PGA (g)	Comment	Median Fragility (g)
15. / D.G.	Concrete wall @ Line Q	above23'-0"	concrete shear	0.39		0.84
16. / D.G.	Concrete slab between Lines R.3 & Q	34'-6"	concrete shear reinforcing tension	0.42 0.36		0.90 0.77
17. / I.S.	North wall of Concrete Core	above 21'-6"	concrete shear	5.17		11.12
18. / I.S.	South wall of Concrete Core	below 21'-6"	concrete shear	4.93		10.60
19. / I.S.	Partial N-S shear walls	below 21'-6"	concrete shear	2.11		4.54

NOTE 1. FSAR (Ref. 2) discusses stresses in concrete walls being in excess of the yield stress. However FSAR does not indicate where these stresses occur. It might be connected to a bending mode of a wall as well as to stress in steel column above El. 117 ft where columns along with precast concrete panels are considered to form a substitute shear wall. Since these critical cases cannot be addressed the effect of uncertainty can be taken into consideration.

NOTE 2. These values of CDFM are considerably conservative as they were calculated on the basis of the total stresses for a (D+SSE+THER) load combination which were figured out by approximation (detailed output of a finite element analysis of the drywell structure is not available).

NOTE 3. This value is derived on the basis of a conservative assumption that the effect of (D+SSE) is the same as that of (D+Tornado) though it is known from FSAR that the effect of (D+Tornado) is greater.

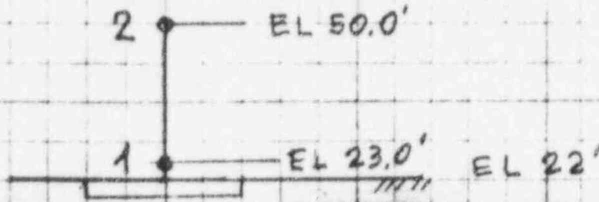
NOTE 4. Value of .82 has been obtained on the basis of 15 ksf allowable soil capacity which is the value used in FSAR for an OBE load combination. Hence this value is conservative.

NOTE 5. See comments on page 66 of this calculation.

FRAGILITY ANALYSIS

DIESEL GENERATOR BUILDING

COMPUTER MODEL



REF. 1 PG C-59
REF 6 PG 10

ELEVATION FT	NODE	WEIGHT KIPS	$S_{A,SSE}$	$S_{A,RLE}$
50'	2	2608	0.17	0.32
23'	1	2234	0.15	0.33

REF 1 FIG A-67 A-68

REDUCE THE WEIGHT OF NODE #1 BY SUBTRACTING THE WEIGHT OF THE SLAB @ EL 23.0', THE WEIGHT OF EQUIPMENT SITTING ON EL 23.0' SLAB AND CONCRETE COLUMNS: REF 6 PG 4, 6

$880^K + 351^K + 21^K = 1252^K$

REDUCED WEIGHT @ NODE 1:

$2234 - 1252 = \underline{982^K} \rightarrow$ USE

CALCULATE $\sigma_{C,SSE}$

	ELEV	W KIPS	$S_{A,SSE}$	HORIZ FORCE	SUM
2	50.0'	2608	0.17	443.0	443.0^K
1	23.0'	982	0.15	147.0	590^K

$\Sigma W = 3590^K$

EVALUATE THE EFFECT OF A TORSIONAL MOMENT USING DATA FROM REF 6 REF 6 APP A, PG 2

APP A, PG 4

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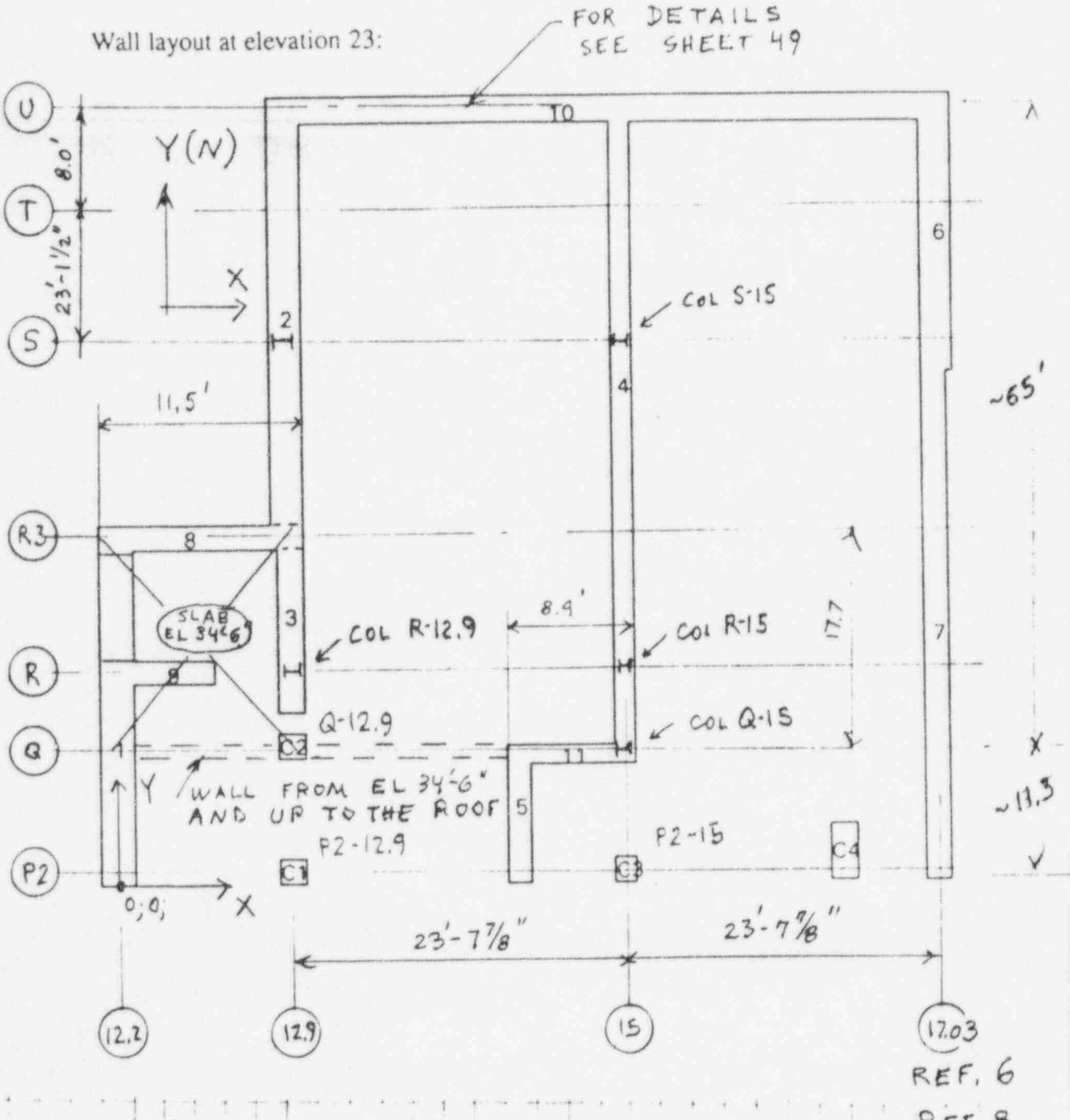
SUBJECT PILGRIM IPEEE JOB No. 91C2672 SHEET 46 OF 77

BUILDING STRUCTURES
FRAGILITY ANALYSIS.
DIESEL GENERATOR
BLDG

REVISIONS
1 K.C. 8/5/93
S.C. 8-5-93

Wall layout at elevation 23:



REF. 6

REF 8

DWG. C-198
C-199
C-200

* EVALUATE THE EFFECT OF A TORSIONAL MOMENT (CONT'D)

TORSION DUE TO N-S EARTHQUAKE:

$e_x = 30.75' - 29.36' = 1.4'$ (MAX)

TORSIONAL MOMENT M_z'

$M_z' = 590^k \times 1.4' = 826^{k\text{-}f}$

CONSERVATIVELY CONSIDER TWO WALLS ONLY: #7 & #2.

