SARGENT & LUNDY
ENGINEERS
CHICAGO

ENRICO FERMI - 2 JOB NO. 6139-38

DETROIT EDISON CO.

RE-EVALUATION OF STRUCTURAL COMPONENTS FOR REVISED SEISMIC RESPONSE SPECTRA

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PROJECT NAME ENRICO FERMI
PROJECT NO. 6139-38 UNIT NO. 2
CLIENT DETROIT EDISON CO.
CALC. NO. & DESCRIPTION SF 0003
CALC. FOR REACTOR BLDG - MAT FOUNDATION - SEISMIC REEVALUATION

☒ SAFETY RELATED ☐ NON SAFETY RELATED

COMMENT NO. | QA SERIAL NUMBER

SIGNATURE & DATE FOR REV.			IDENTIFICATION OF PAGES PREPARED/REVISED/VOIDED & REVIEW METHOD
REV. 0	PREPARER	N.V. Hingorani 5-12-81	Prepared pages 1 & 2
	REVIEWER	J. Nandi 5-16-81	
	APPROVER	S.P. Gupta 5-18-81	REVIEW METHOD Detail Review Method
REV 0	PREPARER	J. Nandi 5-16-81	PREPARED PAGES 3 THRU 15
	REVIEWER	S. Dutta 5-16-81	
	APPROVER	S.P. Gupta 5-18-81	REVIEW METHOD Detail Review Method
REV __	PREPARER	J. Nandi 5-21-81	Calc. Page 6, Rev. 1
	REVIEWER	S. Dutta 5-21-81	
	APPROVER	N.V. Hingorani 5-21-81	REVIEW METHOD
REV __	PREPARER		
	REVIEWER		
	APPROVER		REVIEW METHOD

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Revision

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2.	MOMENT CAPACITY TABLE OF CRITICAL ELEMENTS OF MAT MODEL --- ---	2
3.	TOP & BOTTOM REINFORCEMENT PLAN ---	3 & 4
4.	CALCULATION OF INCREASED SSE MOMENT ---	7 THRU 14
5.	TABLE OF REVISED MAT CAPACITY FOR VARIOUS CRITICAL ELEMENTS ---	15

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The detail discussion regarding analysis, its procedure, assumptions and finally the conclusion of this analysis are well given in page no. 5 of this calculation.

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DESIGN CONTROL SUMMARY
DESIGN INPUT DOCUMENTS

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NO.	INPUT DOCUMENT			REVISION NUMBER OR DATE ON LINE INDICATED								USED IN DESIGN OF	CNT	C/N
	DESCRIPTION	SOURCE	IDENT	DATE	S	DATE	S	DATE	S	DATE	S			
1.	REACTOR BUILDING MAT FOUNDATION REVIEW FOR SRV LOADS	S&L	CALC. # SF-0002	1980										
2.	SEISMIC RE-ANALYSIS FOR 7% DAMPING SITE SPECTRA REAC.-AUX. BUILDING	S&L	CALC. # S00-DECO-003	APRIL 18, 1981										

CNT - Continued CN - Comment Number S - Status

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REACTOR BUILDING MAT :

GENERAL :

Design of Reactor Building mat was reviewed in early 1980 (refer calc. no. SF-0002) for hydrodynamic loads due to SRV & LOCA transfered to the mat by Torus Support. NW-quadrant of the mat was modelled by finite element method using SLSAP computer program.

PRESENT ANALYSIS :

All parameters, including geometry, member properties, dead loads, hydrostatic load, etc. have been kept unchanged ; only SSE loads have been increased by multiplying the seismic loads with appropriate factors using the forces obtained from the seismic report no. SDD-DECO-003 of 4-18-81 and shear wall moments computed in Shear wall calc. no. SC-0001.

Torus is supported at the mat level where the basic DBE acceleration of 0.15 g remains unchanged. However, we have multiplied the present seismic loads by a factor of 1.130 to account for lower frequency range.

EVALUATION :

In the following pages we have shown a plan of mat el. 540', top and bottom reinforcement in the mat and a table giving the moment capacities (Mu) of mat at various critical areas.

The table on page no. 18 gives increased bending moments in the critical areas under new SSE condition.

CONCLUSION :

At all sections $M_B < M_U$ (the moment capacity) : Slab is OK under new SSE. In combination with SSE, the mat can also safely take an uplift load of 1370 K at each Torus Support location.

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Attached sketch # SK-0001/SF-0003 shows the SLSAP finite element model used for review of Reactor Building mat. The following elements were identified to be the critical slab elements based on the computer output (see sketch # SK-0001/SF-0003 for location of those elements).

Element No.	Location	Tension	Mu ft-k
407	Near mid point of external North wall	Bottom	771
412	Midway between pedestal and mid-point of north wall (midway between Torus Supports.	Top	389
152	Midway between pedestal & west wall (in the center of Torus supports)	Top	389
226	At the junction of diagonal wall close to torus support	Bottom	505
314	Near the external North wall adjacent to the junction with diagonal walls	Bottom	575
343	Near the external North wall, close to torus support	Bottom	771
374	Another element close to external north wall	Bottom	771
385	Critical element close to pedestal and near the torus support	Bottom	765

1. Generally the elements closer to North wall were more critical than close to west wall due to heavier dead loads & seismic loads

2. Elements inside the triangular area span in two direction & therefore have very small moments.

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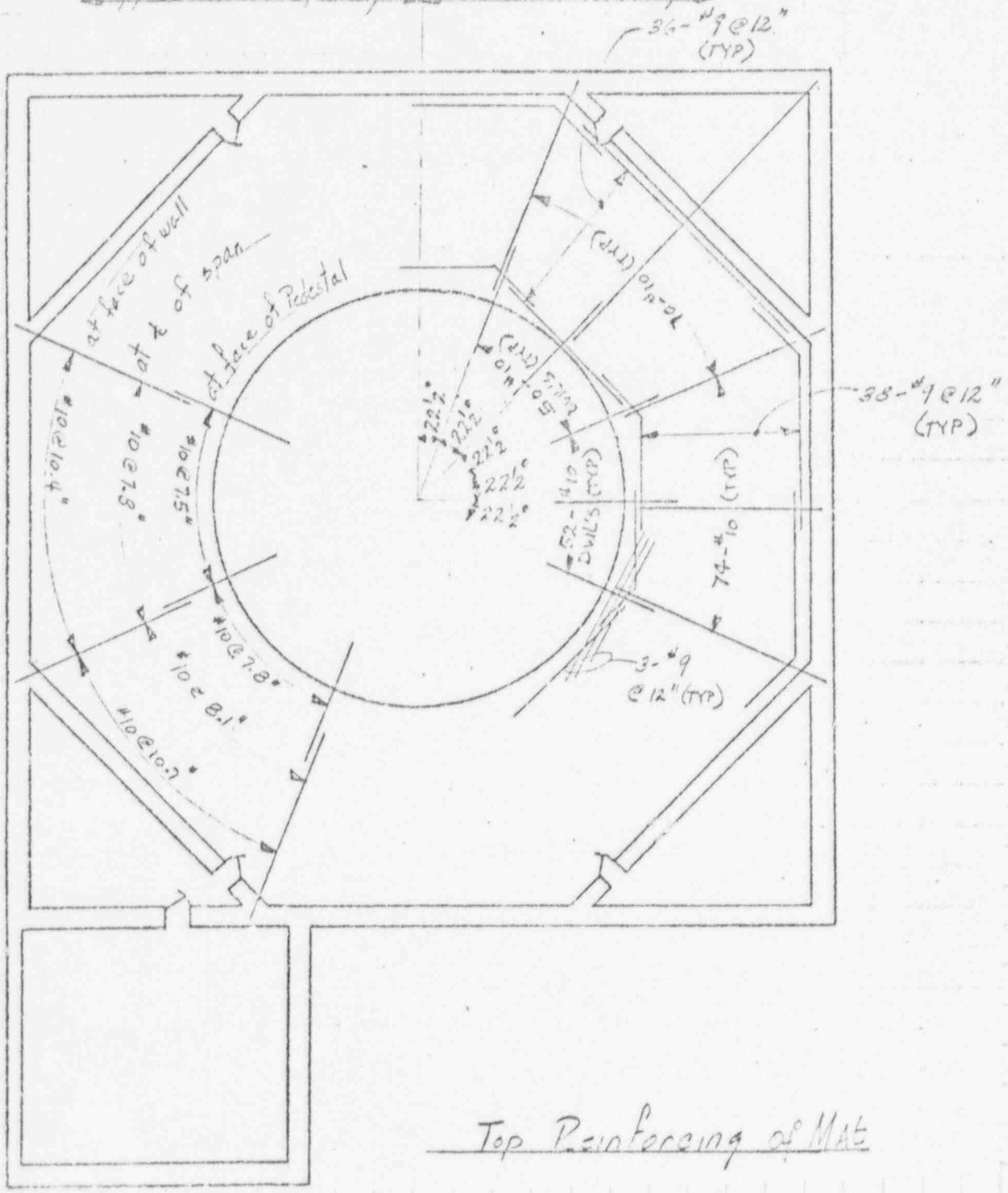
Approx. Rebar Spacing Ⓓ As billed on drwg's



Top Reinforcing of Mat

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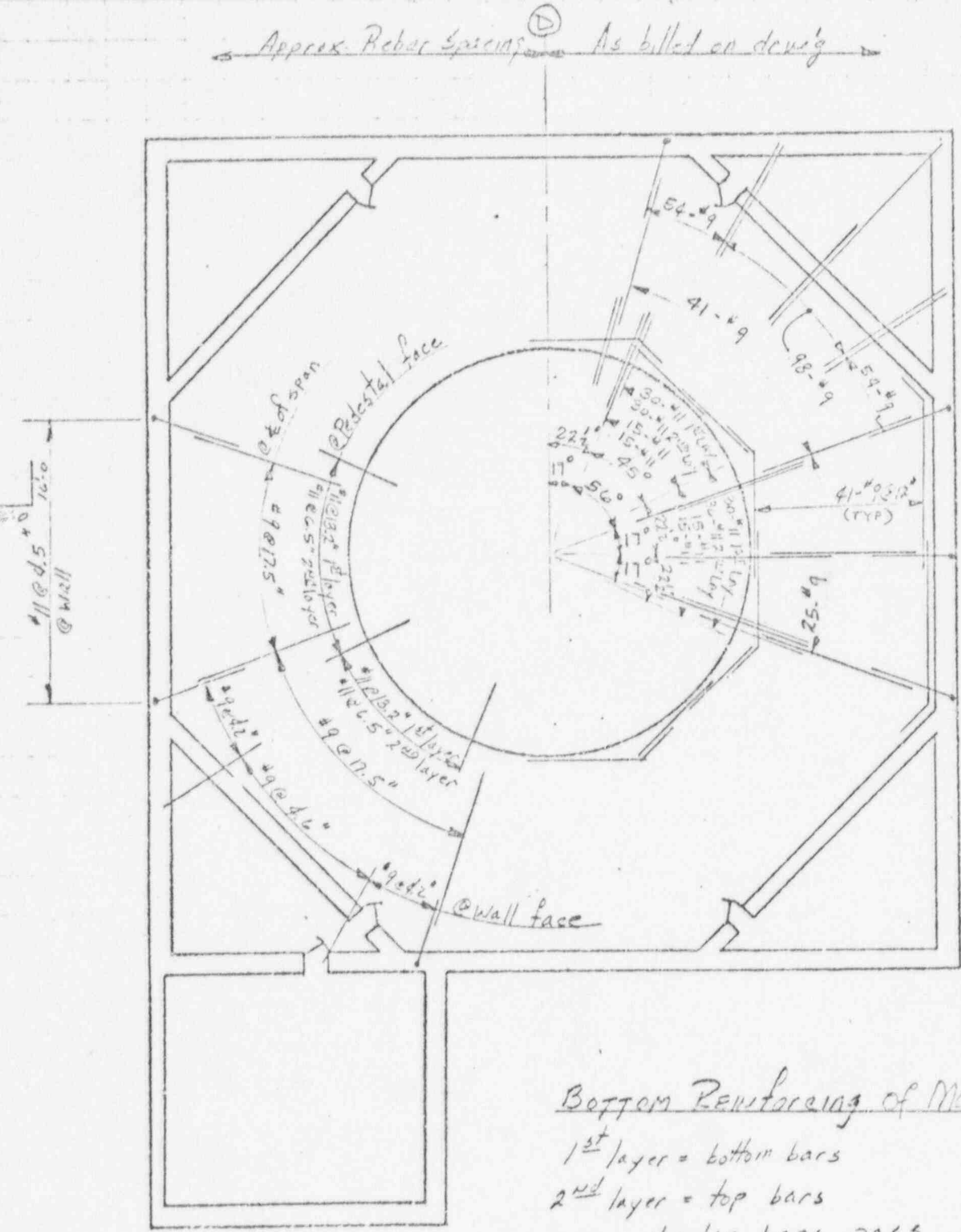
Approx. Rebar spacing — As billed on drw'g



Bottom Reinforcing of Mat

1st layer = bottom bars
2nd layer = top bars
perpendicular bars pass between layers. see drw'g B-16 sec. 5-5

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The Reactor Building Mat Foundation (quarter model) has previously been reviewed for the following seismic SSE loads & moments given in col. ① (see calc. book # SF-0002 for its sources). Col. ② shows the loads & moments for this analysis based on revised response spectra.

No.	ELEMENTS	LOADS & MOMENTS USED PREVIOUSLY	REFERENCE CALC. #	LOADS & MOMENTS USED IN THIS ANALYSIS	REFERENCE CALC. #
1.	WEST WALL ~ LINE A	562,270 FT-KIP.	P.5.12, # SF-0002	748,576 FT-KIPS	P.5, # SC-001
2.	NORTH WALL ~ LINE 17	1203,750 "	P.5.13, # SF-0002	1642,310 "	P.4, # SC-001
3.	DIAGONAL WALL				
	SPRING # 00106 N-S	$614^k \times 43.5 = 26,709$ FT-K	P.5.15, # SF-0002	$1362 \times 43.5 = 59,247$ Ft-K	P.9, # SC-001
	SPRING # 00109 E-W	$830 \times 43.5 = 36,105$ FT-K	P.5.15, # SF-0002	$1667 \times 43.5 = 72,514$ Ft-K	P.9, # SC-001
4.	CENTRAL PEDESTAL				
	X-EXCITATION	699,900 FT-K.	# SL-2682, SEPT '74	1585,000 FT-K.	Page 125 of Computer Output 6139-38, SD-Q-001 of 4-16-81
	Y-EXCITATION	570,000 FT-K.	# SL-2682, SEPT '74	1377,010 FT-K.	
5.	TORUS SUPPORTS AT OPERATING CONDITION	73,648 FT-K.	C.B.I. CALCS, SH. 38A11	33,257 FT-K.	C.B.I. Calcs.

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	① Ratio of SSE & OBE Loading in previous Calculation = A	② Ratio of New SSE & Old SSE Loading = B	③ New Multiplying factor = C = A × B
1. West wall in x-direction	1.452	1.505 *	2.185
2. Diag. wall in x-direction	1.440	$\frac{231}{112.18}$ = 2.059 *	2.965
3. Torus Loading in x-dir.	2.022	$\frac{0.52}{0.46}$ = 1.130	2.285
4. Pedestal Loading in x-dir.	1.425	$\frac{1585 \times 10^3}{699 \times 10^3}$ = 2.268	3.232
5. North Wall in y-direction	1.417	1.408 *	1.995
6. Diag. wall in y-direction	1.440	2.059 *	2.965
7. Torus Loading in y-direction	2.022	$\frac{0.52}{0.46}$ = 1.130	2.285
8. Pedestal Loading in y-direction	1.425	2.268	3.232

Note:- 1. The previous review of mat for 2000^k Torus Load were based on OBE loadings. The factors in col. ① were used to analyze the mat for SSE loadings for previous review.

2. Factors in col. ② are the ratios of new & old SSE loads. These factors were calculated from data given on Page 9.

3. Factors in col. ③ are now used for seismic re-evaluation.

REV.1 * These factors do not match the ratios of figure given in Page 5. However, the values are generally on the higher side and more conservative.

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Calculate New SSE Moment

In following pages 12 thru 17 we will be calculating the total moment in SSE condition during previous analysis. This was done by multiplying the OBE moments from previous analysis by the ratio of exist SSE & OBE. Thus the new SSE moments were obtained by multiplying the previous SSE moments by the ratio of new & previous SSE.

The elements (total 8) that are being selected now are the same elements that were critical in the previous analysis. To aid in determining the location of these elements, we have attached a sketch # SK-0001/SF-0003.