THE DISTANCE BTWN THEM:

$d = 48.73 - \left(\frac{1.92}{2} + \frac{1.5}{2}\right) = 47'$, PG 46

ADDITIONAL SHEAR FORCE ΔV_y DUE TO TORSION

$\Delta V_y = \frac{M_z'}{d} = \frac{826^{k\text{-}f}}{47'} = 17.6^k$;

ADDITIONAL SHEAR Δv_y (WALL #7):

$\Delta V_y = 17600 / (46.5' \times 1.5' \times 144) = \underline{1.8 \text{ PSI}}$

∴ DISREGARD TORSIONAL EFFECT FOR SMALLNESS.

* DEFINE SEISMIC LOAD DISTRIBUTION BTWN SHEAR WALLS IN RELATION WITH THEIR RIGIDITY (ESTIMATE)

$$R_{x,y} = \frac{E}{H^3/3I_{y,x} + 2.2H/0.82A}$$

$H = h_w$ = HIGHT OF THE WALL, TAKEN AS 27'

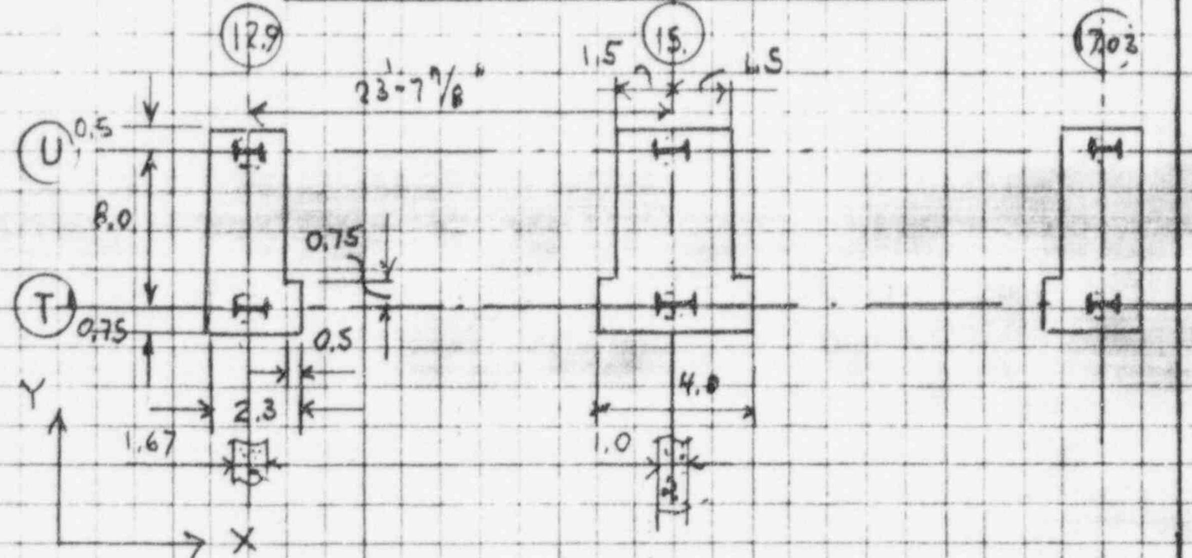
USE WALL PROPERTIES AS SHOWN ON NEXT PG

WALL PROPERTIES *)

WALL	WIDTH ft	THICK'S ft	A ft^2	I_x ft^4	I_y ft^4	
1	31.97	1.17	36.5	2953.	4.2	REF 6
2	47.46	1.17	55.5	10,423.	6.3	
3	12.33	1.5	18.5	234.	3.5	
4	65.13	1.0	65.1	23,023.	5.4	
5	15.0	1.0	15.0	281.	1.3	
6	31.13	1.17	36.4	2941.	4.2	
7	46.5	1.5	69.8	12,568.	13.1	
8	11.5	1.0	11.5	1.0	126.7	REF 8
9	BLOCK WALL - NOT INCLUDED					
10	SEE PG 49.		64.0	150.	40.3	
11	8.4	1.0	8.4	0.8	49.4	

*) WALL SIZES OBTAINED FROM REF 6 ARE USED FOR STIFFNESS COMPARISON ONLY AS PROVIDED IN TABLES ON PGS 51 AND 52 — PGS 51, 52

WALL 10 PROPERTIES



REF 8
C-198
C-199

$$A = (1.8 \times 9.25 + 1.5 \times 0.5)2 + (3 \times 9.25 + 1 \times 1.5)$$

REF 8
C 198
C 199

$$= 34.8 + 29.22 = \underline{64.0\ ft^2}$$

$$\Sigma I_y = 2\left(\frac{1.8^3 \times 7.75}{12} + \frac{2.3^3 \times 1.5}{12}\right) + \left(\frac{3^3 \times 7.75}{12} + \frac{4^3 \times 1.5}{12}\right)$$

$$= 10.57 + 29.7 = 40.3\ ft^4$$

PILON @ L'15': $\frac{29.7}{40.3} = 0.74$; @ L 12.9; 17.03: $\frac{10.57}{40.3} = 0.26$

I_x (DISREGARDING T-PARTS)

$$\Sigma I_x = 9.25^3/12\ (1.8 \times 2 + 3.0) = 435.3\ ft^4$$

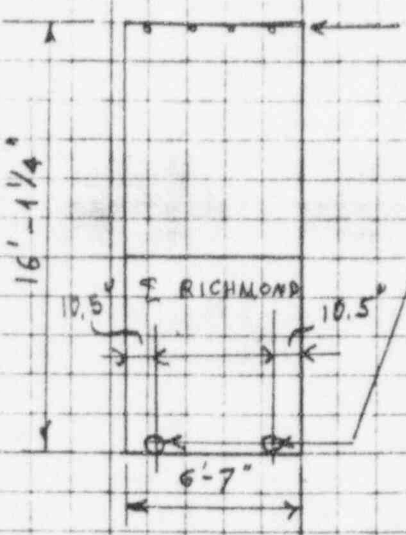
SUBTRACT PART OF WALLS 2, 4, 6 BTWN T & U

$$\Delta I_x = 9.25^3/12\ (1.67 \times 2 + 1.0) = 285.6\ ft^4$$

$$I_{x\,10} = 435.3 - 285.6 = \underline{150\ ft^4}$$

EVALUATE PRECAST PANEL STIFFNESS

REF 8
A-25
C-198



EACH CONNECTION @ THE BOTTOM CONSISTS OF 1-¾" RICHMOND + 1-¾" BOLT INSTALLED THRU A 1" PIPE SLEEVE (A-25, C-198, DET 3)

PER DWG C-198 THE EDGE DISTANCE FOR THE RICHMOND BOLTS IS:

$$(7.75' - 6.0')/2 = 10\tfrac{1}{2}"$$

USE THIS DIMENSION TO DEFINE THE DISTANCE BTWN RICHMOND BOLTS SHOWN ON DWG A25: (6'-7") - 21" = 4'-10" = 58"

USE 60".