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SEISMIC MOMENT IN X - DIRECTION

ELEMENT NUMBER		West Wall			Diagonal Wall			Torus			Pedestal			Total Moment		
	Ratio of Existing SSE & OBE	1.452			1.440			2.022			1.425					
	Ratio of New & Exist. SSE	2.185			2.965			2.285			3.232					
	Load Condition \ Moment	Mx	My	Mxy	Mx	My	Mxy	Mx	My	Mxy	Mx	My	Mxy	Mx	My	Mxy
407	Exist. OBE	1.11	0.11	-0.03	1.4	0.18	0.13	-6.60	-1.32	.42	-11.1	-1.72	0.05	-15.19	-2.75	.57
	Exist. SSE	1.611	.16	-.04	2.02	.26	.19	-13.55	-2.67	.85	-15.82	-2.45	.07	-25.54	-4.7	1.07
	New SSE	3.52	.35	-.09	5.99	.77	.56	-27.0	-5.40	1.72	-51.1	-7.9	.23	-68.59	-12.18	2.42
412	Exist. OBE	.01	-.34	— (NEGLIGIBLE)	.34	-.46	-.01	5.20	-.87	-.36	.24	2.50	-.21			
	Exist. SSE	.015	-.50	— (NEGLIGIBLE)	.49	-.66	-.01	10.50	-.75	-.73	.34	3.6	-.80	11.85	1.69	-1.06
	New SSE	.03	-1.1	— (NEGLIGIBLE)	1.45	-1.96	-.03	24.0	-1.7	-1.67	1.1	11.6	-.97	26.53	6.84	-2.67
152	Exist. OBE	-.51	1.35	.92	-.25	.04	-.06	.82	.76	-.06	.02	.46	-2.3			
	Exist. SSE	-.74	1.96	1.34	-.36	.06	-.09	1.78	1.54	-.12	.03	.66	-3.28	0.71	4.22	-4.41
	New SSE	-1.62	4.28	2.93	-1.07	.18	-.27	4.07	3.5	-.27	.09	2.1	-10.6	1.47	10.1	13.0

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Calcs. For SEISMIC RE-EVALUATION OF REACTOR BUILDING MAT FOUNDATION

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Prepared by J. Naudi Date 5-14-81

Reviewed by A. Della Date 5-15-81

Approved by Date

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SEISMIC MOMENT IN X - DIRECTION

ELEMENT NUMBER		West Wall			Diagonal Wall			Torus			Pedestal			Total Moment		
	Ratio of Existing SSE ÷ OBE	1.452			1.440			2.022			1.425					
	Ratio of New ÷ Exist SSE	2.185			2.965			2.285			2.232					
	Load Condition \ Moment	Mx	My	Mxy	Mx	My	Mxy	Mx	My	Mxy	Mx	My	Mxy	Mx	My	Mxy
226	Exist OBE	5.1	1.6	2.3	4.8	1.2	1.4	-6.0	-1.5	-2.2	-9.3	-2.3	-2.2			
	Exist. SSE	7.41	2.32	3.34	6.9	1.7	2.0	-12.1	-3.0	-4.4	-13.3	-3.22	-3.14	-11.1	-2.3	-2.2
	New SSE	27.0	5.1	7.3	20.5	5.0	5.9	-27.6	6.8	-10.0	-43.	-10.6	-10.1	-23.1	6.3	-6.9
314	Exist. OBE	.44	-.06	-.06	.82	-.08	.70	-3.4	-.32	-1.76	-5.59	-3.97	-1.98			
	Exist. SSE	.64	-.09	-.09	1.18	-.12	1.00	-6.9	-.65	-3.56	-7.97	-5.62	-2.82	-13.1	-6.5	-5.5
	New SSE	1.4	-.20	-.20	3.5	-.36	3.0	-15.9	-1.50	-8.10	-25.8	-18.3	-9.1	-36.7	-20.4	-14.4
343	Exist. OBE	.68	-.09	-.50	.93	-.25	.09	-5.4	-.02	-.93	-7.37	-.72	-.45			
	Exist. SSE	.99	-.13	-.73	1.34	-.36	.13	-10.9	-.04	-1.88	-11.36	-1.03	-.64	-20.0	-1.6	-3.1
	New SSE	2.2	-.28	-1.60	4.0	-1.1	.39	-24.9	-.07	-4.3	-36.7	-3.3	-2.1	-55.4	-4.8	-7.6

SARGENT & LUNDY ENGINEERS CHICAGO

Calcs. For SEISMIC RE-EVALUATION OF REACTOR BUILDING MAT FOUNDATION

X Safety-Related | Non-Safety-Related

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Prepared by J. Nandi | Date 5-14-81

Reviewed by [illegible] | Date 5-15-81

Approved by | Date

SEISMIC MOMENT IN X - DIRECTION

ELEMENT NUMBER		WEST WALL			DIAGONAL WALL			TORUS			PEDESTAL			TOTAL MOMENT		
	RATIO OF EXISTING SSE ÷ OBE	1.452			1.440			2.022			1.425					
	RATIO OF NEW ÷ EXIST SSE	2.185			2.965			2.285			3.232					
	LOAD CONDITION \ MOMENT	M_x	M_Y	M_{XY}	M_x	M_Y	M_{XY}	M_x	M_Y	M_{XY}	M_x	M_Y	M_{XY}	M_x	M_Y	M_{XY}
374	EXIST. OBE	·93	·02	-·36	1.16	·01	·06	-6.9	-·70	·88	-9.9	-1.4	-·01			
	EXIST. SSE	1·35	·03	-·52	1·67	·01	·09	-14·0	-1·4	1·78	-14·1	-2·0	-·014	-25·1	-3·36	-1·34
	NEW SSE	2·96	·06	-1·14	5·0	·03	·27	-32·0	-3·2	4·1	-45·6	6·5	-·05	-69·6	-9·61	3·2
385	EXIST. OBE	-·02	-·34	-·01	-2·5	-·49	-·02	-11·0	-2·0	-·38	38·8	7·6	-·16			
	EXIST. SSE	-·03	-·49	-·01	-3·6	-·71	-·03	-22·2	-4·0	-·77	55·3	10·8	-·23	29·5	5·6	-1·04
	NEW SSE	-·06	-1·07	-·02	-10·7	-2·1	-·09	-50·7	-9·1	1·76	178·7	34·9	-·74	117·2	22·6	-2·61
	EXIST. OBE															
	EXIST. SSE															
	NEW SSE															

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Reviewed by S. Dutta Date 5-15-81
Approved by Date

SEISMIC MOMENT IN Y - DIRECTION

ELEMENT NUMBER		NORTH WALL			DIAGONAL WALL			TORUS			PEDESTAL			TOTAL MOMENT		
	RATIO OF EXISTING SSE ÷ OBE	1.417			1.440			2.022			1.425					
	RATIO OF NEW ÷ EXIST SSE	1.995			2.965			2.285			3.232					
	LOAD CONDITION / MOMENT	Mx	My	Mxy	Mx	My	Mxy	Mx	My	Mxy	Mx	My	Mxy	Mx	My	Mxy
407	EXIST. OBE	7.2	20.1	-12.5	-1.1	-2.9	1.6	-3.5	.23	4.2	-.56	.03	.43			
	EXIST. SSE	10.20	28.48	-17.71	-1.58	-4.2	2.3	-7.1	.47	8.5	-.80	.04	.61	.72	24.79	-6.30
	NEW SSE	20.35	56.32	-35.33	-4.69	-12.45	6.82	-16.22	1.074	19.42	-2.59	.13	1.97	-3.15	45.57	-7.12
412	EXIST. OBE	-.41	2.3	-3.4	-.01	-.31	.56	1.25	1.73	-1.45	.01	.15	2.23			
	EXIST. SSE	-.58	3.26	-4.82	-.01	-.45	.81	2.53	3.5	-2.93	.01	.21	3.18	1.95	6.52	-3.76
	NEW SSE	-.82	6.50	-9.62	-.03	-1.33	2.40	5.78	8.00	-6.70	.03	.68	10.28	4.96	13.85	-3.64
152	EXIST. OBE	-.03	-.70	—	.38	-.37	-.07	1.38	.18	.11	.22	2.63	.51			
	EXIST. SSE	-.04	-1.0	—	.55	-.53	-.10	2.79	.36	.22	.31	3.75	.73	3.61	2.58	.85
	NEW SSE	-.08	-1.995	—	1.63	-1.57	-.30	6.38	.82	.50	1.00	12.12	2.36	8.93	9.38	2.56

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SEISMIC MOMENT IN Y – DIRECTION

Element Number		North Wall			Diagonal Wall			Torus			Pedestal			Total Moment		
	Ratio of Existing SSE & OBE	1.417			1.440			2.022			1.425					
	Ratio of New & Exist. SSE	1.995			2.965			2.285			3.232					
	Load Condition \ Moment	Mx	My	Mxy	Mx	My	Mxy	Mx	My	Mxy	Mx	My	Mxy	Mx	My	Mxy
226	Exist. OBE	10.2	3.17	4.65	-2.14	-.46	-.26	-6.24	-1.7	-2.34	-6.26	-1.2	-1.0			
	Exist. SSE	14.45	4.49	6.59	-3.08	-.66	-.37	-12.6	-3.4	-4.73	-2.92	-1.7	-1.43	-10.15	-1.27	0.06
	New SSE	28.83	8.96	13.15	-9.13	-1.96	-1.10	-28.79	-7.77	-10.81	-28.83	-5.49	-4.62	-37.92	-6.26	-3.88
314	Exist. OBE	2.72	1.25	-.47	-.78	-.12	1.06	-4.67	-1.05	-4.12	-2.0	.08	-.39			
	Exist. SSE	3.85	1.77	-.67	-1.12	-.17	1.53	-9.4	-2.12	-8.33	-2.8	.11	-.56	-9.47	-0.41	-8.03
	New SSE	7.68	3.53	-1.34	-3.32	-.50	4.54	-21.48	-4.84	-19.03	-9.05	.35	-1.81	-26.17	-1.46	-17.64
343	Exist. OBE	3.81	1.50	-2.86	-.74	.08	.41	-7.56	1.89	-5.35	-2.11	-.02	.29			
	Exist. SSE	5.40	2.13	-4.05	-1.07	.12	.59	-15.3	3.82	-10.82	-3.0	-.31	.41	-13.97	5.76	-13.87
	New SSE	10.77	42.42	-8.08	-3.17	.36	1.75	-34.96	8.73	-24.72	-9.70	-1.00	1.33	-37.06	50.51	-29.7[illegible]

Form GQ 3.08.1 Rev. 2

SARGENT & LUNDY
ENGINEERS
CHICAGO

Calcs. For SEISMIC RE-EVALUATION OF REACTOR BUILDING MAT FOUNDATION

X Safety-Related | Non-Safety-Related

Calc. No. SF-0009 | Rev. 0 | Date 5-14-81 | Page 13 of

Client DETROIT EDISON CO.
Project FERMI - 2
Proj. No. 6139-38 Equip. No.

Prepared by J. Nandi Date 5-14-81
Reviewed by G. Dutt Date 5-16-81
Approved by Date

SEISMIC MOMENT IN Y - DIRECTION

ELEMENT NUMBER		NORTH WALL			DIAGONAL WALL			TORUS			PEDESTAL					
	RATIO OF EXISTING SSE ÷ OBE	1.417			1.440			2.022			1.425					
	RATIO OF NEW ÷ EXIST SSE	1.995			2.965			2.285			3.232			TOTAL MOMENT		
	LOAD CONDITION \ MOMENT	Mx	My	Mxy	Mx	My	Mxy	Mx	My	Mxy	Mx	My	Mxy	Mx	My	Mxy
374	EXIST. OBE	6.80	3.57	-6.34	-1.09	-.32	.87	-8.86	.10	7.36	-1.60	-.25	.41			
	EXIST. SSE	9.64	5.06	-8.98	-1.57	-.55	1.25	-17.9	.20	14.9	-2.28	-.36	.58	-12.11	4.35	7.75
	NEW SSE	19.2	10.1	-17.9	-4.66	-1.63	3.71	-40.9	.46	34.0	-7.4	-1.16	1.87	-33.76	7.77	21.68
385	EXIST. OBE	-3.30	-.59	-.39	.89	.16	.11	-8.9	-1.45	-1.19	5.50	1.03	1.05			
	EXIST. SSE	-4.62	-.84	-.55	1.28	.23	.16	-18.0	-2.93	-2.41	7.84	1.47	1.50	-13.56	-2.07	-1.30
	NEW SSE	-9.3	-1.67	-1.1	3.79	.68	.47	-41.1	-6.7	-5.5	25.3	4.75	4.85	-21.31	-2.94	-1.28
	EXIST. OBE															
	EXIST. SSE															
	NEW SSE															

SARGENT & LUNDY ENGINEERS CHICAGO

Calcs. For	Seismic Re-evaluation of Reactor Building Mat Foundation
X Safety-Related	Non-Safety-Related
Calc. No.	SF-0003
Rev.	0
Date	5-14-81
Page	14 of
Client	D. E. Co.
Project	FERMI-2
Proj. No.	6138-38
Equip. No.	
Prepared by	J. Nandi
Date	5-14-81
Reviewed by	[illegible]
Date	5-16-81
Approved by	
Date	

Table of SSE Moment Increament

Elem. No.	Moment	S. R. S. S.			Total		Increase in SSE Moment		Remarks
		Mx	My	Mxy	Mx=Mx+Mxy	My=My+Mxy	Mxi	Myi	
407	Exist SSE	25.6	25.2	6.4	32.0	31.6			
	New SSE	68.7	47.2	7.5	76.2	54.7	44	23	
412	Exist SSE	11.5	6.7	3.9	15.4	10.6			
	New SSE	27.0	15.4	4.5	31.5	20.9	16	10	
152	Exist SSE	3.7	4.9	4.5	8.2	9.4			
	New SSE	9.1	13.8	13.2	22.3	27.0	14	18	
226	Exist SSE	15.0	2.6	2.2	17.2	4.8			
	New SSE	44.4	8.9	7.7	52.1	16.6	35	12	
314	Exist SSE	16.2	6.5	9.7	25.9	16.2			
	New SSE	45.1	20.5	22.8	67.9	43.8	42	28	
343	Exist SSE	24.4	6.0	14.2	38.6	20.2			
	New SSE	66.7	50.7	30.7	97.4	81.4	59	61	
374	Exist SSE	27.9	5.5	7.9	35.8	13.4			
	New SSE	77.4	12.4	21.9	99.3	34.3	64	21	
385	Exist SSE	32.5	6.0	1.7	34.2	7.7			
	New SSE	119.1	22.8	2.9	122.0	25.7	88	18	

Revised Capacity of Slab due to Torus Uplift

In the following page-19. we have presented a revised mat capacity table of which some results were taken from calc. # SF-0002. A few explanatory notes will aid in the clarity of the table.

Notes:-1. Loading #9 & 15 = $0.9D + 1.1T + 1.0H \pm 1.0E_s$

2. Loading #10 & 16 = $0.9D + 1.0H \pm 1.0E_s$

3. In note 1 & 2, D = Dead Load, T = Torus Uplift Load, H = Hydrostatic Load, & E_s = SSE Load. "+" indicates load acting upward & "−" downward.

4. $-E_s$ applies to loading 10 & 15

5. M_T = Induced moment in the mat due to torus uplift only

6. M_B = Induced moment in the mat due to all loading except torus load

7. Both values of M_B & M_T were obtained from Calc.# SF-0002.

Form GO-3.08.1 Rev. 2

SARGENT & LUNDY ENGINEERS CHICAGO	Calcs. For SEISMIC REEVALUATION OF REACTOR BUILDING MAT FOUNDATION	Calc. No. SF-0003
	X Safety-Related / Non-Safety-Related	Rev. 0 Date 5-14-81
		Page 15 of FINAL

Client	D. E. Co.	Prepared by J. Nardi	Date 5-14-81
Project	FERMI-2	Reviewed by S. Gutta	Date 5-16-81
Proj. No.	6139-38 Equip. No.	Approved by	Date

Revised Capacity of Slab due to Torus Uplift

Elem. No.	Loading No.	$M_B^R = M_B \pm M_{xi}$ (Ft-Kips)	M_U (Ft-Kips)	M_T (Ft-Kips)	$M'_T = M_U - M_B^R$ (Ft-Kips)	$P_T^R = 2000 \times \frac{M'_T}{M_T}$ (Kips)
407	9&10	-369+44=-325	771	-177	-446	5039^K
	15&16	-432-44=-476	771	-177	-295	3333^K
412	9&10	120+16=136	389	144	253	3514^K
	15&16	89-16= 73	389	144	316	4329
152	9&10	113+14= 127	389	146	262	3589
	15&16	102-14= 88	389	146	301	4123
226	9&10	-156+35=-121	505	-242	-384	3174
	15&16	-190-35=-225	505	-242	-280	2314
314	9&10	-183+42=-141	575	-225	-434	3857
	15&16	-218-42=-260	575	-190	-315	3315
343	9&10	-172+59 =-119	771	-165	-652	7903
	15&16	-256-59 =-315	771	-169	-456	5396
374	9&10	-286+64 =-222	771	-171	-549	6421
	15&16	-350-64 =-414	771	-178	-357	4011
385	9&10	-250+88=-162	765	-500	-603	2412
	15&16	-300-88=-388	765	-550	-377	1370

Form GQ 3.08.1 Rev. 2

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SARGENT & LUNDY
ENGINEERS

DESIGN CONTROL SUMMARY
DESIGN VERIFICATION

PAGE 1

PROJECT NAME
PROJECT NO. 6130-33 UNIT NO.
CLIENT
CALC. NO. & DESCRIPTION AF 01
CALC. FOR

☑ SAFETY RELATED ☐ NON SAFETY RELATED

COMMENT NO.
QA SERIAL NUMBER

SIGNATURE & DATE FOR REV. 0		IDENTIFICATION OF PAGES PREPARED/REVISED/VOIDED & REVIEW METHOD
PREPARER	Signatures & dates appear on individual pages	Pages 1-15 (Final)
REVIEWER	Signatures & dates appear on individual pages	
APPROVER	[signature] 5-18-81	REVIEW METHOD Detail Review

SIGNATURE & DATE FOR REV ___		IDENTIFICATION OF PAGES PREPARED/REVISED/VOIDED & REVIEW METHOD
PREPARER		
REVIEWER		
APPROVER		REVIEW METHOD

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PREPARER		
REVIEWER		
APPROVER		REVIEW METHOD

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PREPARER		
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APPROVER		REVIEW METHOD

FORM SAS 22.1 Rev. 0

SARGENT & LUNDY
ENGINEERS

DESIGN CONTROL SUMMARY
DESIGN ELEMENT INDEX
PROJECT NO. 4135-38 UNIT NO. 2

PAGE ___ OF CALCULATIONS NO. AF-01 REV. 0 DATE 5-15-1981

No.	DESCRIPTION	PAGES
1.	General	1, 2
2.	'Strudl' Input Preparation for Area 1	3 - 6
3.	Moment Diagram from 'Strudl' Output (Area 1)	7
4.	Computer Run Control Sheet (Area 1)	8
5.	'Strudl' Input Preparation for Area 2	9 - 12
6.	Moment Diagram from 'Strudl' Output (Area 2)	13, 14
7.	Computer Run Control Sheet (Area 2)	15

Revision

FORM SAS 22.3 Rev. 0

Auxiliary Building Mat comprises two distinct areas at El. 540'-0 and El. 551'-0 respectively. These two areas have been reviewed separately for new SSE loads in combination with subsoil water at El. 576'-0 and El. 558'-3. The moments in base mat have been computed with and without lateral soil pressure. The presence of Turbine Building, with base mat at El. 564'-0, reduces the pressures on wall on line H of Auxiliary Building.

The detailed procedure for review of the mat is given on pages 1 & 2. The assumption and reference are given in the calculation wherever used.

For conclusion refer computer run control sheets on pages 8 and 15.

SARGENT & LUNDY
ENGINEERS

DESIGN CONTROL SUMMARY
DESIGN INPUT DOCUMENTS

PROJECT NO. [illegible] UNIT NO. 2

PAGE ___ OF CALCULATIONS NO. AF-01 REV. 0 DATE [illegible]

FORM SAS 22.7 Rev. 0

NO.	INPUT DOCUMENT: DESCRIPTION	SOURCE	IDENT	REVISION NUMBER OR DATE ON LINE INDICATED: DATE	S	DATE	S	DATE	S	DATE	S	USED IN DESIGN OF	CNT	C/N
1.	Seismic Re-analysis for 7% Damping Site Spectra Reactor Aux. Bldg	S&L	Report No. SCR-0[illegible] -003									Aux. Bldg. Mat		
2.	Seismic Analysis of the Reactor Aux. Building Complex	S&L	Report No. SL-[illegible]	Sept. 27, 7[illegible]								"		
3.	Building Code Requirements for Reinforced Concrete	ACI	ACI-318 -77									"		
4.	Superstructure Wall Design Above Grade	S&L	Calc. No. 4.01.06									"		
5.	Substructure Column Loads	S&L	Calc. No. 3.01.00									"		
6.	Rx/Aux. Bldg. Shear Walls - Seismic Re-evaluation	S&L	Calc. No. SC-0001									"		

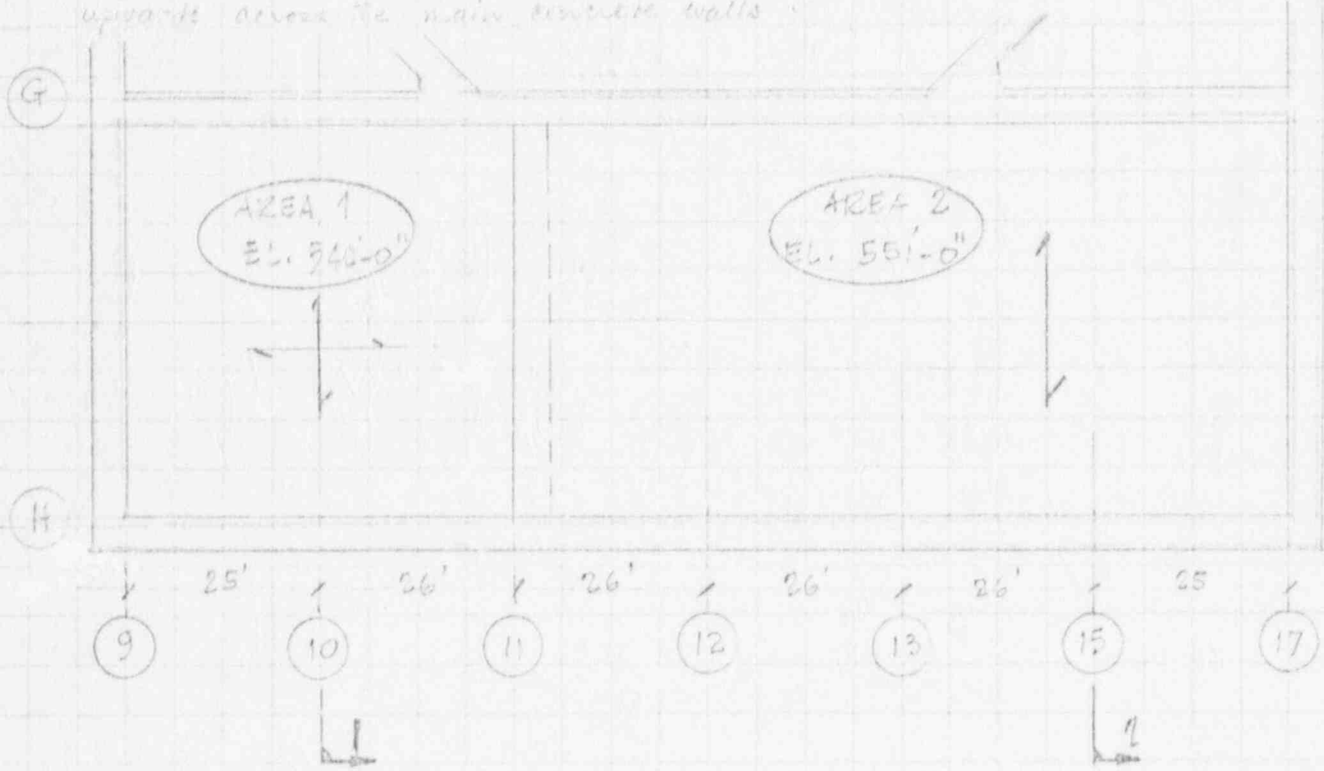
CNT - Continued CN - Comment Number S - Status

SARGENT & LUNDY ENGINEERS CHICAGO	Calcs. For Auxiliary Bldg Mat Seismic Reevaluation	Calc. No. AF-01
	✓ Safety–Related / Non–Safety–Related	Rev. 0 Date / Page 1 of

Client Detroit Edison Co.		Prepared by N. V. Hongsmon	Date 5-10-81
Project Enrico Fermi-2		Reviewed by S. Della	Date 5-15-81
Proj. No. 6139-38	Equip. No.	Approved by	Date

AUXILIARY BUILDING MAT :

For the existing design of the mat the governing load condition is the hydrostatic pressure of 40' above the mat bottom which tends to bend the slab upwards across the main concrete walls.



- Area 1 was considered as spanning in two directions with moment continuity at each of the walls G, H, 9 & 11.
- Area 2 was considered as slab spanning in one direction between walls G & H with moment continuity at both ends.

Soil, hydrostatic pressure & surcharge pressures were considered acting on the exterior walls with supports at El 583'-0", 562'-0" & moment continuity at 540'-0" for Area 1 & support at El. 583'-0" and moment continuity at El. 551'-0" for Area 2.

The design was based on allowable stresses of $f_c = 1800$ psi $(0.45 f'_c)$ and $f_{sc} = 24{,}000$ psi.

Form GQ 3.08.1 Rev. 2

SARGENT & LUNDY ENGINEERS CHICAGO	Calcs. For Auxiliary Bldg Mat — Seismic Re-evaluation	Calc. No. AF-01
	✓ Safety-Related / Non-Safety-Related	Rev. 0 Date
		Page 2 of

Client Detroit Edison Co.	Prepared by N. V. Hrishikesan	Date 5-12-81
Project Enrico Fermi - 2	Reviewed by G. Patta	Date 5-12-81
Proj. No. 6911-22 Equip. No.	Approved by	Date

REINFORCEMENT has been provided as below:

		AREA 1		AREA-2		M_u (k'/1) AREA 1	AREA 2
Near the walls :	Bottom	#11 @ 6"	3.12 □"	#11 @ 4½" #11 @ 24"	4.68 □"	581.	849.
	Top	#9 @ 12"	1.0 □"			195.	
Mid Span	Bottom	#9 @ 12"	1.0 □"			195.	
	Top	#9 @ 6"	2.0 □"	#9 @ 7"	1.71 □"	383.	329.

Under seismic moments there will be additional loads on the walls upwards or downwards. If some part of the mat on soil tends uplift under the combined effect to seismic forces and hydrostatic pressure; there will not be any soil reaction on the mat and the slab will span between the walls supporting the loads directly applied to it e.g. self wt, equipment load and hydrostatic upward load. This situation is similar to the original design basis as described in page 1.

When the seismic forces cause additional downward load on the walls, there will be increase in the soil pressure and its distribution will depend upon the modulus of the sub-grade reaction (k)

Two separate section of the building mat shall be reviewed for this loading condition with SEISMIC model using K = 1000 K/ft³ (Refer Calc SF-0002 for Reactor Bldg mat)

For each of these sections two subsoil water levels shall be considered e.g. EL: 576'-0" normal water level

EL: 552'-0" low water level.

Form GQ.3.08.1 Rev. 2

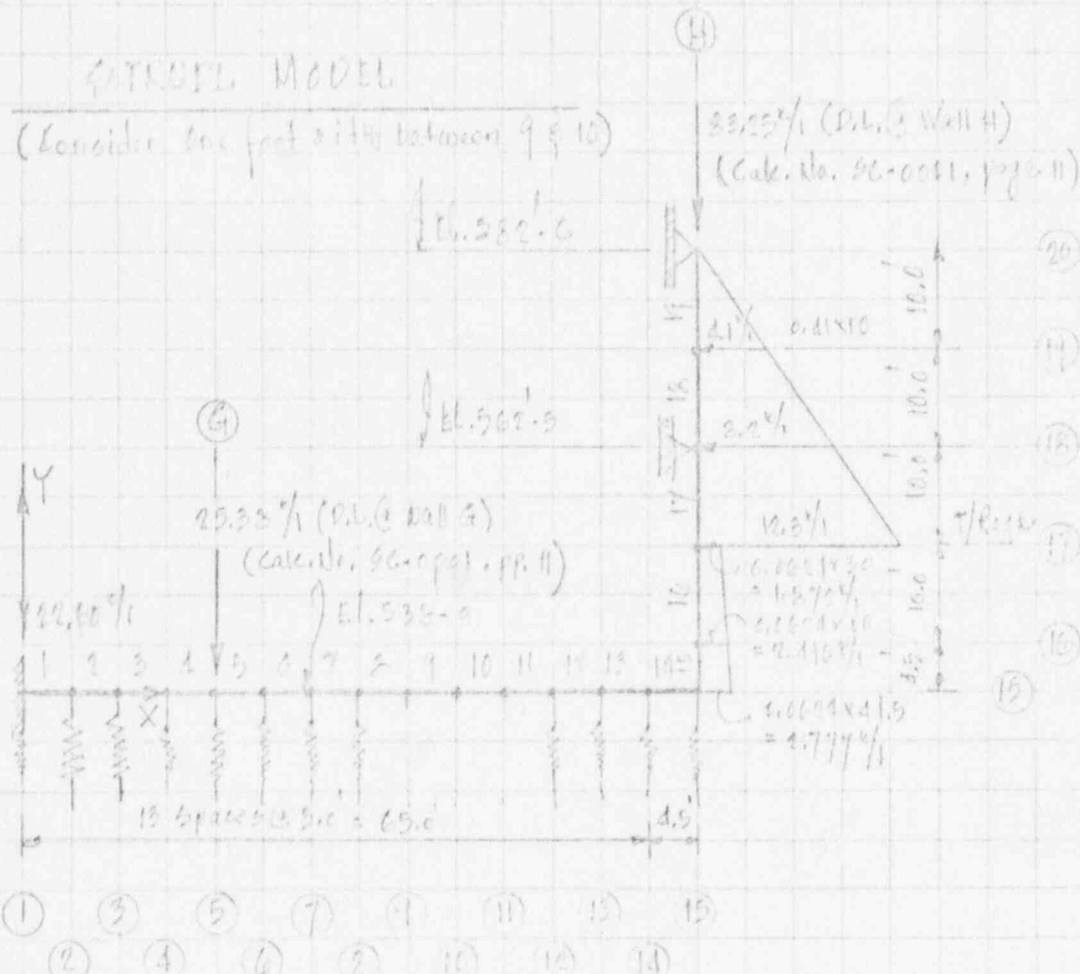
SARGENT & LUNDY ENGINEERS CHICAGO	Calcs. For Aux. Bldg. Mat (1) Evaluation for Revised Seismic Forces	Calc. No. AF-01
	✓ Safety-Related / Non-Safety-Related	Rev. 0 Date / Page 3 of

Client: Detroit Edison Co.	Prepared by: G. Vullo	Date: 5/7/81	
Project: Enrico Fermi-2	Reviewed by: J. Nandi	Date: 5-16-81	
Proj. No. 4132-38	Equip. No.	Approved by	Date

MAT SECTION: 1-1

GTSTRUDL MODEL

(Consider one foot width between 9 & 10)



Soil Modulus, K = 1000 k/ft³ (same as Reactor Building mat)

$K_1 = 1000 \times 2.5 = 2500\ ^{k}/_{ft^2}$

K_2 To K_8, K_{12}, K_{13} = $1000 \times 5.0 = 5000\ ^{k}/_{ft^3}$

$K_{14} = 1000 \times 4.75 = 4750\ ^{k}/_{ft^2}$

$K_{15} = 1000 \times 4.5 = 4500\ ^{k}/_{ft^2}$

Form GO-3.08.1 Rev. 2

SARGENT & LUNDY ENGINEERS CHICAGO	Calcs. For Aux. Bldg. Mat (1) Evaluation for Revised Seismic Forces		Calc. No. AF-41	
	✓ Safety-Related	Non-Safety-Related	Rev. 0	Date
			Page 1	of

Client Detroit Edison Co.	Prepared by E. Putts	Date 5/12/81
Project Enrico Fermi - 2	Reviewed by J. Nandi	Date 5-15-81
Proj. No. 6139-38 Equip. No.	Approved by	Date

'STRUDL' PLANE FRAME (XY plane)

E_c = 3620000 psi (for f'c = 4000 psi) = 521280 k/ft²

Support Joints : 1 thru 15, 18, 20

Joint Releases:
- 2 thru' 8, 12, 13, 14, 15 KFY (20 per x Value on pg 6)
- 9, 10, 11, 18, 20 Force Y
- 2 thru' 15, 18 Moment Z
- 2 thru' 15 Force X

Member Release : 19 End Moment Z

SARGENT & LUNDY ENGINEERS CHICAGO

Calcs. For	Aux. Bldg. Mat (1) Evaluation for (Revised) Seismic	Calc. No.	AF-C1
✓ Safety-Related	Non-Safety-Related	Rev. 0	Date
		Page	of

Client	Detroit Edison Co.	Prepared by	S. Dutta	Date 5/15/87
Project	Enrico Fermi - 2	Reviewed by	J. Nanti	Date 5-16-87
Proj. No. 6139-38	Equip. No.	Approved by		Date

Loading

1/ Dead Load :

Jt. 1 (Diagonals) (See calc. SF-0002, P11) = 1/2(12 + 3.25' × 0.15 × 41.0') = 22.6^k/1'

Jt. 5 (Wall G) = 25.32^k/1 ↓ (Calc. No. SC-0001, p11)

Jt. 20 (Wall H) = 83.25^k/1 ↓ (")

2/ Lateral Soil Pressure : (Refer. FSAR.)

Use 410#/ft³ above T/Rock [≈ (96+320) or (122+280)]

62.4#/ft³ below T/Rock [Neglect Soil Pressure below T/Rock as the excavated rock fill is only 5'0" wide]

3/ Hydrostatic Pressure : (Spanning two ways, hence 1/2 pressure used)

a) Normal Water Level El. 576'-0

$\frac{1}{2}[(576.0-536.0) \times 0.0624] - 4 \times 0.15 = 0.648 \approx 0.65\ ^k/_{\square'}$ ↑

(mat thickness)

b) Low Water Level El. 558'-8

$\frac{1}{2}[(558.67-536.0) \times 0.0624] - 4 \times 0.15 = 0.107$ ↑ $^k/_{\square'}$

($0.107/0.65 = 0.165$, ∴ Loading b) = 0.165 (Loading a))

SARGENT & LUNDY ENGINEERS CHICAGO	Calcs. For Aux. Bldg. Mat (1) Evaluation for Revised Seismic Forces	Calc. No. AB-61
	✓ Safety-Related / Non-Safety-Related	Rev. 0 Date
		Page 6 of

Client Detroit Edison Co.		Prepared by S. Dutta	Date 5/12/81
Project Enrico Fermi - 2		Reviewed by J. Nandi	Date 5-16-81
Proj. No. 6139-32	Equip. No.	Approved by	Date

Loading (Cont.)

4) Seismic Load (SSE) $\sqrt{(X^2+Y^2+Z^2)}$

Combine the effect of three components by 'SRSS' method

See page 18

a) Corner H-9 : $\pm\sqrt{(11.79)^2+(14.80)^2+\left(\frac{0.25\times 23.15}{3}\right)^2} = \pm 20.163^k/_1$

$\text{Load/ft.} = \pm 20.163 \times 3 = 60.505^k/_1 \downarrow$

wall thickness

b) G-9 : $\sqrt{\left(\frac{M_{YY}}{I_{YY}}\cdot C_{XX}(t)\right)^2+\left(\frac{M_{XX}}{I_{XX}}\cdot C_{YY}(t)\right)^2+(0.25\cdot D.L)^2}$

$= \sqrt{\left(\frac{3256138}{29320735}\times 56.12\times 4\right)^2+\left(\frac{3316495}{13261386}\times 59.85\times 4\right)^2+(0.25\times 25.38)^2}$

$= \sqrt{(24.93)^2+(59.26)^2+(6.35)^2}$

$= 64.60\ ^k/_1 \downarrow$

c) Diagonal : $0.25 \times 22.6 = 5.65^k/_1 \downarrow$

LOADING COMBINATIONS :

Loading Combination 5 Combine ① 1.0 ② 1.0 ③ 1.0 ④ 1.0

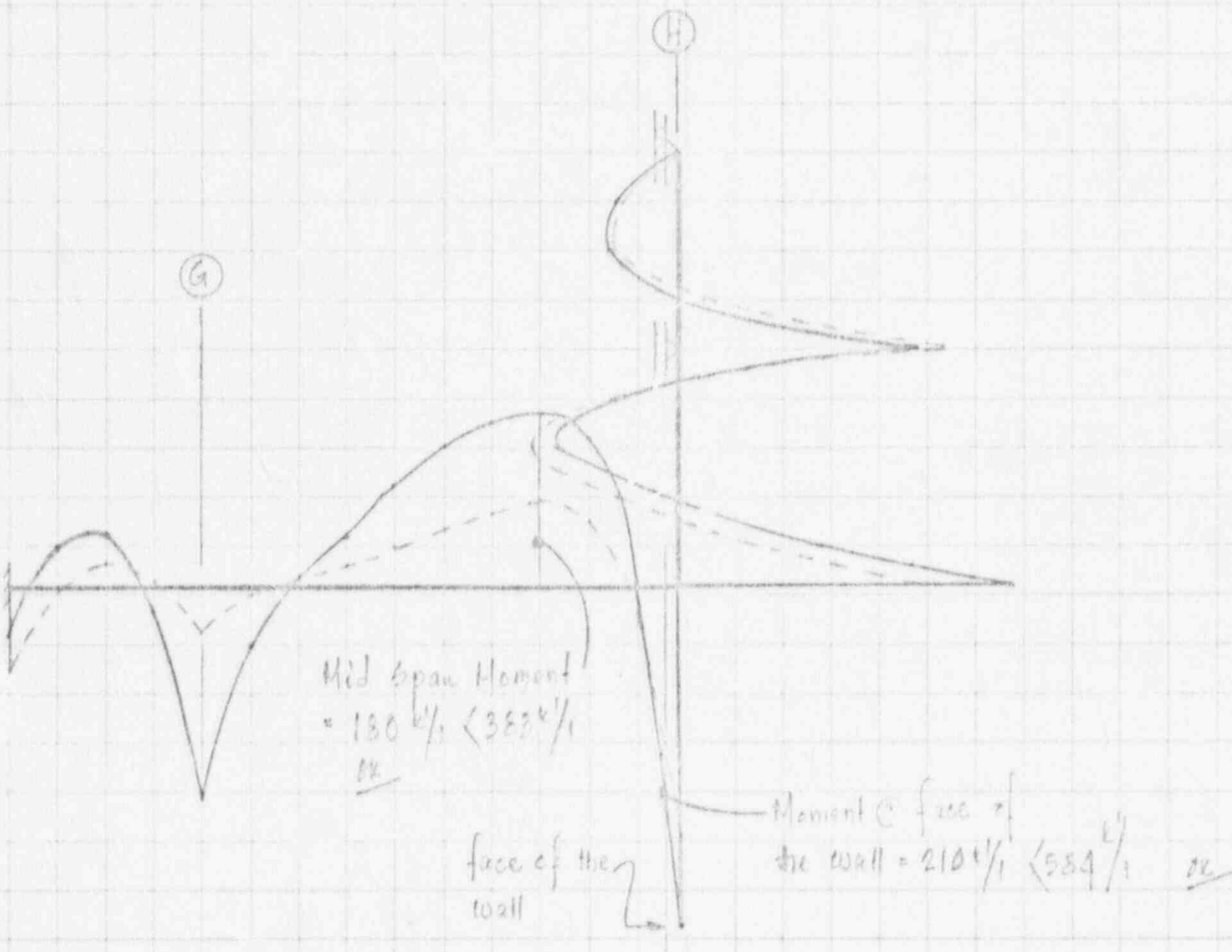
Loading Combination 6 Combine ① 1.0 ② 1.0 ③ 0.165

Form GQ 3.08.1 Rev. 2

SARGENT & LUNDY ENGINEERS CHICAGO	Calcs. For Aux. Bldg. Mat(1) ~ Review Evaluation for Revised Seismic Forces	Calc. No. AF-01
	✓ Safety-Related / Non-Safety-Related	Rev. 0 Date
		Page 7 of

Client	Detroit Edison Co.	Prepared by	S. Dutta	Date 5/14/81
Project	Enrico Fermi-II	Reviewed by	J. Nardi	Date 5-16-81
Proj. No.	6139-38 Equip. No.	Approved by		Date

Ref.: 'STRUDL' output A462SD dt. 5-13-1981



Bending Moment Diagram (k'/1)

Scale : 1" = 200 k'/1 , 1" = 20'-0"

Loading Combination 5 : ——

Loading Combination 6 : - - -

SARGENT & LUNDY ENGINEERS CHICAGO	Calcs. For Re-evaluation of Aux. Bldg. Mat COMPUTER RUN CONTROL SHEET	Calc. No. AF-01
	✓ Safety-Related / ☐ Non-Safety-Related	Rev. 0 Date 5-16-81 Page 8 of

Client	DETROIT EDISON COMPANY	Prepared by S. Dutta	Date 5-16-81
Project	FERMI 2	Reviewed by J. Nandi	Date 5-16-81
Proj. No. 6159-38	Equip. No.	Approved by	Date

RUN INFO

RUN ID	DATE	PROGRAM I.D.	CPU	SRM	NO. OF PAGES	NO. OF MICROFICHE
A4626D	5-13-81	SLS 09704342			61	1

REFERENCE

MICROFICHE I.D.	MODEL	DRAWINGS
Aux. Building Mat-1 AF-01	See page 3	B-23

PREPARER'S REMARKS (INCLUDING RUN DESCRIPTION)

One foot wide strip of mat & wall is used to model the area of aux. bldg. mat. The loads applied and boundary conditions used are explained in the calculations.