DISREGARD THE STIFFNESS OF THE ¾" BOLTS IN THE 1" PIPE SLEEVES (I.D. ≈ 1") - TOO LOOSE FOR ANY STIFFNESS.
DISREGARD ANY FLEXIBILITY OF THE TOP NELSON STUDS CONNECTION. SAY THE FORCE FROM THE STRUCTURES ABOVE THE PANEL (MOSTLY FROM THE ROOF SLAB) IS APPLIED THRU A STIFF CONNECTION AND ONLY THE PANEL BOTTOM CONNECTION HAS FLEXIBILITY.

ASSUME THE RICHMOND CONNECTION FLEXIBILITY IS DUE TO L4×4×¾ ANGLE ONLY AND ONLY WHEN THE FORCE IS APPLIED UPWARDS. WHEN FORCE IS APPLIED DOWNWARDS THE CONNECTION IS STIFF.

A STIFFNESS CALCULATED THIS WAY WILL PROVIDE A CONSERVATIVE*) (GREATER THAN ACTUAL) STIFFNESS VALUE.
*) SEE NEXT PAGE FOR DEFINISION

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SUBJECT PILGRIM IPEEE JOB No. 91C2672 SHEET 50A OF 77

BUILDING STRUCTURES
FRAGILITY ANALYSIS

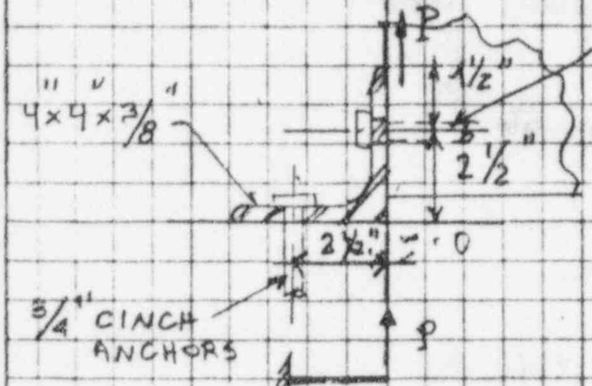
DIESEL GENERATOR BLDG

REVISIONS: 1 K.C. 8/4/93; S.C. 8-5-93

EVALUATE PRECAST PANEL STIFFNESS

(CONT'D)

AT THE BOTTOM EACH OF THE 6 PANELS IS ATTACHED TO THE CONCRETE FOUND'N BY 2 - 3/4" RICHMOND BOLTS — REF 8 A-25 C-198



$$n = \frac{E_s}{E_c} = 9.35$$

BENDING IN L4x4 HORIZONTAL LEG

SAY WIDTH $b = (2.5 + 2.5)2 + 1 = 11''$

$$b_{AV} = 11''/2 = 5.5'' \qquad t = 0.38''$$

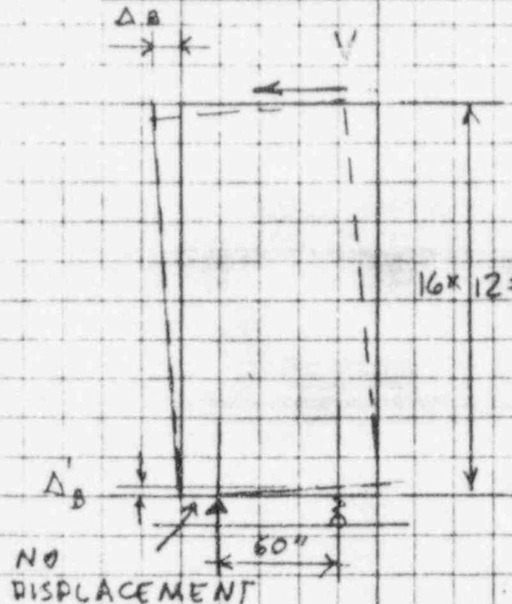
$$I = \frac{0.38^3 \times 5.5}{12} = 0.025\ in^4 \qquad l = 2\tfrac{1}{2}''$$

$$\Delta'_B = \frac{Pl^3}{3E_sI} = \frac{l^3}{3InE_c} = \frac{2.5^3}{9.35 \times 3 \times 0.025 E_c} = 22.3 \frac{1/in}{E_c}$$

$$E_c\Delta'_B = 22.3\ 1/in \times 12\ in/ft = \underline{267.4}\ 1/ft$$

*) THE OBJECTIVE OF THIS ANALYSIS IS TO SHOW WHAT CAN BE THE <u>MOST</u> THAT THE PRECAST PANELS CAN CONTRIBUTE TOWARD WALL'10' STIFFNESS. IF THIS CONTRIBUTION WILL BE STILL SMALL (EVEN WITH EXAGGERATED PANEL STIFFNESS VALUE) THE PANELS CONTRIBUTION TO THE TOTAL BUILDING STIFFNESS CAN BE IGNORED. THIS IS CONSERVATIVE.

EVALUATE PRECAST PANEL STIFFNESS (CONT'D)



DISPLACEMENT @ THE TOP:

$$\Delta_B = \Delta_B' \frac{192}{60} = 3.2\,\Delta_B'$$

$$E_C \Delta_B = 267.4 \times 3.2 =$$
$$= 855.7 \text{ }^{1}/_{ft}$$

COMPARE PANEL STIFFNESS TO TOTAL WALL '10' STIFFNESS

PILON @ L. 15 $= \dfrac{13^3}{3(29.7)} = 24.66 \text{ }^{1}/_{ft}$

PILONS @ L 12.96 17.03 $= \dfrac{13^3 \times 2}{3 \times 10.57} = 138.57 \text{ }^{1}/_{ft}$

TOTAL STIFFNESS OF WALL 10 INCLUDING 3 PILONS AND 6 PANELS

$$\frac{1}{E_c} R_y = \frac{1}{24.66} + 2 \times \frac{1}{138.57} + 6 \times \frac{1}{855.7} =$$

$$= 0.041 + 0.014 + 0.007 = 0.062$$

PANEL CONTRIBUTION: $\dfrac{0.007}{0.062} = 0.11$;

∴ ACTUAL CONTRIBUTION OF THE 6 PRECAST PANELS IS EVEN SMALLER DUE TO CONSERVATISM DESCRIBED ABOVE.

∴ DISREGARD PRECAST PANELS STIFFNESS

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SUBJECT: PILGRIM IPEEE JOB NO. 91C2672 SHEET 51 OF 77

BUILDING STRUCTURES
FRAGILITY ANALYSIS
DIESEL GENERATOR
BUILDING

REVISIONS: 1 — K.C 8/5/93, S.C. 8-5-93

WALL STIFFNESS COMPARISON (X-DIR'N)
(SEE FORMULA ON PG 47)

MEMBER	AREA A ft^2	Hight H ft	$\frac{H^3}{3}$ ft^3	$2.7\frac{H}{A}$ 1/ft	I_y ft^4	$\frac{H^3}{3I_y}$ 1/ft	$\frac{R_x}{E}$ $\times 10^{-4}$	$\frac{R_x}{\Sigma R_x}$	SHEAR V_x KIPS
WALLS									
1	36.5	27	6561	2.000	4.2	1562.1	6.4		
2	55.5	27	6561	1.314	6.3	1041.4	9.6		
3	18.5	27	6561	3.941	3.5	1874.6	5.3		
4	65.1	27	6561	1.120	5.4	1215.0	8.2		
5	15.0	27	6561	4.860	1.3	5047.0	2.0		
6	36.4	27	6561	2.030	4.2	1562.1	6.4		
7	69.8	27	6561	1.044	13.1	500.8	19.9	0.05	
8	11.5	27	6561	6.339	126.7	51.8	172.0	0.47	
9					BLOCK WALL - NOT INCLUDED				
10	64.0	27	6561	1.14	40.3	162.8	61.0	0.17	
11	8.4	27	6561	8.679	49.4	132.8	70.6	0.19	
COLUMNS									
C1 P.2-12.9	2.0	27	6561	36.45	0.7	9373.0	1.1		
C2 Q-12.9	3.0	27	6561	24.30	1.0	6561.0	1.5	0.005	
C3 P.2-15	1.0	27	6561	72.90	0.1	65,610.0	0.2		
C4 P.2-16.8	6.3	27	6561	11.571	1.2	5468.0	1.8	0.006	
COL/WALL									
S-12.9	0.5	27	6561	145.8	0.3	21,870.	0.5		
Q-15	0.5	27	6561	145.8	0.3	21,870.	0.5		
R-15	0.5	27	6561	145.8	0.3	21,870.	0.5		
S-15	0.5	27	6561	145.8	0.3	21,870.	0.5		

$\Sigma R_x = 368.0 \times 10^{-4}$

WALL STIFFNESS COMPARISON (Y-DIR)

(SEE FORMULA ON PG 47)

MEMBER	AREA A ft^2	Hight H ft	$\frac{H^3}{3}$ ft^3	$2.7\frac{H}{A}$ 1/ft	I_x ft^4	$\frac{H^3}{3I_x}$ 1/ft	$\frac{R_y}{E}$ $\times 10^3$	$\frac{R_y}{\Sigma R_y}$	SHEAR V_y KIPS
WALLS									
1	36.5	27	6561	2.000	2953.0	2.2	238.0	0.10	59.0
2	55.5	27	6561	1.314	10,423.0	0.6	527.0	0.21	124.0
3	18.5	27	6561	3.941	234.0	28.0	31.3	0.01	6.0
4	65.1	27	6561	1.120	23,023.0	0.3	704.0	0.29	171.1
5	15.0	27	6561	4.860	281.0	23.3	35.5	0.02	12.0
6	36.4	27	6561	2.000	2941.0	2.2	238.0	0.10	59.0
7	69.8	27	6561	1.044	12,568.0	0.5	649.0	0.27	159.3
8	11.5	27	6561	6.339	1.0	6561.0	0.1		
9	BLOCK	WALL.	NOT	INCLUDED					
10	84.0	27	656	1.14	130.0	437	223.0		
11	8.4	27	6561	8.68	0.8	8201.	0.1		
COLUMNS									
C1 P.2-12.9	2.0	27	6561	36.45	0.2	32,805.	—		
C2 Q-12.9	3.0	27	6561	24.30	0.6	10,935.	0.1		
C3 P.2-15	1.0	27	6561	72.90	0.1	65,610.	—		
C4 P.2-16.8	6.3	27	6561	11.571	9.5	6906	1.4		
COL/WALL									
S-12.9	0.5	27	6561	145.8	0.04	164,000	—		
Q-15	0.5	27	6561	145.8	0.04	↓	—		
R-15	0.5	27	6561	145.8	0.04	↓	—		
S-15	0.5	27	6561	145.8	0.04	↓	—		

$\Sigma R_y = 2642.7 \times 10^{-3}$ $\Sigma V = 590.4^K$

EVALUATE E-W LOAD PATH

SEISMIC LOAD GENERATED BY THE ROOF AND ADJACENT PARTS OF WALLS IS TAKEN BY WALLS ON LINES 'U' AND 'Q'
SINCE THOSE TWO ARE THE ONLY ONE BTWN EL 50' & 34',
THE LOAD WILL BE DISTRIBUTED BTWN THEM IN A PROPORTION CALCULATED LATER (AFTER THE SUM OF WALLS '8' & '11' STIFFNESS IS CALCULATED)

WALLS '8' & '11' STIFFNESSES

DIVIDE THE SHEAR FORCE ON LINE 'Q' BTWN WALLS '8' & '11' (DISREGARD BLOCK WALL '9')
SAY WALL @ EL 34 & UP ON LINE 'Q' (BTWN 12.2 & 15) IS STIFF ENOUGH TO DISTRIBUTE THE FORCE ALONG 'Q'

COMPARE WALL 11 R_y TO (WALL 8 + SLAB @ EL 34'-6") R_y

SLAB $\frac{H^3}{3 I_y} = \frac{17.7^3 \times 12}{3 \times 1.00 \times 11.5^3} = 14.6$

WALL 8 $= \frac{11.5^3 \times 12}{3 \times 1.0 \times 11.5^3} = 4.0$

$\left.\begin{matrix} 14.6 \\ 4.0 \end{matrix}\right\} 18.6$

WALL '11' $= \frac{11.5^3 \times 12}{3 \times 1.0 \times 8.4^3} = 10.2$

STIFFNESSES

COMBINED SLAB + WALL 8 + WALL 11

SLAB + WALL 8 $= 1/18.6 = 0.054$
WALL 11 $= 1/10.2 = 0.098$
$\Sigma = \underline{0.152}$

SINCE WALL 8 AND SLAB ARE IN SERIES.

WALL 8 CONTRIBUTION $0.054/0.152 = \underline{0.36}$
WALL 11 $0.098/0.152 = \underline{0.64}$

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SUBJECT PILGRIM IPEEE JOB No. 91C2672 SHEET 54 OF 77
BUILDING STRUCTURES
FRAGILITY ANALYSIS
DIESEL GENERATOR
BLDG

REVISIONS
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EVALUATE E-W LOAD PATH (CONT'D).

CALCULATE WALL '10' R_y : $H = 36' - 23' = 13'$

WALL '10' $\frac{H^3}{3 I_y} = \frac{13^3}{3 \times 40.3} = \underline{18.2}$ 1/ft PG 48

$H = 13'$ IS TAKEN CONSIDERING THAT
I_y EXTENDS FROM EL 23' UP TO EL 36'
WHERE AN E-W BEAM STARTS.