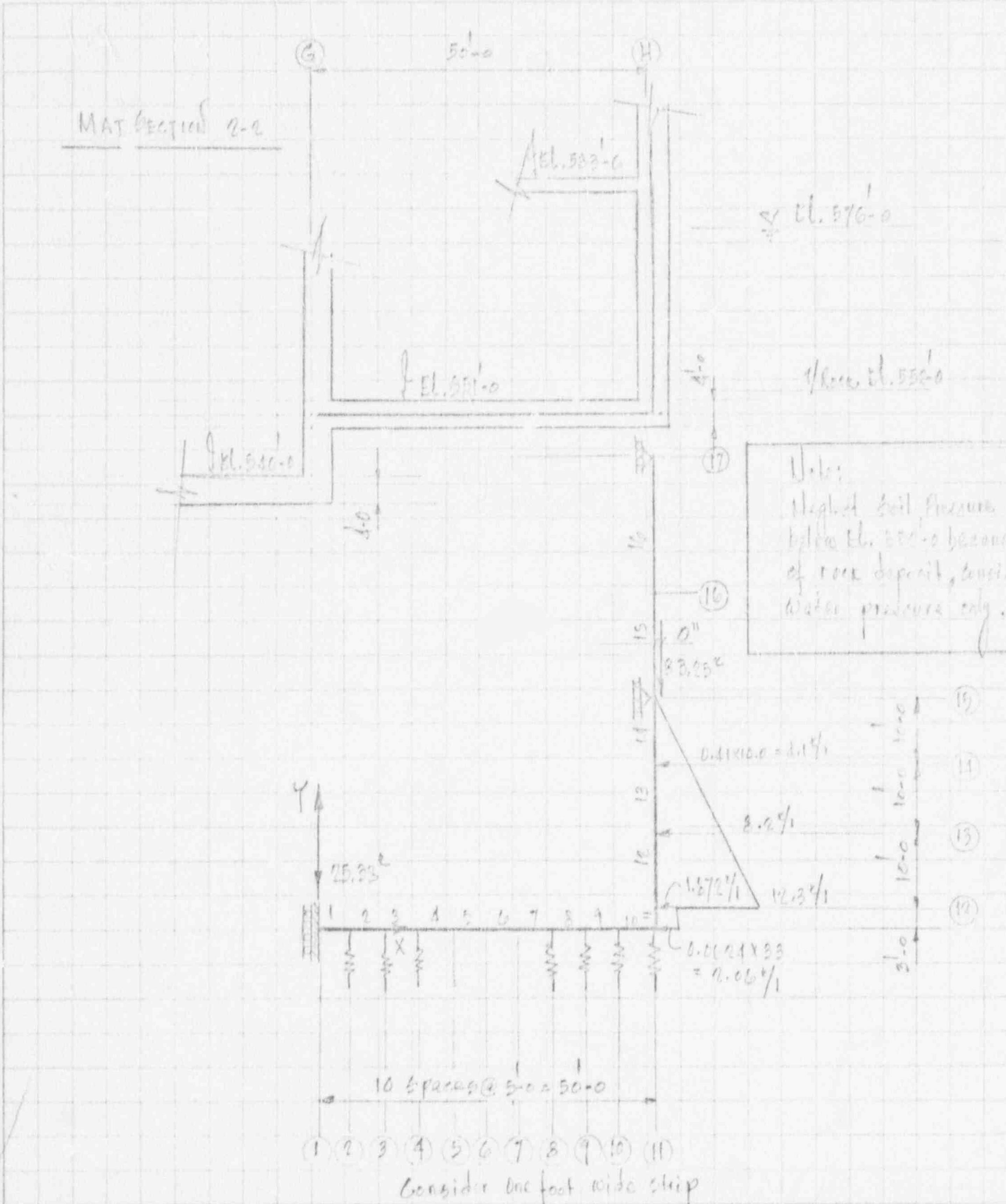
The bending moment diagrams for two critical loading combinations have been plotted on page 7. From the bending moment diagram it is found that the mat section is adequate for two critical loading combinations.

CHECKER'S REMARKS

RUN STATUS: ☐ PRELIMINARY ☑ ACTIVE ☐ VOID

Form GQ 3.08.1 Rev. 2

SARGENT & LUNDY ENGINEERS CHICAGO	Calcs. For Aux Bldg. Mat (Column Rows 11 & 17) Evaluation for (Design) Response Spectra	Calc. No. AT-81	
	✓ Safety-Related / Non-Safety-Related	Rev. 0	Date
		Page 9	of
Client: Detroit Edison Co.	Prepared by: S. Dutta	Date 8/17/81	
Project: Enrico Fermi-2	Reviewed by: J. Nardi	Date 5-16-81	
Proj. No. 4139-38 Equip. No.	Approved by	Date	



Form GO-3.08.1 Rev. 2

SARGENT & LUNDY ENGINEERS CHICAGO	Calcs. For Aux. Bldg. Mat (?) Evaluation For Revised Seismic Forces	Calc. No. AF-61
	✓ Safety-Related / Non-Safety-Related	Rev. 0 Date
		Page 10 of

Client Detroit Edison Co.	Prepared by S. Dutta	Date 5/?/81
Project Enrico Fermi-2	Reviewed by J. Nandi	Date 5-16-81
Proj. No. 6159-38 Equip. No.	Approved by	Date

'STRUDL'

Soil Modulus, k = 1000 k/ft³ (Same as Reactor Bldg. Mat)

$K_2, K_3, K_4, K_8, K_9, K_{10}$ = 1000 × 5.0 = 5000. k/ft²

K_{11} = 1000 × 4.0 = 4000. k/ft²

TYPE PLANE FRAME (XY plane)

E_c = 3620000 psi (for f'c = 4000 psi) = 521280 k/ft'

Support Joints : 1 thru 11, 15, 17

Joint Releases
- 2, 3, 4, 8, 9, 10, 11 KFY (as per k Value above)
- 1, 5, 6, 7, 15, 17 FORCE Y
- 2 thru' 11 Force X
- 2 thru' 11, 15, 17 Moment Z

Form GQ-3.08.1 Rev. 2

SARGENT & LUNDY ENGINEERS CHICAGO

Calcs. For Aux. Bldg. Mat (?) Evaluation For Revised Seismic Forces		Calc. No. AF-01	
✓ Safety-Related	Non-Safety-Related	Rev. 0	Date
		Page 11	of

Client: Detroit Edison Co.	Prepared by: S. Dutta	Date 5/14/81
Project: Enrico Fermi - 2	Reviewed by: J. Nandi	Date 5-16-81
Proj. No. 6189-38	Equip. No.	Approved by	Date

Loading :

1/ Dead Load :

St. 1 (Wall G) = 25.88 k/1 ↓ (Calc. No. 56-0001 pp. 11)

St. 15 (Wall H) = 83.25 k/1 ↓ (")

2/ Lateral Soil Pressure : (Refer FSAR)

Use 410 #/ft³ above T/Rock [≈ (96 + 300) or (122 + 280)]

62.4 #/ft³ below T/Rock [Neglect Soil Pressure below T/Rock El. because of Rock deposit]

3/ Hydrostatic Pressure :

a) Normal Water Level El. 576'-0

$(576.0 - 547.0) \times 0.0624 - 4 \times 0.15 = 1.201 \text{ k/}\square'$

b) Low Water Level El. 558'-8

$(558.67 - 547.0) \times 0.0624 - 4 \times 0.15 = 0.128 \text{ k/}\square'$

$(0.128 / 1.201 = 0.107$, ∴ Loading b) $= 0.107$ (Loading a)

Form GQ.3.08.1 Rev. 2

SARGENT & LUNDY ENGINEERS CHICAGO	Calcs. For Aux. Bldg. Mat (e) Evaluation for Revised Seismic Forces	Calc. No. AF-01	
	✓ Safety-Related / Non-Safety-Related	Rev. 0	Date
		Page 12	of
Client Detroit Edison Co.	Prepared by S. Dutta	Date 5/12/81	
Project Enrico Fermi-2	Reviewed by J. Nandi	Date 5-16-81	
Proj. No. 6139-38 Equip. No.	Approved by	Date	

Loading (Cont.)

4/ Seismic Load (SSE) $\sqrt{(X^2+Y^2+Z^2)}$

Combine the effect of three Components by 'SRSS' method

a) Wall G (middle) $= \sqrt{\left(\frac{M_{YY}}{I_{YY}} \cdot C_{XX}(t)\right)^2 + \left(\frac{M_{XX}}{I_{XX}} \cdot C_{Y(\cdot)}(t)\right)^2 + (0.25 \times D.L)^2}$

$= \sqrt{\left(\frac{3250733 \times 56.10 \times 1}{29310733}\right)^2 + 0 + (0.25 \times 25.38)^2}$

$= \sqrt{(24.93)^2 + (6.345)^2}$

$= \pm\ 25.725\ ^k/_{ft}$

b) Wall H (middle) $= \pm \sqrt{(\approx 0)^2 + (14.82)^2 + \left(\frac{0.25 \times 83.25}{3}\right)^2}$

See page 18

$= \pm\ 16.363\ ^k/_{ft}$

Load/ft $= 16.363 \times 3.0 = 49.090\ ^k/_{ft}$

(3.0 = wall thickness)

LOADING COMBINATIONS :

Loading Combination 5 Combine ① 1.0 ② 1.0 ③ 1.0 ④ 1.0

Loading Combination 6 Combine ① 1.0 ② 1.0 ③ 0.107

Loading Combination 7 Combine ① 1.0 ③ 1.0 ④ 1.0

Loading Combination 8 Combine ① 1.0 ③ 0.107

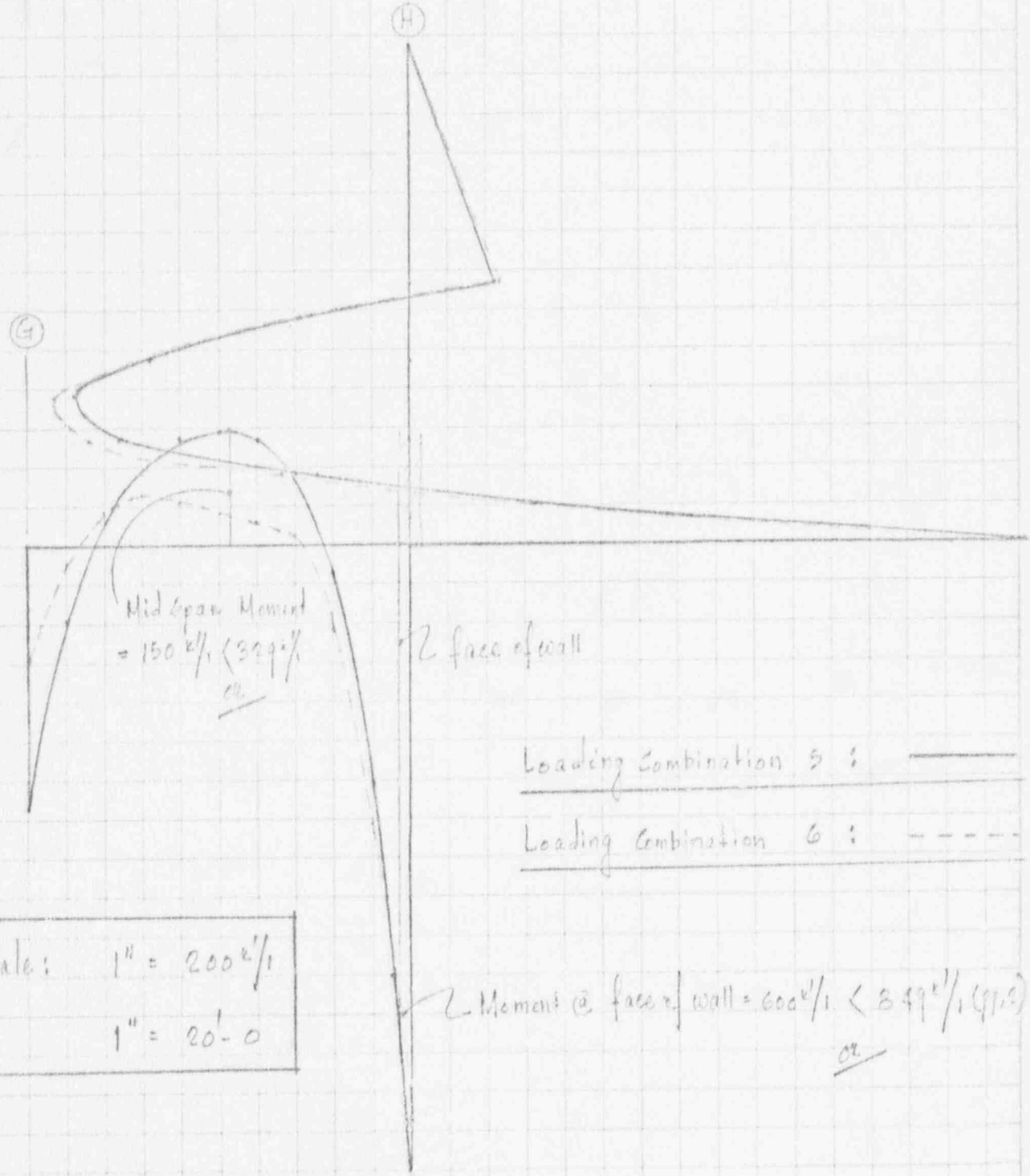
Form GQ-3.08.1 Rev. 2

SARGENT & LUNDY ENGINEERS CHICAGO

Calcs. For	Aux. Bldg. Mat (Sect. 2-2) - Review Evaluation for Revised Seismic Forces
✓ Safety-Related	Non-Safety-Related
Calc. No.	AF-01
Rev.	0
Date	
Page	18 of

Client	Detroit Edison Co.	Prepared by	S. Dutta	Date 5/15/81
Project	Enrico Fermi - 2	Reviewed by	J. Nandi	Date 5-16-81
Proj. No.	6139-38 Equip. No.	Approved by		Date

Ref.: 'STRUDL' Output A76250 dt. 5-14-81



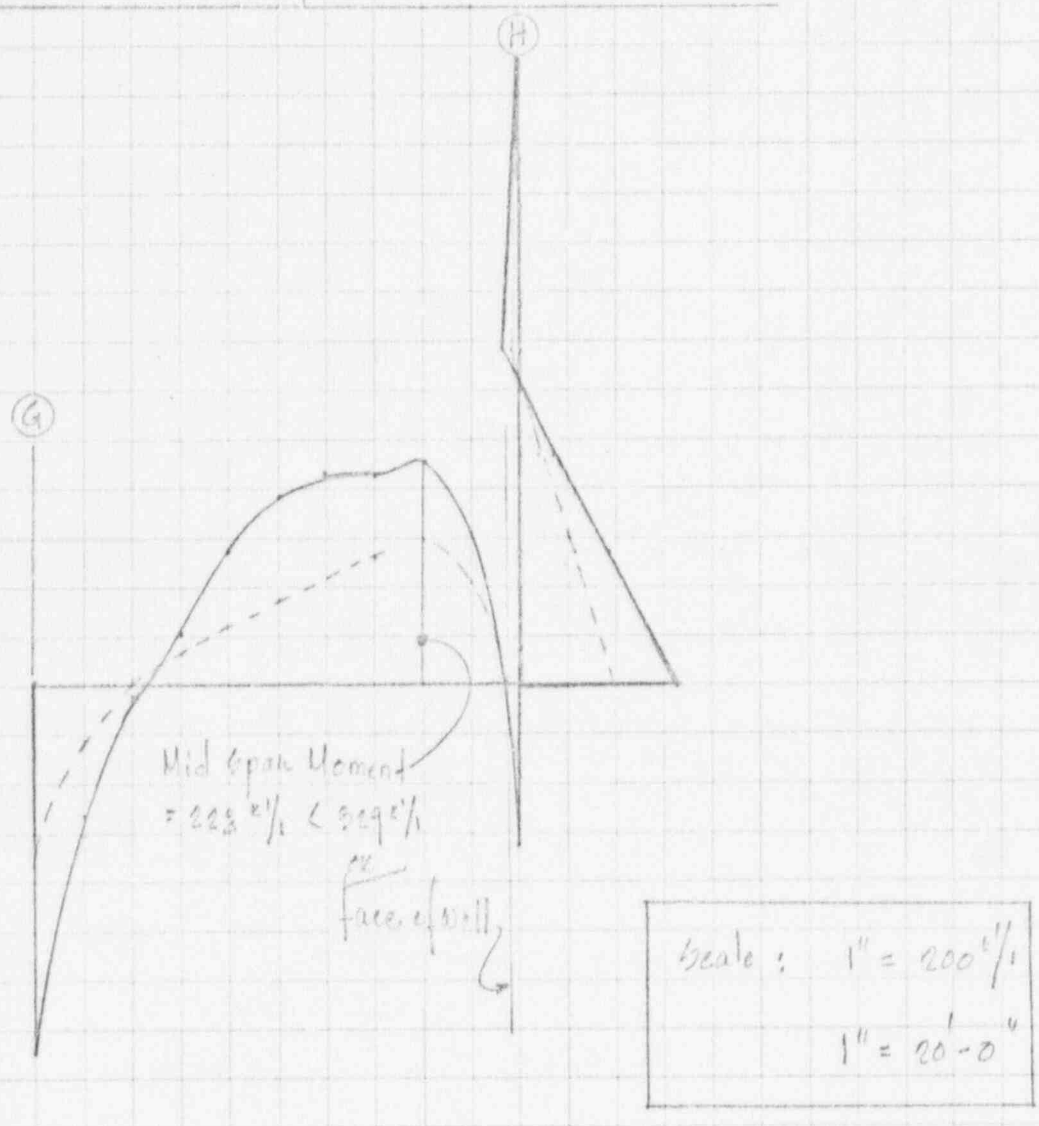
Bending Moment Diagram (k'/1)