WALL '10' STIFFNESS $1/18.2 = 0.055$;

PORTION OF ENTIRE F_y HORIZONTAL
FORCE CARRIED BY:

WALL 8 : $\frac{0.152}{0.152 + 0.055} \times 0.36 = 0.26$

WALL 10 : $\frac{0.055}{0.152 + 0.055} = 0.27$

WALL 11 : $\frac{0.152}{0.152 + 0.055} \times 0.64 = 0.47$

SAY 20% OF THE TOTAL HORIZONTAL
FORCE OF 590^k IS TAKEN BY
ALL OTHER WALLS SPANNING IN
N-S (Y) DIRECTION (SEE TABLE ON PG 51) PG 51

THUS $F_y = 590 \times 0.8 = \underline{470}^k$

FORCES ARE DISTRIBUTED AS FOLLOWS:

SLAB : $F = 470 \times 0.26 = 122^k$
WALL 8 : $F = 470 \times 0.26 = 122^k$
WALL 10 : $F = 470 \times 0.27 = 127^k$
WALL 11 : $F = 470 \times 0.47 = 220^k$

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SUBJECT PILGRIM IPEEE JOB No. 91C2672 SHEET 55 OF 77

BUILDING STRUCTURES FRAGILITY ANALYSIS DIESEL GENERATOR BUILDING

REVISIONS: 1 K.C 8/5/93 S.C. 8-5-93

SHEAR STRESSES IN WALLS:

WALL #	V_x KIPS (PG 54)	AREA (PG 48) ft²	V_x : σ_{OSSE} PSI
8	122.0	11.5	74.
10, ℄ 15	27×0.74	34.8	19.
10, ℄ 12.9	127×0.13	14.6	8.
11	220	8.4	PG 63
12" THK SLAB @ EL 34'-6"	122	11.5	74

WALL #	AREA (PG 48) ft²	V_y KIPS	V_y psi
1	36.5	59.0	11.2
2	55.5	124.0	15.5
3	18.5	6.0	2.2
4	65.1	171.1	18.3
5	15.0	12.0	5.6
6	36.4	59.0	11.2
7	69.8	159.3	15.8

PG 54

THESE MEMBERS GOVERN (WALLS IN X-DIR'N)

SEE PG 52

P52

CALCULATE σ_u

WALL 8, CONCRETE WALL ON LINE R.3 — REF. 8 C.198 C.199

$$h_w = 11.5'; \quad t_w = 1.0' \quad L_w = 11.5'; \quad h_w/L_w = 1.0$$

REF 3 APP'L'

$$V_u = 6.8\sqrt{f'_c} - 2.8\sqrt{f'_c}\left[h_w/L_w - 0.5\right] + \frac{N_A}{4 l_w t_n} + \rho_{se} f_y$$

* $6.8\sqrt{f'_c} = 6.8\sqrt{3000} = \underline{372}$ psi
* $2.8[\ ] = 2.8\sqrt{3000}\,[1 - 0.5] = \underline{77}$ psi
* $N_A/(4 l_w t_n) = 0$
* CALCULATE ρ_{SE}

STEVENSON & ASSOCIATES a structural-mechanical consulting engineering firm	SUBJECT PILGRIM IPEEE JOB No. 91C2672 SHEET 56 OF 77
	BUILDING STRUCTURES FRAGILITY ANALYSIS DIESEL GENERATOR BUILDING
	REVISIONS: 1 K.C. 8/5/93 S.C. 8-5-93

CALCULATE ρ_{SE}

$A = -h_w/L_w + 1.5 = -1.0 + 1.5 = 0.5$

$B = h_w/L_w - 0.5 = 1.0 - 0.5 = 0.5$

$\rho_v = \dfrac{2 \times 0.79}{144} = 0.011$ (#8 @ 12)

$\rho_u = \dfrac{2 \times 0.31}{144} = 0.0043$ (#5 @ 12)

$f_y \times \rho_{SE} = A \rho_v f_v + B \rho_u f_y$;

$f_y \times \rho_{SE} = 0.5 (0.011 \times 60{,}000 + 0.0043 \times 40\,000)$

$= \underline{416 \text{ psi}}$ USE 70% OF THIS VALUE

CONSIDER POSSIBLE REDUCED SPLICE LEGTH, SINCE THIS IS A SHORT WALL.

$V_u = 372 - 77 + 416 \times 0.7 = \underline{586 \text{ psi}}$

COMPARE TO $7\sqrt{3000} = 383 \text{ psi} < 586$

∴ SHEAR FRICTION CAPACITY IS ADEQUATE

$\sigma_u = 586 \times 0.6 = 352 \text{ psi}$ REF 3 L-6

$CDFM = 1.25 \times 352 \times \dfrac{1}{1.96} \times \dfrac{0.4}{74} = \underline{1.21g}$

→ FOLLOWING EQ $\left[\dfrac{\Sigma(W_A S_{A,SSE})}{\Sigma(W_A S_{A,RLE})}\right]$ THE VALUE IN [ ] WAS CALCULATED AS:

$$\frac{2608 \times 0.32 + 982 \times 0.33}{2608 \times 0.17 + 982 \times 0.15} = 1.96$$

FOR INPUT SEE PG 45

SLAB @ EL 34'-6" BTWN R.3 & Q.

$h_w = 17.7'$ $L_w = 11.5'$ $h_w / L_w = \frac{17.5}{11.5} = 1.52$

(1) SHEAR CAPACITY — REF 8 C-199

$t_w = 1.0$

$\times\ 6.8\sqrt{3000} = \underline{372\ psi}$

$\times\ 2.8\sqrt{3000}\left[1.52 - 0.5\right] = \underline{156\ psi}$

$\times\ \rho_{SE}$

$A = 0; \quad B = 1.0$

$\rho_v = \rho_h = \frac{2 \times 0.31}{12 \times 12} = 0.0043;$ (#5 @ 12") — REF 8 C-199

$f_y \rho_{SE} = 0.0043 \times 40000 = \underline{172\ psi}$

SEE PG 56 — PG 56

$v_u = 372 - 156 + 172 \times 0.7 = \underline{336\ psi}$

$v_u > 7\sqrt{3000} = 383\ psi > 336\ psi$ — REF 3

∴ SLIDING SHEAR GOVERNS $F_\mu = 1.0$ — PG L-5

SLAB REINFORCEMENT : #5 @ 12 T & B

$A_{vf} = 0.31 \times 12 \times 2 = 7.44\ in^2$

$\mu = 1.0$ (DWG C-121, NOTE B) — C-121

$f_y = 40000\ psi$ — REF 8

$V_n = (1.0 \times 40 \times 7.44) = 298^k \leq 0.2 f_c'$

$0.2 f_c' \times 12 \times 11.5' \times 12 = 994^k \gg 298^k$ - USE

$0.2 f_c' : 0.2(3000) = 600\ psi < 800\ psi$

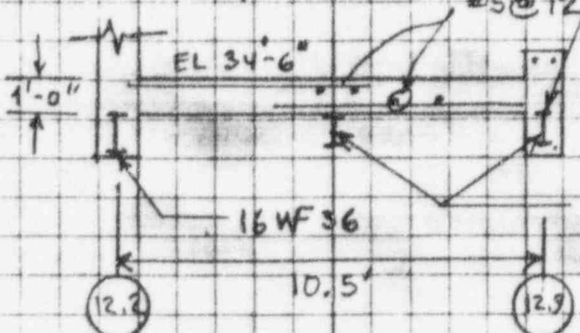
$CDFM = 1.0 \times \frac{0.85(298)}{122} \times \frac{1}{1.96} \times 0.4 = \underline{0.42g}$ — PG 55

SLAB @ EL 34'-6" BTWN R.3 & Q (CONT'D)

BENDING CAPACITY:

REF DWG C-200

• PER DWG C-199 THIS 10WF21 BEAM IS EMBEDDED IN CONCRETE — REF 8 C-199 C-200



• BOLT CONNECTIONS - PER AISC FRAME CONNECTIONS.

10WF21 WILL HAVE:

6 - 7/8" A325 BOLTS

6 - 7/8" BOLT CAPACITY

REDUCTION FACTOR FOR SHEAR IN BOLT

$T = 6 \times 0.55 \times 44\ ksi = 145^k$;

CONSERVATIVE

BEAM CAPACITY $A = 6.2\ in^2$, $T = 24_{ksi} \times 6.2^{\#} = 149^k$

USE SMALLER $T = 145^k$ - THE BOLT CONNECTION TO COL ON LINE 12.9.

DISREGARD ANY OTHER TENSION REINFORCEMENT AND CONSIDER THIS 10WF BEAM ONLY.

IN A 12" THK SLAB CONCRETE "d"

$$d = \frac{145\,000}{12 \times 0.85 \times 0.9 \times 3000} = 5.2";$$

SAY ARM IS 10.5' AND

MOMENT CAPACITY $M = 145 \times 10.5 = \underline{1522^{k\text{-}1}}$

THIS IS A VERY CONSERVATIVE ESTIMATE CONSIDERING ALLOWABLE STRESS APPROACH FOR BENDING CAPACITY.

SLAB @ EL 34'-6" BTWN R, 3 & Q (CONT'D)

BENDING MOMENT IN SLAB

THE SLAB IS BOXED BY SHEAR WALLS, IT HAS A CONTINUATION BEYOND LINE Q (MOMENT CONNECTION), PLUS THE SHEAR WALLS SURRONDING THE SLAB HAVE A VERY STIFF DISK CONNECTION AT THE TOP - THE ROOF SLAB.

THUS THE BENDING MOMENT CAN BE CALCULATED AS

$M = 0.5 L P = 0.5 (17.5') 122^{k} = 1068^{k-1}$ PG 55

SINCE THE STEEL BEAMS CONSIDERED AS THE IN-PLANE SLAB REINFORCEMENT ARE CONTINEOUS @ ALL 4 EDGES THE SLAB HAS A BENDING CAPACITY @ THE SUPPORTS AS WELL.

$CDFM = 1.25 \times \frac{1522}{1068} \times \frac{1}{1.96} \times 0.4 = \underline{\underline{0.36g}}$ PG 56

WALL 10, PILONS ON LINES 'U' & 'T'

A. PILON @ LINE 15

(1) SHEAR CAPACITY

* $v_c = 2\sqrt{3000} = 109.5$ psi — REF 4

* SHEAR REINFORCEMENT

THE WIDTH OF THE PILON IS TOO SMALL TO COUNT ON VERTICAL REBARS.

THE HORIZONTAL REINFORCEMENT DOES NOT MEET THE ACI REQUIREMENTS (SEE PG 66 OF THIS CALC. — PG 66

$\therefore v_s = 0$

$\sigma_u = 0.85(v_c + v_s) = 0.85(109.5 + 0) = 93$ psi

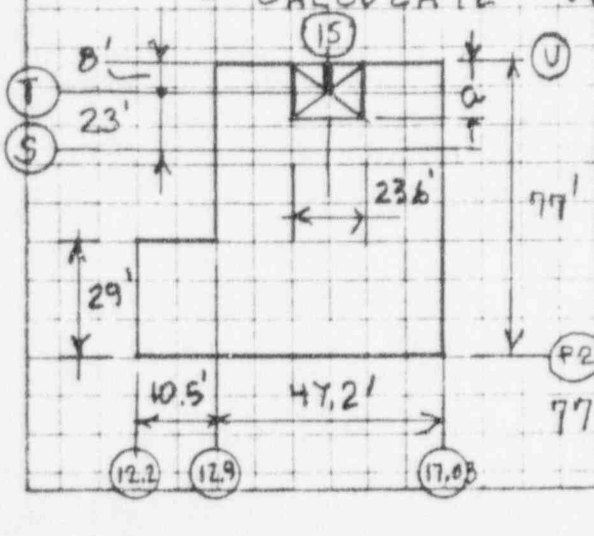
$\sigma_{c,SSE} = 19$ psi — PG 55

$CDFM = 1.0 \times 93 \times \frac{1}{1.96} \times \frac{0.4}{19} = \underline{1.00\,g}$

NOTE: DUE TO LACK OF VERTICAL REBAR SPLICE LENGTH NO DUCTILITY ($F_\mu = 1.0$) AND NO LOAD REDISTRIBUTION IS CONSIDERED

(2) BENDING CAPACITY
(FLEXURE WITH AXIAL LOAD)

* CALCULATE VERTICAL LOAD FOR THE PILON @ LINE 15



$a = 8' + \frac{23'}{2} = 19.5'$

AREA: $23.6 \times 19.5 = \underline{460}\,\square'$

TOTAL BLDG AREA

$77 \times 47.2 + 10.5 \times 29 = \underline{3940}\,\square'$

FRACTION OF THE TOTAL VERTICAL LOAD THAT GOES TO COL ON ℄ 15 & ℄ U & T

$460/3940 = 0.12$

TOTAL LOAD - 3590^K PG 45

VERTICAL LOAD @ (U+T) 15

$P = 3590 \times 0.12 = 431^K$

ADD SSE SEISMIC LOAD $0.17 \times 2/3 = 0.11$

$P_{max} = 431\,(1 + 0.11) = 483^K$

$P_{min} = 431\,(1 - 0.11) = 379^K$.

* CALCULATE BENDING MOMENT

TOTAL E-W FORCE @ COL LINE 'U-T'

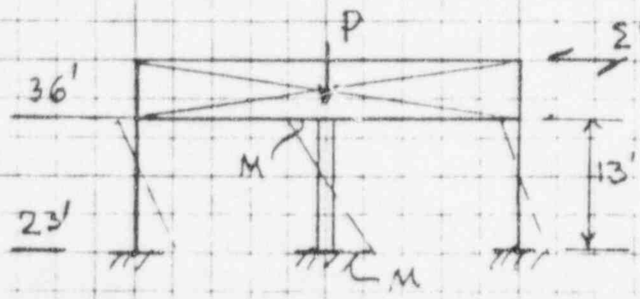
$V = 127^K$ PG 55

THE PILON ON LINE '15' IS THE MOST RIGID ONE, SO IT WILL SEE THE GREATEST HORIZONTAL (SEISMIC) FORCE.

PG.55

$V_{15} = 127 \times 0.74 = 94^K$

$M_{15} = V_{15} \times 13'/2 = 94^K \times 6.5' = 611^{K\text{-}'}$



FOLLOWING REF 11 RECOMMENDATIONS CALCULATE PILON MOMENT CAPACITY CONSIDERING A 20" SPLICE LENGTH IN THE # 8 REBARS (LINE 15.)