Form GQ-3.08.1 Rev. 2

SARGENT & LUNDY ENGINEERS CHICAGO	Calcs. For Aux. Bldg. Mat (Sect 2.2) ~ Review Evaluation for Revised Seismic Forces		Calc. No. AF-01	
	✓ Safety-Related	Non-Safety-Related	Rev. 0	Date
			Page 16	of

Client	Detroit Edison Co.	Prepared by S. Dutta	Date 5/15/81
Project	Enrico Fermi-2	Reviewed by J. Nandi	Date 5-16-81
Proj. No. 6139-33	Equip. No.	Approved by	Date

Ref. : 'STRUDL' Output A76250 dt. 5-14-81



Bending Moment Diagram (k'/1)

Loading Combination 7 : ————

Loading Combination 8 : - - - -

SARGENT & LUNDY ENGINEERS CHICAGO	Calc. For Re-evaluation of Aux. Bldg. Mat COMPUTER RUN CONTROL EW ✓ Safety-Related / Non-Safety-Related	Calc. No. AF-01 Rev. 0 Date 5-16-81 Page 15 of 15

Client	DETROIT EDISON COMPANY	Prepared by S. Dube	Date 5-16-81
Project	FERMI 2	Reviewed by J. Nandi	Date 5-18-81
Proj. No. 6139-38	Equip. No.	Approved by	Date

RUN INFO	RUN ID	DATE	PROGRAM I.D.	CPU	SRM	NO. OF PAGES	NO. OF MICROFICHE
	A73850	5-14-81	SLS 09704312			67	1

REFERENCE	MICROFICHE I.D.	MODEL	DRAWINGS
	Aux. Bldg. Mat-2 AF-01	See page 9	B-28

PREPARER'S REMARKS (INCLUDING RUN DESCRIPTION)

One foot wide strip of mat & wall is used to model the area 2 of aux. bldg. mat. The loads applied and boundary conditions used in the program are explained in the calculations. Two different sets of loadings, with and without lateral soil pressures, have been used. This is because of the presence of Turbine Bldg. along wall line H.

The bending moment diagrams for four critical loading combinations have been plotted as shown on pages 13 & 14. From these moment diagrams it is found that the mat is adequate for the critical loading combinations.

CHECKER'S REMARKS

RUN STATUS: ☐ PRELIMINARY ☑ ACTIVE ☐ VOID

Form GQ-3.08.1 Rev. 2

SARGENT & LUNDY ENGINEERS

DESIGN CONTROL SUMMARY
DESIGN VERIFICATION

PAGE I

PROJECT NAME ENRICO FERMI
PROJECT NO. 6129-EA UNIT NO. 2
CLIENT DETROIT EDISON CO.
CALC. NO. & DESCRIPTION SC 0001
CALC. FOR RX/AUX. BLDG SHEAR WALLS - SEISMIC REEVALUATION

☒ SAFETY RELATED ☐ NON SAFETY RELATED

COMMENT NO.
QA SERIAL NUMBER

SIGNATURE & DATE FOR REV. 0		IDENTIFICATION OF PAGES PREPARED/REVISED/VOIDED & REVIEW METHOD
PREPARER	Signatures & dates appear on individual pages	pages 1 - 19 (Final)
REVIEWER	Signatures and dates appear on individual page	
APPROVER	[signature] 5-18-81	REVIEW METHOD Detail Review

SIGNATURE & DATE FOR REV __		IDENTIFICATION OF PAGES PREPARED/REVISED/VOIDED & REVIEW METHOD
PREPARER		
REVIEWER		
APPROVER		REVIEW METHOD

SIGNATURE & DATE FOR REV __		IDENTIFICATION OF PAGES PREPARED/REVISED/VOIDED & REVIEW METHOD
PREPARER		
REVIEWER		
APPROVER		REVIEW METHOD

SIGNATURE & DATE FOR REV __		IDENTIFICATION OF PAGES PREPARED/REVISED/VOIDED & REVIEW METHOD
PREPARER		
REVIEWER		
APPROVER		REVIEW METHOD

FORM SAS 22.1 Rev. 0

SARGENT & LUNDY
ENGINEERS

DESIGN CONTROL SUMMARY
DESIGN ELEMENT INDEX
PROJECT NO. 6157-38 UNIT NO. 2

PAGE ___ OF CALCULATIONS NO. SC 0001 REV. 0 DATE 5-15-1981

Revision

No.	Description	Pages
1.	General	1
2.	Design Moment Computation	2 thru' 9
3.	Dead Loads	10 & 11
4.	Evaluation @ El. 525'-6	12 thru' 15
5.	Evaluation @ El. 540'-0	16 thru' 19

FORM SAS 22.3 Rev. 0

SARGENT & LUNDY ENGINEERS

DESIGN CONTROL SUMMARY
ASSUMPTIONS AND PROCEDURES
PROJECT NO. 6131-32 UNIT NO. 2

PAGE ____ OF CALCULATIONS NO. CS-2001 REV. 0 DATE 5-15-81

Revision

These calculations include the design review of shear walls of Reactor/Auxiliary Building due to seismic forces induced by revised ground response spectra.

The shear forces in the walls are obtained from Tables 3 and 4 of Sargent and Lundy Calc. No. SDD-DECO-003

Two levels of shear walls eg. El. 582'-6" (just above grade level) and El. 540'-0" (just above the mat) are identified as the most critical walls for seismic re-evaluation.

The moments at various elevations are computed individually for each wall. Dead loads on the walls were assembled from the latest column and wall tabulations.

Shears were reviewed in each wall separately and the effect of the seismic moment in two direction was reviewed considering the shear wall grouping at an elevation acting as an overall box-type cross-section. Effect of vertical seismic forces was combined on SRSS basis.

CONCLUSION

Both at El. 583'-0" and El. 540'-0" the walls remain under compression all around under the dead load and seismic moments. The shear stresses in the concrete are fairly low; the concrete thicknesses and reinforcement provided are adequate to the seismic shears.

For subgrade walls (all under compression) the existing reinforcement will remain unaffected for lateral bending since the dynamic soil pressures on the wall remain same due to unchanged horizontal acceleration of $g = 0.15$.

FORM SAS-22.4 Rev. 0

SARGENT & LUNDY
ENGINEERS

DESIGN CONTROL SUMMARY
DESIGN INPUT DOCUMENTS

PROJECT NO. 6139.38 UNIT NO. 2

PAGE ___ OF CALCULATIONS NO. SC 0001 REV. 0 DATE 5-14-81

FORM SAS 22.7 Rev. 0

NO.	INPUT DOCUMENT: DESCRIPTION	INPUT DOCUMENT: SOURCE	INPUT DOCUMENT: IDENT	REVISION NUMBER OR DATE ON LINE INDICATED: DATE	S	DATE	S	DATE	S	DATE	S	USED IN DESIGN OF	CNT	C/N
1.	Seismic Re-analysis for 7% Damping Site Spectra Reactor Aux. Building	S&L	Report No. SAD-1500-003									Rx/Aux. Shear Walls		
2.	Seismic Analysis of the Reactor Aux. Building Complex	S&L	Report No. SL-2885	Sept. 27, 71								"		
3.	Building Code Requirements for Reinforced Concrete	ACI	ACI-318-77									"		
4.	Superstructure Wall Design Above Grade	S&L	Calc. No. 1.01.20									"		
5.	Substructure Column Loads	S&L	Calc. No. 3.01.00									"		

CNT - Continued CN - Comment Number S - Status

SARGENT & LUNDY ENGINEERS CHICAGO	Calcs. For Reactor/Aux. Bldg. Shear Walls - Re-evaluation for Revised Seismic R.S.		Calc. No. [illegible]
	Safety-Related	Non-Safety-Related	Rev. Date
			Page 1 of 11

Client Detroit Edison Co.	Prepared by N. V. Hrisa[illegible]	Date 1-1-81
Project Enrico Fermi - 2	Reviewed by [illegible]	Date 5-28-81
Proj. No. 6139-38 Equip. No.	Approved by	Date

GENERAL

Re-evaluation of shear walls for Reactor/Aux Bldg is generally based on the design methods used for initial calculations.

The seismic shear in the walls due to combined excitation is taken by concrete and steel in these walls at various elevations. For re-evaluation we have selected El. 583'-6" to be the most critical level for shear forces and overall seismic moments. Subgrade walls at El. 540'-0" have also been reviewed, this being the lowest shear walls in the structure.

The structure was modelled as shearwall-diaphragm system, assuming the seismic forces predominantly resisted by stiff shear-walls. For static equilibrium, we have computed the moments due to seismic shears and reviewed the shearwall grouping as an overall box-type cross-section.

CONCLUSION

The review shows that there is no tension at all in the concrete for the seismic moment effect. The stresses in the wall reinforcement are fairly small assuming concrete shear stress $V_c = 2\sqrt{f_c'}$. These are very conservative values (Refer ACI 318-77 and 11.10.6 which allows $V_c = 3.3\sqrt{f_c'}\, h d + \frac{N_u d}{4 l_w}$ for walls with axial load).

Form GQ.3.08.1 Rev. 2

SARGENT & LUNDY ENGINEERS CHICAGO	Calcs. For Rx/Aux Bldg ~ Shear Walls Review for Revised Response Spectra	Calc. No.
	X Safety-Related / Non-Safety-Related	Rev. / Date
		Page 2 of

Client Detroit Edison Co.	Prepared by	Date 4-25-81
Project Enrico Fermi - 2	Reviewed by	Date 4-29-81
Proj. No. 6129-38 Equip. No.	Approved by	Date

Design Moments for Shear Walls

SD-Q-001 Fermi-2 SSE 7% DAMP computer run on 4-16-81 is a time-history run primarily used for generation of Seismic Response Spectra at various elevations for equipment and system design.

This computer run calculates shear forces in various springs but does not give any equilibrium moments in shear walls. Also the shear force in the walls is given for combined X & Y excitation and separate information for each excitation is not available.

Following procedure has been adopted to determine the design moments in the shear wall design

(Ref. Report No. SDP-DECO-003)

a) Assemble the X & Y springs and add the shear values of springs forming a continuous wall - see exhibits 10 to 15 of S&L Report -2682 of 9-27-79

b) For each column line assemble the wall/shears at each elevation forming a single continuous wall group

c) Find the cumulative moments in the shear walls by multiplying shears by the wall heights and then successively adding these values - See calc. sheets nos. to .

d) Since these moments are based on maximum shears in X and Y springs respectively, their effect on structure is combined by SRSS method. For rationale of this method following explanation is offered.

Refer page 18 of the above referred Computer output.

Story Moments for slab # 1 are: @ t = 2.55 $M^*_{yy} = 2757 \times 10^3$; $M_{xx} = 1912 \times 10^3$

@ t = 2.44 $M_{yy} = 1225 \times 10^3$; $M^*_{xx} = 2589 \times 10^3$

Combined effect @ t = 2.55 = 4169×10^3

@ t = 2.48 = 3814×10^3

If only M^*_{yy} & M^*_{xx} were available combined effect could be determined by SRSS method i.e. $\left[(2757 \times 10^3)^2 + (2589 \times 10^3)^2\right]^{1/2} = 3782 \times 10^3$ which is not much different from the two values given above

Form GO-3.08.1 Rev. 2

SARGENT & LUNDY ENGINEERS CHICAGO	Calcs. For [illegible] Shear Walls Design Review for Revised Seismic [illegible]	Calc. No. [illegible]
	✓ Safety-Related / Non-Safety-Related	Rev. / Date / Page 3 of

Client Detroit Edison Co.	Prepared by [illegible]	Date 4-21-81
Project Enrico Fermi-2	Reviewed by [illegible]	Date 5-5-81
Proj. No. 6139-33 Equip. No.	Approved by	Date

SHEAR WALL ALONG LINE 9 (A to H).

SLAB NO	ELEVATION	SHEAR WALLS	HC	SHEAR FORCE	TOTAL SHEAR FORCE	MOMENT	TOTAL MOMENT
5	684'-6"	050601	51.0	362 ✓	840 ✓	24676	24676
		050901	13.0	478 ✓			
4	659'-6"	040501	25.0	4995 ✓	5517 ✓	137925	162601
		040503	25.0	522 ✓			
3	641'-6"	030401	18.0	6976 ✓	7623 ✓	137214	299815
		030403	18.0	647 ✓			
2	613'-6"	020301	28.0	7149 ✓			
		020303	28.0	918 ✓	9721 ✓	272188 ✓	572003
		020305	28.0	634 ✓			
		020307	28.0	1020 ✓			
1	583'-6"	010201	30.0	15 ✓			
		010203	30.0	2688 ✓			
		010205	30.0	6071 ✓	10782 ✓	323460 ✓	895463 ✓
		010207	30.0	753 ✓			
		010209	30.0	1255 ✓			
0	540'-0"	000101	43.5	8176 ✓	10085	438698	1334161 ✓
		000103	43.5	1909 ✓			

X - DIRECTION

Slab 6 : El. 735'-6

Slab 9 : El. 697'-6

Wall 17 Wall 9

Total Shear @ El. 583'-6 = 14542 + 10782 = 25324 ✓

Total Moment @ El. 583'-6 = 1144365 + 895463 = 2039828 ✓

$\bar{h}_1 = 2039828 / 25324 = 80.55'$ ✓

SARGENT & LUNDY ENGINEERS CHICAGO	Calcs. For Rx/Aux Bldg. - Shear Walls [illegible]	Calc. No. [illegible]
	✓ Safety-Related / Non-Safety-Related	Rev. / Date
		Page 1 of

Client Detroit Edison Co	Prepared by [illegible] Date 4-21-81
Project Enrico Fermi - 2	Reviewed by [illegible] Date 5-5-81
Proj. No. [illegible] Equip. No.	Approved by Date

SHEAR WALL ALONG LINE 17 (A to H)

SLAB NO.	ELEVATION	SHEAR WALLS	Ht	SHEAR FORCE	Total	Moment	Total Moment
5	684'-6"	050603	51.0	358 ✓	2095 ✓	40839 ✓	40839
		050911	18.0	1737 ✓			
4	659'-6"	040509	25.0	1456 ✓	4040	101000 ✓	141839
		040511	25.0	2059 ✓			
		040513	25.0	525 ✓			
3	641'-6"	030409	18.0	1140 ✓	6998	171911 ✓	313750 ✓
		030411	18.0	939 ✓			
		030413	18.0	2551 ✓			
		030415	18.0	520 ✓			
		030501	43.0	1848 ✓			
2	613'-6"	020313	28.0	1331 ✓	14184 ✓	394352 ✓	708105 ✓
		020315	28.0	2083 ✓			
		020317	28.0	2416 ✓			
		020319	28.0	8354 ✓			
1	583'-6"	010211	30.0	8673 ✓	14542 ✓	436260 ✓	1144365 ✓
		010213	36.0	3262 ✓			
		010215	36.0	1378 ✓			
		010217	36.0	1229 ✓			
0	540'-0"	000113	43.5	2402	11447	497945	1642310
		000115	43.5	9045			

SLAB 6 : EL. 735'-6"
SLAB 9 : EL. 697'-6"

Form GQ 3.08.1 Rev. 2

SARGENT & LUNDY ENGINEERS CHICAGO

Calcs. For	[illegible] Bldg. Shear [illegible] Design Rec. for Revised [illegible] R-S.
✓ Safety-Related	Non-Safety-Related
Calc. No	[illegible]
Rev.	Date
Page 5	of
Client	Detroit Edison Co.
Project	Enrico Fermi-2
Proj. No.	6139-23
Equip. No.	
Prepared by	N. V. [illegible]
Date	4-21-81
Reviewed by	[illegible]
Date	5-6-81
Approved by	
Date	

SHEAR ALONG LINE A (9-17)

SLAB	EL.	Shear wall	Ht	Shear Force	Moment	Accumul. Moment
5	684'-6"	050602	31.0	388	19635	19635
4	659'-6"	040502	25.0	778	19450	39085
3	641'-0"	030402	18.0	1969	35442	74527
2	613'-6"	020302	28.0	5288	148064	222591
1	583'-6"	010202	30.0	6747	202410	425001
0	540'-6"	000102	43.0	7525	323575	748576

Total Moment in Y direction @ 583'-6"

Wall	
A	425001
F	605704
G	331,500
H	524497
	1886,752 k-ft.

Slab 6 : El. 735'-6

Form GQ-3.08.1 Rev. 2

SARGENT & LUNDY ENGINEERS CHICAGO	Calcs. For Rx Aux Bldg – Shear walls [illegible] – Loads	Calc. No. DC-1001
	✓ Safety-Related / Non-Safety-Related	Rev. / Date / Page 6 of

Client Detroit Edison Co.	Prepared by V. [illegible]	Date [illegible]-21-81
Project Enrico Fermi-2	Reviewed by [illegible]	Date 5-6-81
Proj. No. 6[illegible]-3B Equip. No.	Approved by	Date

SHEAR WALL ALONG LINE F (9 to 17)

SLAB	EL.	Shearwall	Ht	Shear Force	Total Shear	Moment	Total Moment
5	684'-6"	050604	51.0	414	1968	41316	41316
		050802	13.0	834			
		050904	13.0	720			
4	659'-6"	040504	25.0	992	4374	109350	150666
		040506	25.0	1980			
		040508	25.0	1402			
3	641'-6"	030404	18.0	2566	6033	108594	259260
		030406	18.0	3467			
2	613'-6"	020304	28.0	896	12373	346444	605704
		020306	28.0	9921			
		020314	28.0	1556			

SHEAR WALL ALONG G (9 to 17)

SLAB	EL.	Shearwall	Ht	Shear Force	Total Shear	Moment	Total Moment
1	583'-6"	010204		317	13085	392550	392550
		010214	30.0	3692			
		010206		488			
		010216		8588			
0	540'-0"	000112	43.5	1203	8610	374535	767085
		000114		7407			

Form GQ 3.08.1 Rev. 2

SARGENT & LUNDY ENGINEERS CHICAGO	Calcs. For [illegible] Bldg. - Shear Wall Design [illegible] for Revised Seismic [illegible]	Calc. No. [illegible]
	✓ Safety-Related / Non-Safety-Related	Rev. / Date
		Page 7 of

Client [illegible] Edison Co.	Prepared by N. B. [illegible]	Date 4-01-81
Project [illegible] Fermi - 2	Reviewed by [illegible]	Date 5-6-81
Proj. No. 6139-58 Equip. No.	Approved by	Date

SHEAR WALL ALONG LINE H (9-17)

Slab	EL.	Shear Wall	Ht.	Shear force	Total Shear	Moment	Tot Moment
5	684'-6"	050906	13.0	1762	2185	28405	28405
		050908	13.0	423			
4	653'-6"	040510	25.0	2518	3858	96450	124855
		040512	25.0	1340			
3	641'-6"	030408	18.0	3260	5136	97848	222703
		030410	18.0	1340			
		030412	18.0	526			
2	613'-6"	020310	28.0	4023	6828	191184	413887
		020312	28.0	2805			
1	583'-6"	010208	30.0	1801	4987	149610	563497
		010210	30.0	1119			
		010212	30.0	2067			
0	550'-0"	000116	43.5	2497	9279	403637	967134
		000118	43.5	5266			
		000120	43.5	1516			

Form GQ-3.08.1 Rev. 2

SARGENT & LUNDY ENGINEERS CHICAGO	Calcs. For [illegible]	Calc. No. [illegible]
	✓ Safety-Related / Non-Safety-Related	Rev. / Date
		Page 9 of

Client [illegible]	Prepared by N. V. [illegible]	Date [illegible]
Project [illegible]	Reviewed by B. Dette	Date 5-5-81
Proj. No. 6180-58 Equip. No.	Approved by	Date

Diagonal Shear Walls Below 586-0 [illegible]

			Shear	Moment
South West Corner (C to 11)	X-Spring	000117	1555	67642 ✓
	Y-Spring	000109	1262	59247
North West Corner (13 to C)	X-Spring	000109	[illegible]	72514
	Y-Spring	000106	1262	59247
North East Corner (F to 13)	X-Spring	000111	1667	72514 ✓
	Y-Spring	000110	1603 ✓	69730 ✓
South East Corner (11 to F)	X-Spring	000117	1555	67642
	Y-Spring	000108	1603	69730

Max. Shear in Diagonal wall $= \sqrt{2}\,(1667+1603) = 4624^{k}$

Σ(X spring Moment) $= 2(67642+72514) = 280312^{k'}$

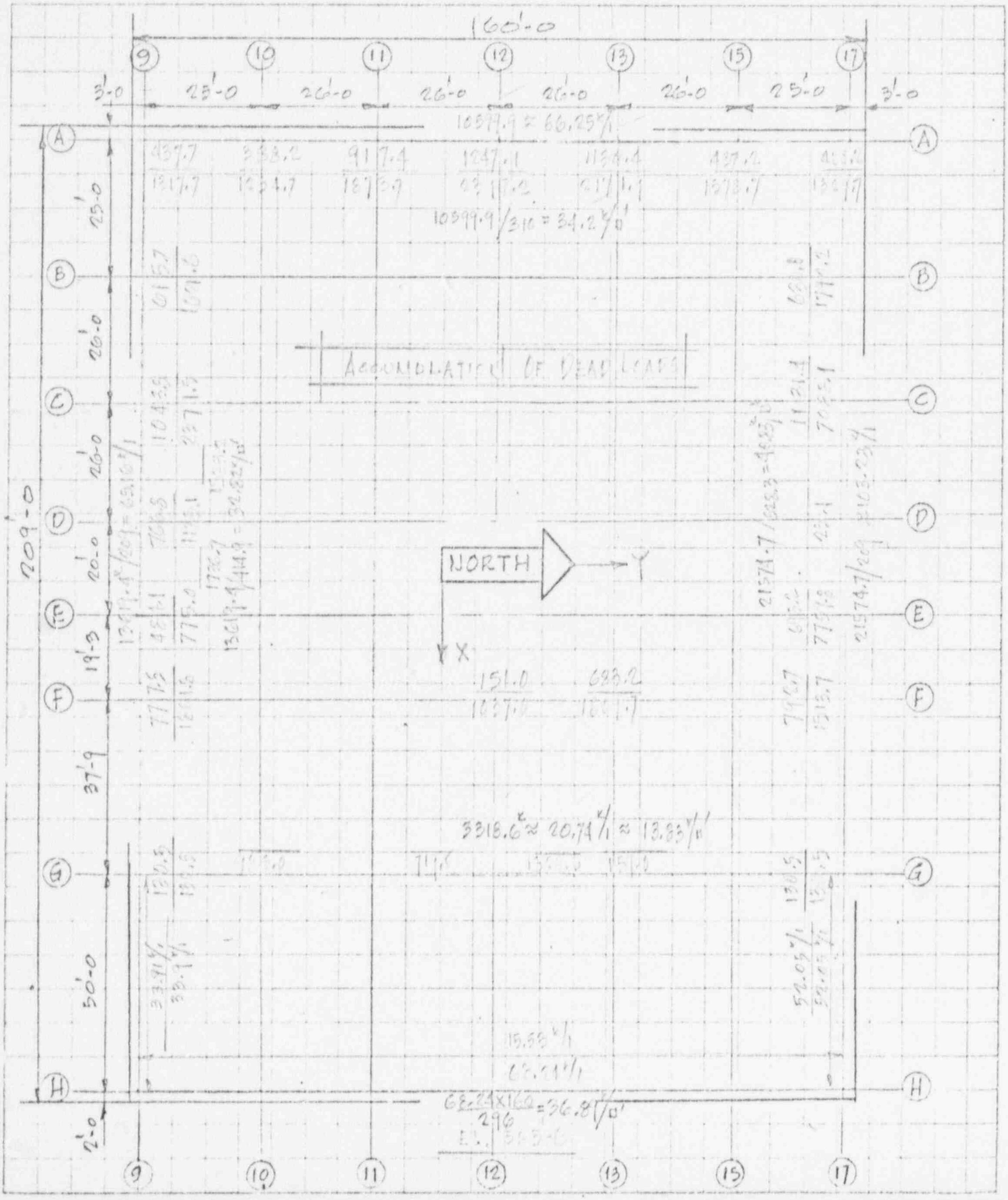
Σ(Y spring Moment) $= 2(59247+69730) = 257954^{k'}$ ✓

Form GQ-3.08.1 Rev. 2

SARGENT & LUNDY
ENGINEERS
CHICAGO

Calcs. For Rx/Aux. Bldg. ~ Shear Walls @ El. Evaluation For Revised Seismic Forces		Calc. No.
✓ Safety-Related	Non-Safety-Related	Rev. / Date
		Page 10 of

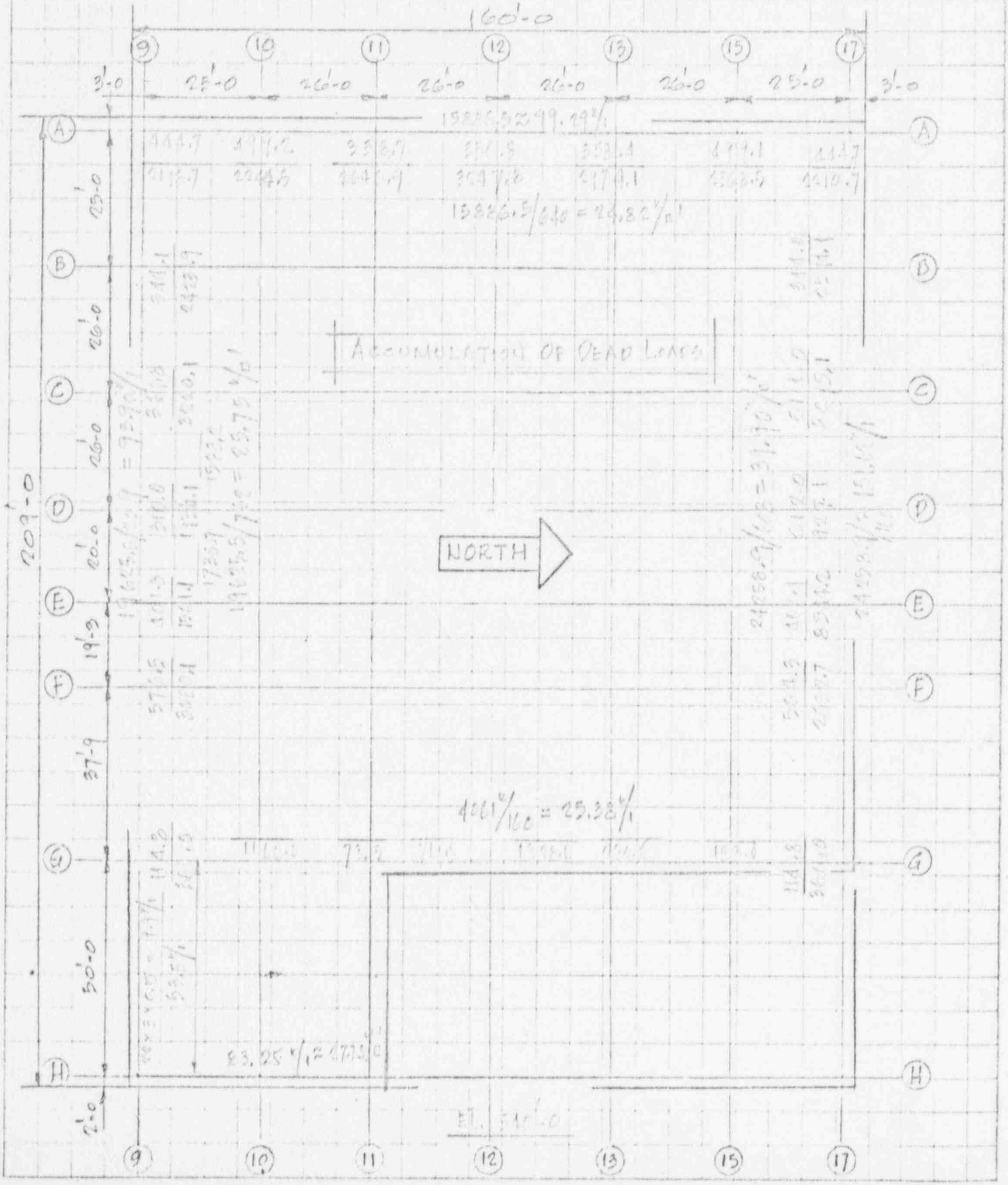
Client: Detroit Edison Co.		Prepared by: S. Dutta	Date
Project: Enrico Fermi - 2		Reviewed by:	Date 5-6-81
Proj. No. 6139-38	Equip. No.	Approved by	Date



Form GQ-3.08.1 Rev. 2

SARGENT & LUNDY ENGINEERS CHICAGO	Calcs. For Rx/Aux. Bldg. – Shear Walls @ El. Evaluation for Revised Seismic Forces	Calc No.	Rev.	Date	Page	of
	✓ Safety-Related / Non-Safety-Related					

Client: Detroit Edison Co.	Prepared by: S. Dutta	Date	
Project: Enrico Fermi - 2	Reviewed by: [illegible]	Date	
Proj. No. 6139-32	Equip. No.	Approved by	Date



SARGENT & LUNDY ENGINEERS CHICAGO

Calcs. For Rx/Aux. Bldg. Shear walls — Evaluation for Revised Seismic Forces

Calc. No. [illegible] Rev. Date

✓ Safety-Related Non-Safety-Related Page [illegible] of

Client: Detroit Edison Co.

Project: Enrico Fermi - 2

Proj. No. 6139-38 Equip. No.

Prepared by [illegible] Date 5-5-81

Reviewed by [illegible] Date 5-6-81

Approved by Date

Evaluation of Shear Walls (overall basis):

I_{xx} (shear wall group) = 8017200 ft^4, I_{yy} (shear wall group) = 9519400 ft^4

(Ref. Calc. Book No. 4.01.06 page 13, 14)

SECTION MODULUS	A-9	A-17	H-9	H-17	G-9	G-17
S_{xx} ft^3	8017200/84.5 = 94878.	8017200/75.5 = 106188.	8017200/84.5 = 94878.	8017200/75.5 = 106188.	8017200/84.5 = 94878.	8017200/75.5 = 106188.
S_{yy} ft^3	9519400/115.0 = 82777.	9519400/115.0 = 82777.	9519400/94.0 = 101270.	9519400/94.0 = 101270.	9519400/45.0 = 211542.	9519400/45.0 = 211542.

MOMENT k'	A-9 M/S k/□'	A-17 M/S k/□'	H-9 M/S k/□'	H-17 M/S k/□'	G-9 M/S k/□'	G-17 M/S k/□'
ΣM_{xx} (Wall A, F, G, H) = 425001 + 392580 + 568497 + 600704 = 1986792	1986792 / 94878 = 20.94	1986792 / 106188 = 18.71	1986792 / 94878 = 20.94	1986792 / 106188 = 18.71	1986792 / 94878 = 20.94	1986792 / 106188 = 18.71
ΣM_{yy} (Wall 9, 17) = 1144365 + 895463 = 2039828	2039828 / 82777 = 24.67	2039828 / 82777 = 24.67	2039828 / 101270 = 20.14	2039828 / 101270 = 20.14	2039828 / 211542 = 9.64	2039828 / 211542 = 9.64

VERTICAL EXITATION	A-9	A-17	H-9	H-17	G-9	G-17
Vertical accln. = 0.25g; Vertical Seismic Load = 0.25 (D.L.)	0.25 (32.8+34.2)/2 = 0.25 x 33.51 = 8.38 k/□'	0.25 (34.2+40.8)/2 = 0.25 x 37.51 = 9.38 k/□'	0.25 (32.8+30.89)/2 = 0.25 x 34.86 = 8.71 k/□'	0.25 (36.89+40.8)/2 = 0.25 x 38.86 = 9.72 k/□'	0.25 x 32.83 = 8.21 k/□'	0.25 x 40.83 = 10.21 k/□'

Form GC-3.08.1 Rev. 2

SARGENT & LUNDY ENGINEERS CHICAGO	Calcs. For Rx/Aux Bldg. – Shear walls @ El. 583-6 Evaluation for Revised Seismic Forces	Calc. No.
	✓ Safety–Related / Non–Safety–Related	Rev. / Date
		Page 13 of

Client Detroit Edison Co.		Prepared by	Date 5/5/81
Project Enrico Fermi-2		Reviewed by	Date 5-6-81
Proj. No. 6139-33	Equip. No.	Approved by	Date

A-9 : $33.51 \pm \sqrt{(20.94)^2+(24.67)^2+(8.58)^2} = 66.98\ ^k/_{'}\ ,\ 0.09\ ^k/_{'}$
(465.1 psi comp.) , (0.6 psi comp.)

A-17 : $37.51 \pm \sqrt{(18.71)^2+(24.64)^2+(9.38)^2} = 69.84\ ^k/_{'}\ ,\ 5.18\ ^k/_{'}$
(485.0 psi comp.) , (36.0 psi comp.)

H-9 : $34.86 \pm \sqrt{(20.94)^2+(20.14)^2+(8.71)^2} = 65.19\ ^k/_{'}\ ,\ 4.53\ ^k/_{'}$
(452.7 psi comp.) , (31.5 psi comp.)

H-17 : $38.86 \pm \sqrt{(18.71)^2+(20.14)^2+(9.72)^2} = 68.02\ ^k/_{'}\ ,\ 9.70\ ^k/_{'}$
(472.4 psi comp.) , (67.4 psi comp.)

G-9 : $32.82 \pm \sqrt{(20.94)^2+(9.64)^2+(8.01)^2} = 57.29\ ^k/_{'}\ ,\ 8.35\ ^k/_{'}$
(397.8 psi comp.) , (58.0 psi comp.)

G-17 : $40.83 \pm \sqrt{(18.71)^2+(9.64)^2+(10.21)^2} = 64.22\ ^k/_{'}\ ,\ 17.44\ ^k/_{'}$
(446.0 psi comp.) , (121.1 psi comp.)

Wall A (Center) : $34.2 \pm \sqrt{(24.67)^2+(0.25\times34.2)^2} = 60.31\ ^k/_{'}\ ,\ 8.09\ ^k/_{'}$
(418.8 psi comp.) , (56.2 psi comp.)

Wall G (Center) : $13.83 \pm \sqrt{(9.64)^2+(0.25\times13.83)^2} = 24.07\ ^k/_{'}\ ,\ 3.59\ ^k/_{'}$
(167.2 psi comp.) , (24.9 psi comp.)

Wall H (Center) : $36.89 \pm \sqrt{(20.14)^2+(0.25\times36.89)^2} = 59.04\ ^k/_{'}\ ,\ 14.74\ ^k/_{'}$
(410.0 psi comp.) , (102.4 psi comp.)

Wall 9 (Center) : $32.82 \pm \sqrt{(20.94)^2+(0.25\times32.82)^2} = 55.31\ ^k/_{'}\ ,\ 10.33\ ^k/_{'}$
(384.1 psi comp.) , (71.7 psi comp.)

Wall 17 (Center) : $40.83 \pm \sqrt{(18.71)^2+(0.25\times40.83)^2} = 62.14\ ^k/_{'}\ ,\ 19.52\ ^k/_{'}$
(431.5 psi comp.) , (135.6 psi comp.)