REF 11

REF 8
C-198

$\phi_B = 0.9; \quad f_{cc} = 3000\ psi; \quad A_b = 0.785\ in^2$

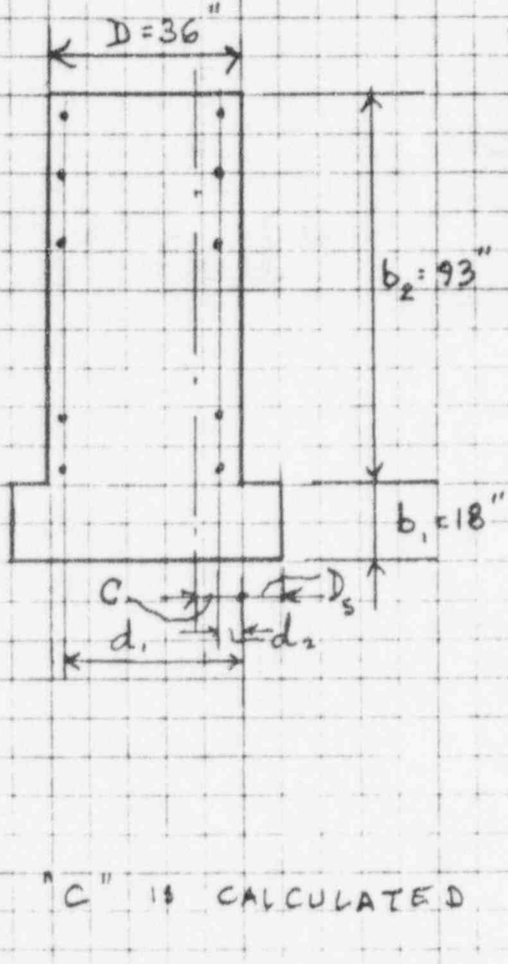
$l_d = 20''; \; C_{min} = C_b = 1 + 5/8 = 1.63''; \quad d_b = 1''$

$C_{max} = C_s = (12'' - 1'')/2 = 5.5''$

$$f_{sc} = \frac{0.9\sqrt{3000}}{0.785}\left[6.67(20)(1.63 + 0.5)\left(0.92 + 0.08 \times \frac{5.5}{1.63}\right) + \ldots\right.$$

$$\left. \ldots + \overset{235.5}{300 \times 0.785}\right] = 36{,}020\ psi$$

USE 36 KSI



VERTICAL - # 8 @ 12"
10 - # 8 BARS ON EACH SIDE

COVER = 1.63"

$A_{s1} = A_{s2} = 10 \times 0.785 = 7.85\ in^2$

$n = \dfrac{29 \times 10^6}{3.1 \times 10^6} = 9.35$

$d_2 = 1.63 + \frac{1''}{2} = 2.13''$

$d_1 = 36 - 2.13 = 33.87''$

$D_s = 6''$

$P = P_{min} = 379^k$

"C" IS CALCULATED ON NEXT PAGE

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$$A = \frac{b_1 + b_2}{2n} = \frac{93 + 18}{2(9.35)} = \underline{5.94}$$

$$B = \frac{n-1}{n} \times A_{s2} + A_{s1} + \frac{b_1 D_s}{n} + \frac{P}{f_{sc}} =$$

$$= \frac{9.35-1}{9.35} \times 7.85 + 7.85 + \frac{18'' \times 6''}{9.35} + \frac{379}{36.0} =$$

$$= 7.01 + 7.85 + 11.55 + 10.52 = \underline{36.93}$$

$$C = \left(\frac{n-1}{n}\right) A_{s2} d_2 + A_{s1} d_1 - \frac{b_1 D_s}{n}\left(\frac{D_s}{2}\right) + \frac{P}{f_{sc}} d_1 =$$

$$= 7.01(2.13) + 7.85(33.87) - 11.55\left(\frac{6''}{2}\right) + 10.52(33.87) =$$

$$= 14.93 + 265.88 - 34.65 + 356.3 = \underline{602.47}$$

$$c = \frac{-B + \sqrt{B^2 + 4AC}}{2A} = \frac{-36.93 + \sqrt{36.93^2 + 4(5.94)602.47}}{2(5.94)} =$$

$$c = \underline{7.43 \text{ in}}$$

CALCULATE AXIAL FORCE AND MOMENT

$$P_{c1} = \frac{f_{sc} b_1 (c + D_s)^2}{2n(d_1 - c)} = \frac{36(18)(7.43 + 6)^2}{2(9.35)(\underbrace{33.87 - 7.43}_{26.44})} = \underline{236^K}$$

$$P_{c2} = \frac{f_{sc} b_2 (c)^2}{2n(d_1 - c)} = \frac{36(93)7.43^2}{2(9.35)\ 26.44} = \underline{374^K}$$

$$P_{s2} = \frac{f_{sc}(c - d_2)(n-1)}{(d_1 - c)\,n} A_{s2} = \frac{36(7.43 - 2.13)8.35}{26.44 \times 9.35} \times 7.85 = \underline{51^K}$$

$$P_{s1} = f_{sc} A_{s1} = 36 \times 7.85 = \underline{283^K}$$

$P = P_{c1} + P_{c2} + P_{s2} - P_{s1} =$

$= 236 + 374 + 51 - 283 = \underline{379^k}$ OK

MOMENT CAPACITY:

$M = P_{c1}\left[\frac{D}{2} - \frac{c+D_s}{3}\right] + P_{c2}\left[\frac{D}{2} - \frac{c}{3}\right] + P_{s2}\left[\frac{D}{2} - d_2\right] + \ldots$

$\ldots + P_{s1}\left[d_1 - \frac{D}{2}\right] =$

$= 236\left(18 - \frac{\overset{13.52}{7.43+6}}{3}\right) + 374\left(18 - \frac{\overset{15.52}{9.43}}{3}\right) + 51\left(\overset{15.87}{18 - 2.13}\right) + \ldots$

$+ 283\left(33.87 - 18\right) =$

$= 3190.7 + 5804.5 + 809.4 + 4491.2 = 14{,}298^{k-''}$

$M = \underline{1191^{k-'}}$

$CDFM = 1.0 \times \frac{1191}{611} \times \frac{1}{1.96} \times 0.4 = \underline{0.40g}$

PILONS @ LINES 12.9 & 17.03

(1) SHEAR CAPACITY:

* $V_c = 2\sqrt{3000} = 109.5$ psi.

* SHEAR REINFORCEMENT IS NOT CONSIDERED (SEE PILON @ LINE 15, PAGE 66) — PAGE 66

$\sigma_u = 0.85\, V_c = 0.85(109.5) = 93$ psi

$\sigma_{c,SSE} = 8$ psi. — PG 55

$$CDFM = 1.0 \times 93 \times \frac{1}{1.96} \times \frac{0.4}{8} = \underline{2.37g}$$

(2) BENDING CAPACITY

VERTICAL LOAD - USE 0.5 OF THE LOAD APPLIED @ PILON ON LINE 15

$P = 379/2 = \underline{190}^{K}$.

BENDING MOMENT

EACH PILON WILL SEE $(1-0.74)\frac{1}{2} = 0.13$ OF THE ENTIRE 127^{K} SHEAR LOAD APPLIED @ WALL # 10 — PG 55

$V = 127 \times 0.13 = \underline{16.5}^{K}$.