NOTE : Max^m moments in each wall due to Combined X & Y excitations have been used, Since these do not occur at the same time their effect and the vertical seismic effect have been combined by 'SRSS' method and then algebraically added to dead load

Form GQ-3.08.1 Rev. 2

SARGENT & LUNDY
ENGINEERS
CHICAGO

Calcs. For	Rx/Anx Bldg. ~ Shear walls @ El. 583-6 Review for Revised Seismic Forces
✓ Safety–Related	Non–Safety–Related
Calc. No.	[illegible]
Rev.	Date
Page 11	of

Client	Detroit Edison Co.	Prepared by	S. Datta	Date 5/5/81
Project	Enrico Fermi-2	Reviewed by	[illegible]	Date 5-5-81
Proj. No.	4137-35 Equip. No.	Approved by		Date

Evaluation Of Shear Walls (overall basis)

Shear Stress (Diagonal Tension Method)

Wall	Area, A (ft^2)	Shear Force, V (k)	Shear Stress, v_u $v_u = \frac{V}{A}$	Wall thickness, t, in	Steel/ft A_s $in^2/1$	Steel Stress, fy $= \frac{(v_u - \phi v_c)(t \times 12)}{(A_s) 1000}$ ksi	Remarks
9	414.9 ✓	10782 ✓	25.99 $k/'$ = 180.5 psi	18	#8@12 (each face) = 1.17 $in^2/1$	13.47 ✓	OK
17	528.3 ✓	14542 ✓	27.53 $k/'$ = 191.2 psi	18	1.17 $in^2/1$	15.45 ✓	OK
A	310.0 ✓	6747 ✓	21.76 $k/'$ = 151.1 psi	18	1.17 $in^2/1$	8.04 ✓	OK
G	471.5	13085 ✓	27.75 $k/'$ = 192.7 psi	18	1.17 $in^2/1$	15.72 ✓	OK
H	296.0 ✓	4987 ✓	16.85 $k/'$ = 117.0 psi	24	1.17 $in^2/1$	2.33	OK

$$\phi = 0.85 \ , \quad v_c = 2\sqrt{f_c'} = 2\sqrt{4000} = 126.5 \text{ psi}$$

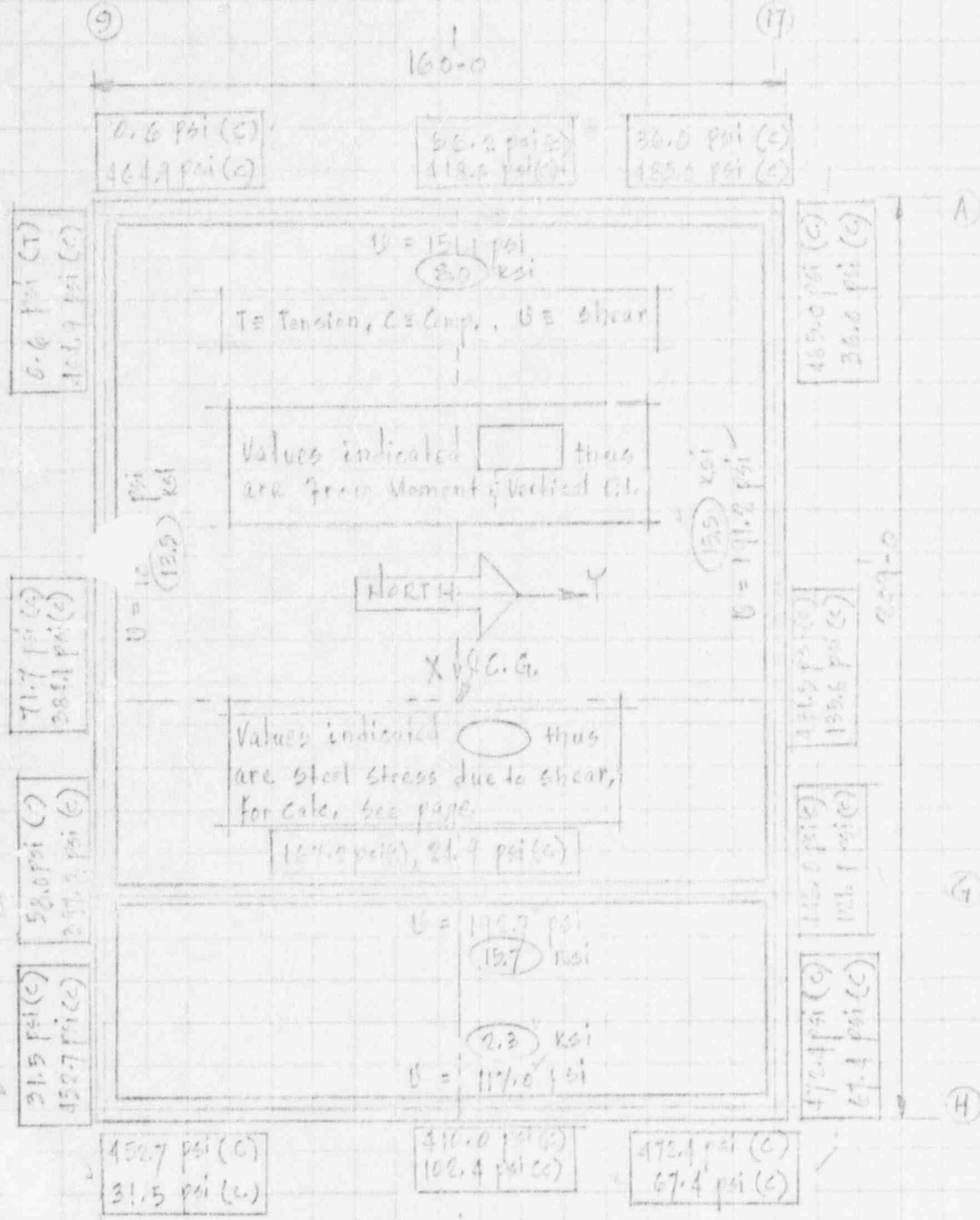
Note: This value of $v_c = 2\sqrt{f_c'}\, hd$ is rather Conservative. According to ACI Code 318-77 § 11.10.6, $V_c = 3.3\sqrt{f_c'}\, hd + \frac{N_u d}{4 l_w}$ may be used.

Form GO-3.08.1 Rev. 2

SARGENT & LUNDY ENGINEERS CHICAGO

Calcs. For Rx/Aux Bldg Shear Walls @ El. 583-6 Evaluation For Revised Seismic Forces		Calc. No.	
✓ Safety-Related	Non-Safety-Related	Rev.	Date
		Page 15	of

Client: Detroit Edison Co.		Prepared by: S. Dutta	Date: 5/5/81
Project: Enrico Fermi-2		Reviewed by: [illegible]	Date: 5-8-81
Proj. No.: 6139-38	Equip. No.	Approved by	Date



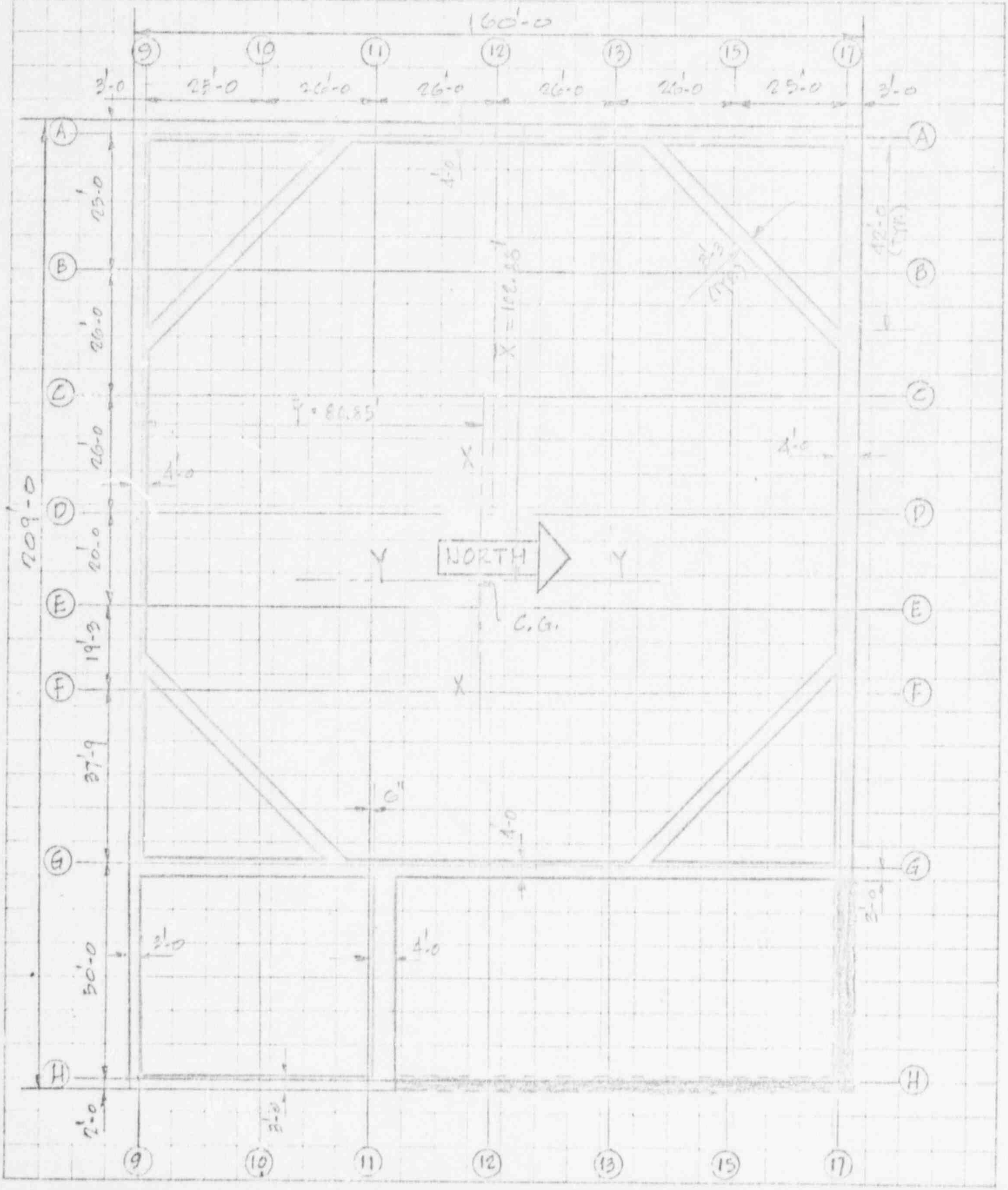
Stress levels on Shear Walls (overall Basis)

EL. 583'-6

Form GQ.3.08.1 Rev. 2

SARGENT & LUNDY ENGINEERS CHICAGO	Calcs. For Rx/Aux. Bldg. ~ Shear Walls @ El. 540'-0 Evaluation for Revised Seismic Forces ☑ Safety–Related ☐ Non–Safety–Related	Calc. No. Rev. Date Page of

Client	Detroit Edison Co.	Prepared by	D. Dutta	Date
Project	Enrico Fermi - 2	Reviewed by		Date
Proj. No.	6139-38 Equip. No.	Approved by		Date



Form GQ-3.08.1 Rev. 2

SARGENT & LUNDY
ENGINEERS
CHICAGO

Calcs. For Ex/Aux. Bldg. ~ Shear walls @ EL 5[illegible] Evaluation for Revised Seismic Forces		Calc. No. [illegible]	
✓ Safety-Related	Non-Safety-Related	Rev.	Date
		Page 17	of

Client Detroit Edison co.		Prepared by [illegible]	Date [illegible]
Project Enrico Fermi - 2		Reviewed by [illegible]	Date 5-[illegible]
Proj. No. 6139-33	Equip. No.	Approved by	Date

Calculation of C.G. of wall group

Exterior Corner of A-9 (0,0)

WALL	AREA A, ft²	AX ft³	AY ft³	MOMENT OF INERTIA $I_{xx} + AR_y^2$ ft⁴	MOMENT OF INERTIA $I_{yy} + AC_x^2$ ft⁴
A	160×4 = 640	640×2 = 1280 ✓	640×80 = 51200 ✓	$\frac{1}{12}\times 4\times (160)^3 + 640\times 0.85^2$ ✓ = 1365796	$\frac{1}{12}\times (4)^3 \times 160 + 640(100.88)^2$ = 6513981
G	160×4 = 640	640×158 ✓ = 101120 ✓	640×80 ✓ = 51200 ✓	$\frac{1}{12}\times 4\times (160)^3 + 640(0.85)^2$ = 1365796	$\frac{1}{12}\times 160\times 4^3 + 640(55.12)^2$ = 6513981
H	160×3 = 480	480×207.5 = 99600	480×80 = 38400	$\frac{1}{12}\times 3\times 160^3 + 480(0.85)^2$ = 1004347	$\frac{1}{12}\times 160\times 3^3 + 480(104.62)^2$ = 5254125
9	46×3 + 156×4 = 138 + 624 = 762	138×133 + 624×78 = 74550	138×1.5 + 624×2 = 1455 ✓	$\frac{1}{12}\times 46\times (3)^3 + 138(79.35)^2 + \frac{1}{12}\times 156(4)^3 + 624(78.85)^2$ = 4719451	$\frac{1}{12}\times 3\times (46)^3 + 138(30.12)^2 + \frac{1}{12}\times 4\times (156)^3 + 624(22.88)^2$ = 2502218
17	202×4 = 808	808×104.5 = 84436 ✓	808×158 = 127664 ✓	$\frac{1}{12}\times 202\times (4)^3 + 808(77.15)^2$ = 4810812	$\frac{1}{12}\times 4\times 202^3 + 808(1.62)^2$ = 2741590
Diagonals U	4(61.87×3.25) = 4×201 = 804	2×201×25 + 2×201×135 = 64320 ✓	2×201×45 + 2×201×115 = 64320 ✓	$2[(\frac{1}{12}\times 42\times (3.25)^2)2 + 201(35.35^2+34.15^2)] + 4[(\frac{1}{12}\times 3.25\times 42^3)2 + 201(35.35^2+34.15^2)]$ = 4946104	$2[(\frac{1}{12}\times 42(3.25)^3)2 + 201(77.83^2+32.1^2)] + 2[(\frac{1}{12}\times 3.25\times (42)^3)2 + 201(77.88^2+32.12^2)]$ = 5786722
	4134 ✓	425306	334239	18221886	29320733

$\bar{X} = 425306/4134 = 102.88'$; $\bar{Y} = 334239/4134 = 80.85'$ ✓

Form GO-3.08.1 Rev. 2

SARGENT & LUNDY ENGINEERS CHICAGO	Calcs. For Rx/Aux. Bldg Shear Walls at EL. 5[illegible] Evaluation for Critical Seismic Forces	Calc No. [illegible]
	✓ Safety-Related / Non-Safety-Related	Rev. / Date / Page 15 of

Client Detroit Edison Co.	Prepared by [illegible]	Date [illegible]
Project Fermi-2	Reviewed by [illegible]	Date 5-5-81
Proj. No. 6139-35 Equip. No.	Approved by	Date

Evaluation of shear walls (overall basis)

I_{XX} (shear wall group) = 18261886 ft⁴ ; I_{YY} (shear wall group) = 29920733 ft⁴

SECTION MODULUS	A-9	A-17	H-9	H-17
S_{XX}	18261886/80.85 = 225874.	18261886/79.15 = 230725. ✓	18261886/80.85 = 225874. ✓	18261886/79.15 = 230725. ✓
S_{YY}	29920733/105.06 = 284799. ✓	29920733/105.06 = 284799. ✓	29920733/106.12 = 276293. ✓	29920733/108.12 = 276738. ✓

MOMENT k'	A-9 M/S k/ft'	A-17 M/S k/ft'	H-9 M/S k/ft'	H-17 M/S k/ft'	SPRING	REMARKS
(9) ✓ M_{YY} (Diagonal) 1334161+1642310+280312 = 3256783 ✓	11.43 ✓	11.43	11.79	11.79	X	
743576+613704+767625+1071[illegible]+237[illegible] (M xx) = 3346[illegible]	14.82 ✓	14.50	14.82	14.50 ✓	Y	

	A-9	A-17	H-9	H-17	
Vertical accln. = 0.25g Vertical Seismic Load = 0.25 (D.L.)	0.25((25.75+26.[illegible])/2) = 0.25×25.28 = 6.32 k/ft'	0.25((34.88+31.[illegible])/2) = 0.25×32.36 = 8.09 k/ft'	0.25((25.75+27.75)/2) = 0.25×26.75 = 6.69 k/ft'	0.25((27.75+3[illegible])/2) = 0.25×33.82 = 8.46 k/ft'	Average D.L. of two intersecting walls are used.

TOTAL STRESS

A-9	$25.28 \pm (11.43^2+14.82^2+6.32^2)^{1/2}$ = 45.03 k/ft', 5.53 k/ft'	H-9	$26.75 \pm (11.79^2+14.82^2+6.69^2)^{1/2}$ = 46.83 k/ft', 6.67 k/ft'
A-17	$32.36 \pm (11.43^2+14.50^2+8.09^2)^{1/2}$ = 52.52 k/ft' ✓, 12.20 k/ft'	H-17	$33.82 \pm (11.79^2+14.50^2+8.46^2)^{1/2}$ = 54.35 k/ft' ✓, 13.30 k/ft' 377.3 psi

All compression & No tension

Form GO-3.08.1 Rev. 2

SARGENT & LUNDY ENGINEERS CHICAGO	Calcs. For Rx/Aux. Bldg. - Shear walls [illegible] Evaluation for Revised Seismic Forces	Calc. No. [illegible]
	✓ Safety-Related / Non-Safety-Related	Rev. / Date / Page [illegible] of [illegible]

Client	Detroit Edison Co.	Prepared by	[illegible]	Date [illegible]
Project	Enrico Fermi - 2	Reviewed by	[illegible]	Date [illegible]
Proj. No.	6139-38	Equip. No.	Approved by	Date

Review of Shear Forces on walls

Wall	Sectional Area ft^2	Shear Forces (k)	Shear Stress (psi)	Vertical Reinforcement Available, A_v (in^2)	Remarks
A	640 ✓	7525 ✓	81.7 ✓	See DWG. B-31 thru B-33	$< 2\sqrt{f_c'} = 126.5$ psi ok
G	640 ✓	8610 ✓	93.4 ✓	"	< 126.5 psi ok.
H	480 ✓	9279 ✓	134.2 ✓	"	> 126.5 psi See Note Below
9	762	10085 ✓	91.9 ✓	"	< 126.5 psi ok.
17	868	11417	93.4 ✓	"	< 126.5 psi ok.
Diagonal	222.9	4624	144.0	See B-19 (Section 19-19)	> 126.5 psi See Note Below

NOTE: As per the table above shear stresses for Wall H & Diagonal wall exceed $2\sqrt{f_c'}$ by a small margin. $2\sqrt{f_c'}$ is a rather conservative value and according to ACI-318-77 § 11.10.6 $V_c = 3.3\sqrt{f_c'}\,hd + \frac{N_u d}{4 l_w}$ may be used for walls with shear and axial load.

Form GQ-3.08.1 Rev. 2

SARGENT & LUNDY
ENGINEERS

DESIGN CONTROL SUMMARY
DESIGN VERIFICATION

PAGE 1

PROJECT NAME ENRICO FERMI
PROJECT NO. [illegible] UNIT NO. 2
CLIENT DETROIT EDISON CO.
CALC. NO. & DESCRIPTION SE 0013
CALC. FOR REACTOR BLDG. - CABLE TRAY HANGERS - SEISMIC REEVALUATION

☒ SAFETY RELATED ☐ NON SAFETY RELATED

COMPANY NO.
QA SERIAL NUMBER

SIGNATURE & DATE FOR REV.			IDENTIFICATION OF PAGES PREPARED/REVISED/VOIDED & REVIEW METHOD
REV. 0	PREPARER	J. Nandi 5-6-81	Prepared pages 1.1, 1.2, 2.1 thru 2.3, 3.1 thru 3.7
	REVIEWER	TKonstantellos 5/16/81 W. Allen Walker 5/16/81 5-16-81	
	APPROVER	N.V. Hingnani 5-16-81	REVIEW METHOD DETAIL REVIEW METHOD
REV. 0	PREPARER	N.V. Hingnani 5-6-81	Prepared page 1.3
	REVIEWER	J. Nandi. 5-6-81	
	APPROVER	[illegible] 5-18-81	REVIEW METHOD Detail Review Method
REV. 1	PREPARER	J. Nandi 5-21-81	Page 3 Rev1 of Design Control Summary.
	REVIEWER	TKonstantellos 5-21-81	
	APPROVER	N.V. Hingnani 5-21-81	REVIEW METHOD DETAILED
REV.	PREPARER		
	REVIEWER		
	APPROVER		REVIEW METHOD

FORM SAS 22.1 Rev. 0

SARGENT & LUNDY
ENGINEERS

DESIGN CONTROL SUMMARY
ASSUMPTIONS AND PROCEDURES
PROJECT NO. [illegible] UNIT NO. [illegible]

Revision

In order to perform seismic re-evaluation of electrical cable tray hangers it was decided to analyse a random sample of hangers from each floor. Thus we analyzed a 10% sample of 345 hangers from an approximate total of 3500. Sargent & Lundy proprietory computer program "Seismic Analysis of Hangers (SEISHANG)" was employed. "Seishang" designs the hanger as a two-dimensional frame for both static & dynamic loads from the input of geometric configuration of the hanger, material properties, member sizes, dead load & dynamic response spectra. The program determines the lightest member size by iterative process if the input size was found overstressed. In that case program supplies the actual stress in the member. Input size of each member in a hanger is the existing member size.