BENDING MOMENT:

$M = 16.5 \times \frac{13'}{2} = \underline{107}^{K-'}$

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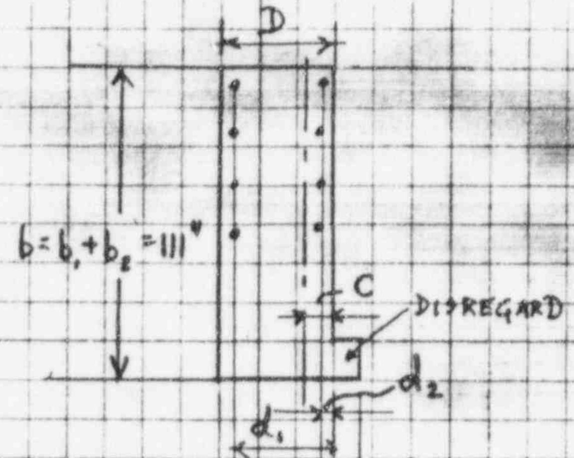
CALCULATE PILON MOMENT CAPACITY
(LINES 12.9 AND 17.03)

USE SAME INPUT AS FOR THE PILON ON LINE '15', EXCEPT



$D = 1'\text{-}8\tfrac{7}{8}'' = 20.87''$,

$b = b_1 + b_2 = 111''$

$D_s = 0$;

$P = P_{min} = \frac{379}{2} = \underline{190^k}$

$d_1 = 20.87'' - 2.13'' = 18.75''$

$d_2 = 2.13''$

CALCULATE C VALUE

$A = b/2n = 111/(9.35 \times 2) = \underline{5.94}$

$B = 7.01 + 7.85 + \frac{190}{36} = \underline{20.14}$

$C = 7.01(2.13) + 7.85(18.75) + \frac{190}{36} \times 18.75 = \underline{261.08}$

$c = \frac{-20.14 + \sqrt{20.14^2 + 4(5.94)261.08}}{2(5.94)} = \underline{\underline{5.15''}}$

$P_c = \frac{f_{sc}\, b}{2n(d_1 - c)} = \frac{36(111) \times 5.15^2}{2(9.35)(18.75 - 5.15)} = \underline{416.4^k}$

$P_{s2} = 36 \frac{(5.15 - 2.13)\, 8.35}{13.6\,(9.35)} \times 7.85 = \underline{56.0^k}$

$P_{s1} = 36 \times 7.85 = \underline{282.6}$

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$P = 416.4 + 56 - 282.6 = \underline{190^k}$ - ok

<u>MOMENT CAPACITY:</u>

$$M = 416\left(\frac{20.87}{2} - \frac{5.15}{3}\right) + 56\left(\frac{20.87}{2} - 2.13\right) + 282\left(18.75 - \frac{20.87}{2}\right) =$$

$$= 3625 + 465 + 2346 = 6436^{k-''} =$$

$$= \underline{536^{k-'}}$$

$$CDFM = 1.0 \times \frac{536}{107} \times \frac{1}{1.96} \times 0.4 = \underline{1.02g}$$

SINCE NO DUCTILITY IS CONSIDERED AND STIFFNESS OF THE PRECAST PANELS IS LOW THEIR CONTRIBUTION TO THE OVERALL CDFM VALUE IS IGNORED.
(SEE PGS 50, 50A, 50B)

PGS 50 50A 50B

<u>WALL 11. CONCRETE WALL ON ℄ 'Q'</u>

$h_w = 11.5'$; $L_w = 8.4'$ $t_w = 1.0'$

$h_w / L_w = 1.37$ $V = 220^k$ PG 55

$$\sigma_{c,SSE} = 220\,000 / (8.4 \times 1.0 \times 144) = 182 \text{ psi}$$

WALL '11' (CONT'D)

CALCULATE v_u

(1) $v_u = 6.8\sqrt{f_c'} - 2.8\sqrt{f_c'}\left[h_w/L_w - 0.5\right] + \frac{N_A}{4 l_w t_n} + \rho_{SE} f_y$

* $6.8\sqrt{f_c'} = 6.8\sqrt{3000} = \underline{372}$ psi
* $2.8\sqrt{f_c'}[\ ] = 2.8\sqrt{3000}\left[1.37 - 0.5\right] = \underline{133}$ psi
* $N_A/(4 l_w t_n) = 0$;
* CALCULATE ρ_{SE}

$A = -h_w/L_w + 1.5 = -1.37 + 1.5 = 0.13$;

$B = h_w/L_w - 0.5 = 1.37 - 0.5 = 0.87$;

$\rho_v = 2 \times 0.79/144 = 0.011$ (#8 @ 12")

$\rho_h = 2 \times 0.31/144 = 0.0043$ (#5 @ 12")

$f_y \rho_{SE} = A \rho_v f_y + B \rho_h f_y$

$f_y \rho_{SE} = 0.13 \times 0.011 \times 60 \times 10^3 + 0.87 \times 0.0043 \times 40 \times 10^3 =$

SEE NOTE PG 56 $= \underline{235}$ psi PG 56

$v_u = 372 - 133 + 0.7(235) = 404$ psi

COMPARE TO $7\sqrt{3000} = 383$ psi < 404 psi

$\therefore$ SHEAR FRICTION CAPACITY IS ADEQUATE

ADD SHEAR CAPACITY OF 4-ϕ1" A.BOLTS FROM COL INSIDE THE WALL. REF 8 C-198

$V_{AB} = 0.79 \times 4 \times 10\text{ksi} \times 1.5 = 47.4^K$

or $47400/(8.4 \times 144) = 39$ psi.

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$$\sigma_u = 0.6(404) + 39 = \underline{281}\text{ psi}$$

$$\text{CDFM} = 1.25 \times 281 \times \frac{1}{1.96} \times \frac{0.4}{182} = \underline{0.39\,g}$$

COMMENTS REGARDING WALL 10 CDFM VALUES

THE REINFORCED CONCRETE PILONS THAT FORM THE STRUCTURAL WALL '10' DO NOT MEET THE ACI CODE (REF 4) REQUIREMENTS ON FOLLOWING PARAGRAPHS: REF 4

1) 10.5.1 ρ_{min} IN BENDING $= \frac{200}{f_y} = \frac{200}{60\times10^3} = \underline{0.0033}$ PILON @ ℄ 15

ACTUAL $\rho = \frac{10\times0.79}{33\times111} = \underline{0.0022}$

2) 10.9.1 $\rho_{min} = 0.01$ IN COMPRESSION

ACTUAL $\rho = \frac{27\times0.79}{36\times111} = \underline{0.006 < 0.01}$

3) 7.10.5.3 TIE ARRANGEMENT. BECHTEL DWG C-199 SHOWS ONLY TWO TYPES OF TIES: ⊏ ⊐ IN THE T-PART OF THE PILON AND ⊓ - IN THE REST (90" LONG SIDE) OF THE PILON. REF 7

4) 21.4.4.1 (2) MIN SHEAR REINFORCEMENT.

CODE REQ'TS:

$$A_{SH} = 0.3\, s\, h_c \frac{f_c'}{f_{yh}}\left(\frac{A_g}{A_{CH}} - 1\right) =$$

$$= 0.3 \times 12 \times 31.75 \times 3000/40000 \left(\frac{3996}{3706} - 1\right) = \underline{0.67 in^2}$$

$$A_{SH} = 0.09\, s\, h_c \frac{f_c'}{f_{yh}} = 0.09 \times 12 \times 31.75 \times 0.08 = \underline{2.57 in^2}$$

ACTUAL: 2-#5@12" $A_{sh} = 0.62 in^2 < 0.67 < 2.57$

$h_c = 36" - 2(1 + 5/8 + 1/2) = 31.75"$

$A_g = 36" \times 111" = 3996 in^2$

$A_{CH} = (36"-2")(111"-2") = 3706 in^2$

5) 21.4.3.2 LAP SPICES LOCATION. CODE REQUIRES THEM @ THE MIDHIGHT OF THE PILON. THEIR ACTUAL LOCATION IS AT THE BOTTOM.