Design horizontal & vertical seismic spectra were obtained by enveloping N-S & E-W, and, wall & slab spectra respectively at 10% damping for SSE condition. For tabulation of spectra reference can be made to pages 2.1 thru 2.3. Envelopes of spectra at different elevations are given in pages 3.1 thru 3.7. Revised horizontal & vertical spectra were used.

Summary of the sample hanger analysis (see Page 1.1) shows that 65% of the hangers are within allowable stress limits. There are some conservatism used in this analysis which we have discussed in pargraph 4, page 1.3. In paragraph 5 & 6 we have suggested the solutions to those overstressed hangers. It is expected that correct seismic R.S. at various elevations and more accurate values of actual cable loads on trays with acceptable stress limit of 1.1 Fy should qualify most of remaining hangers. 1

FORM SAS-22.4 Rev. 0

FORM SAS 22.7 Rev. 0

SARGENT & LUNDY
ENGINEERS

DESIGN CONTROL SUMMARY
DESIGN INPUT DOCUMENTS
PROJECT NO. [illegible] UNIT NO. 2

PAGE 4 OF CALCULATIONS NO. EE 6-15 REV. 0 DATE 5.[illegible].1981

NO.	INPUT DOCUMENT			REVISION NUMBER OR DATE ON LINE INDICATED								USED IN DESIGN OF	CNT	C/N
	DESCRIPTION	SOURCE	IDENT	DATE	S	DATE	S	DATE	S	DATE	S			
1.	SEISMIC ANALYSIS OF THE Rx/Aux BUILDING	S&L	REPORT #SL-[illegible]	SEPT. 27, 1974										
2.	SEISMIC RE-ANALYSIS OF 7% DAMPING SITE SPECTRA REACTOR-AUXILIARY BUILDING	S&L	REPORT #[illegible]	APRIL 12, 1981										
3.	SPEC. FOR DESIGN OF COLD-FORMED STEEL STRUCTURAL MEMBERS	AISI	—	1968 EDITION										
4.	UNISTRUT GENERAL ENGINEERING CATALOG	UNISTRUT CORP.	CATALOG # 9	1978										
5.	S&L PROPRIETARY COMPUTER PROGRAM — "SEISHANG"	S&L	PROGRAM #[illegible]	1981										

CNT - Continued CN - Comment Number S - Status

SARGENT & LUNDY ENGINEERS CHICAGO

Calcs. For Rx-Aux. Elec. Cable Tray Hgrs. Review per New Seismic Response Spectra		Calc. No. EE 0013
X Safety-Related	Non-Safety-Related	Rev. 0 Date 5-6-81
		Page 1.1 of

Client D. E. Co.		Prepared by J. Nandi	Date 5-6-81
Project Enrico Fermi-2		Reviewed by W. A. Wolfe	Date 5/7/81
Proj. No. 6139-32	Equip. No.	Approved by	Date

Summary

Elev.	Total No. of Hgrs. Reviewed	No. of Hangers O.K.	No. of Hangers Overstressed	Percent Overstress 0-10%	10-25%	25% & over
583'-6"	73	48	25	5	7	13
613'-6"	90	59	31	5	6	20
641'-6"	141	83	58	5	10	43
659'-6"	31	27	4	–	–	4
684'-6"	10	7	3	--	1	2
Total =	345	224	121	15	24	82

Review Criteria

1. Above table is a summary of sample standard hgrs. in Rx-Aux. Bldg.
2. Input for above hgrs. were obtained from latest DCN or DCR.
3. The D.L. of cable trays including cables, tray, etc. is 40 PSF for all areas except in relay room, cable spreading room & directly below relay floor is 50 PSF.
4. Design is based on 1968 edition of AISI spec. for cold-formed steel structural members.

Form GD-3.08.1 Rev. 2

SARGENT & LUNDY ENGINEERS CHICAGO	Calcs. For Rx-Aux-Elec. Cable Tray Hgrs. Review for New Seismic Response Spectra	Calc. No. EE 0015
	X Safety–Related / Non–Safety–Related	Rev. 0 Date 5-6-81 / Page 1-2 of

Client DETROIT EDISON CO.	Prepared by J. Nowicki	Date 5-6-81
Project FERMI - 2	Reviewed by W.A. White	Date 5/7/81
Proj. No. 6130-28 Equip. No.	Approved by	Date

5. THE HORIZONTAL SEISMIC RESPONSE SPECTRA WERE OBTAINED BY ENVELOPING HORIZ. SSE EXCITATIONS IN TWO ORTHOGONAL DIRECTIONS FOR 10% DAMPING.

6. THE VERTICAL SEISMIC RESPONSE SPECTRA WERE OBTAINED BY ENVELOPING VERT. SSE EXCITATIONS FOR WALL & SLAB AT THE SAME ELEVATION FOR 10% DAMPING.

7. MAX. ALLOWABLE STRESS IS THE SMALLER VALUE OF 1.6× AISI ALLOWABLE OR 0.95Fy.

Form GQ-3.08.1 Rev. 2

SARGENT & LUNDY ENGINEERS CHICAGO	Calcs. For Reactor/Aux. Bldg. – Cable Tray Hangers – Seismic Reevaluation		Calc. No. ES 0015	
	Safety-Related	Non-Safety-Related	Rev. 0	Date
			Page 1.5	of FINAL

Client Detroit Edison Co.	Prepared by	Date 5-6-8
Project Enrico Fermi – 2	Reviewed by J. Nandi	Date
Proj. No. 6138-38 Equip. No.	Approved by	Date

1. Seismic report gives the following 'Vertical' response spectra etc.;

a) Reactor & Aux. Building **Wall** El. 583'-6", 613'-6" (C-16R)
b) Reactor & Aux. Building **Slab** El. 583'-6", 613'-6" (C-15R)
c) Reactor & Aux. Building **Wall** El. 641'-6", 659'-6" & 684'-6" (C-18R)
d) Reactor Building **Slab** El. 641'-6", 659'-6" & 684'-6" (C-17R)
e) Auxiliary Building **Slab** El. 641'-6", 659'-6" & 684'-6" (C-18R)

2. Spectra a) & b) are enveloped and used as Vertical response spectra for all types of cable trays (whether supported @ mid span, near the walls or on the walls) for all hangers upto El. 613'-6".

3. Spectra c), d) & e) are enveloped and used as 'vertical' response spectra for all cable tray hangers from 641'-6" to 684'-6".

4. The above criteria is specially conservative for
   i) all hangers @ El. 583'-6" and 641'-6" where actual vertical response will be much less than used in re-evaluation
   ii) all hangers supported on and near the main walls.
   iii) hangers in upper levels of Aux. Bldg areas since the enveloped response from Reactor Bldg spectra is higher.

5. Also the standard load of 40 lbs per sq.ft. of tray (50 lbs per sq.ft. for relay room area) used in the design is perhaps excessive for most of the trays.

6. Special review, based on actual loads and the actual seismic response, shall be made for those hangers which indicate overstressing based on initial criteria.

Form GO-3.08.1 Rev. 2

SARGENT & LUNDY ENGINEERS CHICAGO

Calcs. For Response Spectra for R.-Aux. Bldg Elec. Cable Tray Hgrs. by "SEISHANG"		Calc. No. EA-0013
X Safety-Related	Non-Safety-Related	Rev. 0 Date 5/2/81
		Page 2.1 of

Client DETROIT EDISON CO.		Prepared by J. Naidi	Date 5/2/81
Project FERMI - 2		Reviewed by Tsountoutellos	Date 5/2/81
Proj. No. 6139-38	Equip. No.	Approved by	Date

```
 1:         17                 15
 2:RX-AUX HORIZ RESPONSE SPECTRUM AT EL 583-6
 3:        .001        .220
 4:        .020        .220
 5:        .035        .240
 6:        .060        .330
 7:        .070        .350
 8:        .110        .400
 9:        .140        .400
10:        .170        .520
11:        .220        .520
12:        .300        .430
13:        .400        .300
14:        .600        .210
15:        .800        .160
16:       1.000        .130
17:       1.500        .070
18:       2.000        .055
19:      20.000        .055
20:RX-AUX VERT. RESPONSE SPECTRUM AT EL 583-6
21:        .001         .36
22:        .020         .36
23:        .030         .50
24:        .040         .85
25:        .050        1.50
26:        .070        1.50
27:        .075        1.00
28:        .080         .82
29:        .090         .82
30:        .150         .25
31:        .180         .25
32:        .300         .22
33:        .600        .210
34:       1.000        .210
35:      20.000        .105
```

ELEMENT NAME:-SPECTRA/EL-583-6SEI

```
 1:         21                 15
 2:RX-AUX HORIZ RESPONSE SPECTRUM AT EL 613-6
 3:        .001        .300
 4:        .020        .300
 5:        .030        .320
 6:        .035        .330
 7:        .040        .360
 8:        .050        .390
 9:        .060        .390
10:        .070        .350
11:        .100        .400
12:        .110        .420
13:        .150        .890
14:        .180        .970
15:        .220        .970
16:        .440        .340
17:        .600        .230
18:        .800        .170
19:        .900        .155
20:       1.000        .145
21:       1.500        .060
22:       2.000        .052
23:      20.000        .052
24:RX-AUX VERT. RESPONSE SPECTRUM AT EL 613-6
25:        .001         .36
26:        .020         .36
27:        .030         .50
28:        .040         .85
29:        .050        1.50
30:        .070        1.50
31:        .075        1.00
32:        .080         .82
33:        .090         .82
34:        .150         .25
35:        .180         .25
36:        .300         .22
37:        .600         .22
38:       1.000         .22
39:      20.000         .21
```

ELT. NAME:-SPECTRA/EL-613-6SEI

Form GQ 3.08.1 Rev. 2

SARGENT & LUNDY ENGINEERS CHICAGO

Calcs. For Response Spectra for Rx-Aux. Bldg. Elec. Cable Tray Hangers by "SEISHNGS"	Calc. No. EE0013
X Safety-Related / Non-Safety-Related	Rev. 0 Date 5/4/81
	Page 2.2 of

Client DETROIT EDISON CO.	Prepared by J. Nandi Date 5/4/81
Project FERMI - 2	Reviewed by Thomas Rautenstein Date 5/4/81
Proj. No. C139-38 Equip. No.	Approved by Date

1:	21	14
2:RE-AUX HORIZ RESPONSE SPECTRUM AT EL 641-6		
3:	.001	.400
4:	.020	.400
5:	.028	.430
6:	.036	.450
7:	.057	.490
8:	.065	.430
9:	.080	.500
10:	.090	.570
11:	.110	.750
12:	.140	1.150
13:	.220	1.150
14:	.320	.700
15:	.430	.400
16:	.600	.230
17:	.750	.170
18:	.800	.160
19:	.900	.145
20:	1.000	.125
21:	1.500	.065
22:	2.000	.054
23:	20.000	.054
24:RE-AUX VERT. RESPONSE SPECTRUM AT EL 641-6		
25:	.001	.400
26:	.020	.400
27:	.028	.400
28:	.050	1.200
29:	.082	1.200
30:	.100	.850
31:	.123	.500
32:	.150	.340
33:	.180	.340
34:	.300	.240
35:	.600	.210
36:	1.000	.210
37:	2.000	.210
38:	20.000	.210

ELT. NAME :- SPECTRA/EL-641-6SEI

1:	21	14
2:RX-AUX HORIZ RESPONSE SPECTRUM AT EL 659-6		
3:	.001	.430
4:	.020	.430
5:	.032	.450
6:	.040	.470
7:	.053	.460
8:	.060	.500
9:	.080	.600
10:	.107	.780
11:	.135	1.280
12:	.150	1.350
13:	.220	1.350
14:	.500	.340
15:	.600	.260
16:	.700	.200
17:	.800	.180
18:	.900	.160
19:	1.000	.140
20:	1.100	.130
21:	1.500	.070
22:	2.000	.060
23:	20.000	.060
24:RX-AUX VERT. RESPONSE SPECTRUM AT EL 659-6		
25:	.001	.400
26:	.020	.400
27:	.028	.400
28:	.050	1.200
29:	.082	1.200
30:	.100	.850
31:	.123	.500
32:	.150	.340
33:	.180	.340
34:	.300	.240
35:	.600	.210
36:	1.000	.210
37:	2.000	.210
38:	20.000	.210

ELT. NAME :- SPECTRA/EL-659-6SEI

SARGENT & LUNDY ENGINEERS CHICAGO

Calcs. For Response Spectra for Rx-Aux. Bldg. Elec. Cable Tray Hangers by "SEISHANG"		Calc. No. EEC013
X Safety-Related	Non-Safety-Related	Rev. 0 Date 5/4/81
		Page 2 of [illegible]

Client DETROIT EDISON CO.	Prepared by J. Nandi Date 5/4/81
Project FERMI - 2	Reviewed by Montantelloi Date 5/4/81
Proj. No. 6139 - 38 Equip. No.	Approved by Date

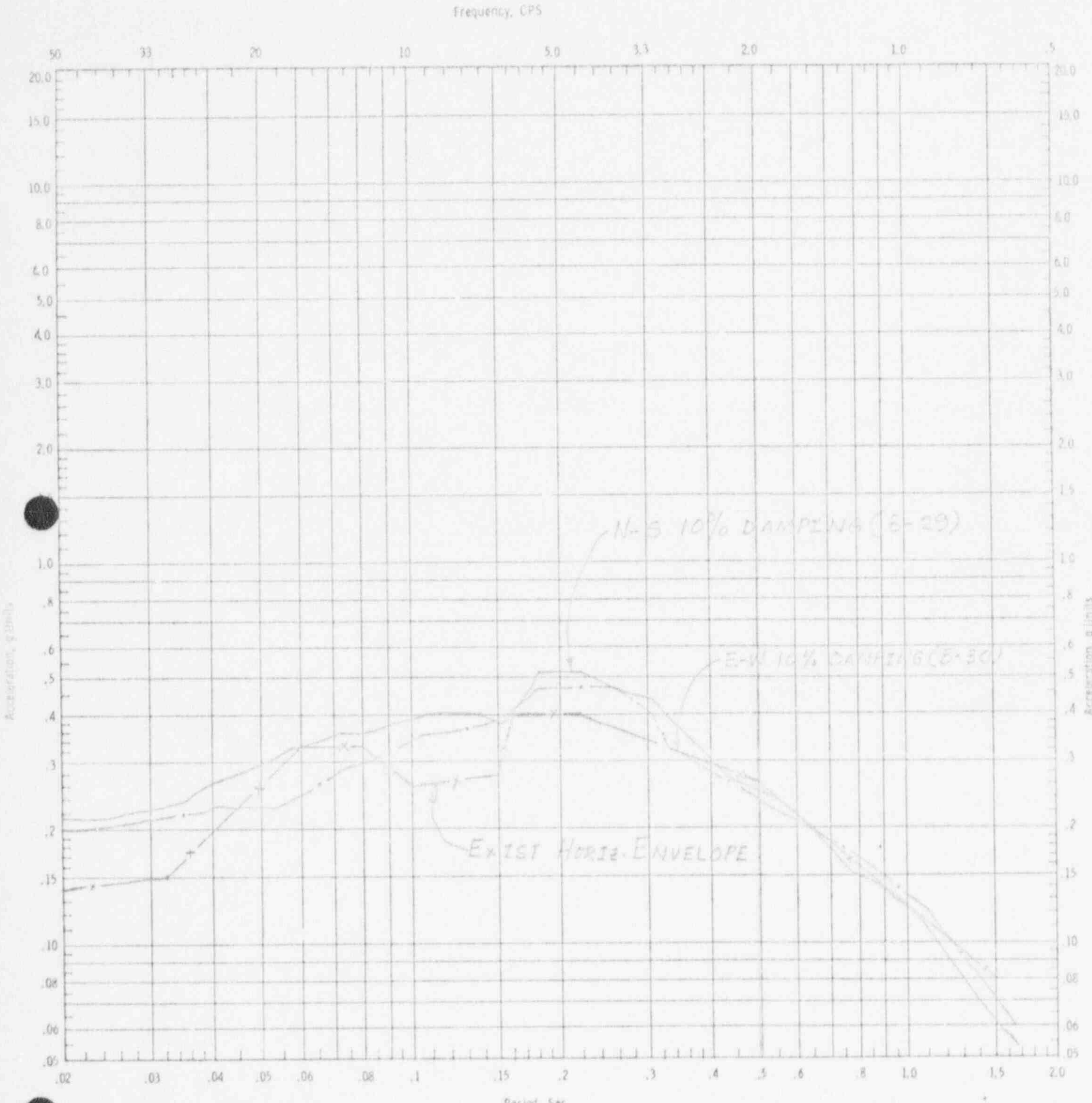
```
 1:        14                  14
 2:RX-AUX HORIZ RESPONSE SPECTRUM AT EL 684-6
 3:       .001       .460
 4:       .020       .460
 5:       .030       .470
 6:       .040       .500
 7:       .050       .600
 8:       .080       .720
 9:       .150      1.520
10:       .210      1.520
11:       .650       .220
12:       .900       .160
13:      1.200       .120
14:      1.500       .070
15:      2.000       .060
16:     20.000       .060
17:RX-AUX VERT. RESPONSE SPECTRUM AT EL 684-6
18:     .001      .4
19:     .02       .4
20:     .028      .4
21:     .050     1.2
22:     .082     1.2
23:     .100      .85
24:     .123      .50
25:     .150      .34
26:     .180      .34
27:     .300      .24
28:     .600      .21
29:    1.00       .21
30:    2.00       .21
31:   20.00       .21
```

ELT. NAME:- SPECTRA/EL-684-6SEI.

Form GQ 3.08.1 Rev. 2

SARGENT & LUNDY ENGINEERS CHICAGO	SPECTRA FOR Rx-Aux. Bldg. Elec. Cable Tray Horz. — El. 583'-6"			Calc. No. EE0013	
	X	Safety–Related	Non–Safety–Related	Rev. 0	Date 4-21-81
				Page 3-1 of	

Client D.E.Co.		Prepared by J. Nawa	Date 4-21-81
Project FERMI-2		Reviewed by T Constantine	Date 5/7/81
Proj. No. 6130-38	Equip. No.	Approved by	Date



EXCITATION Horiz. SSE

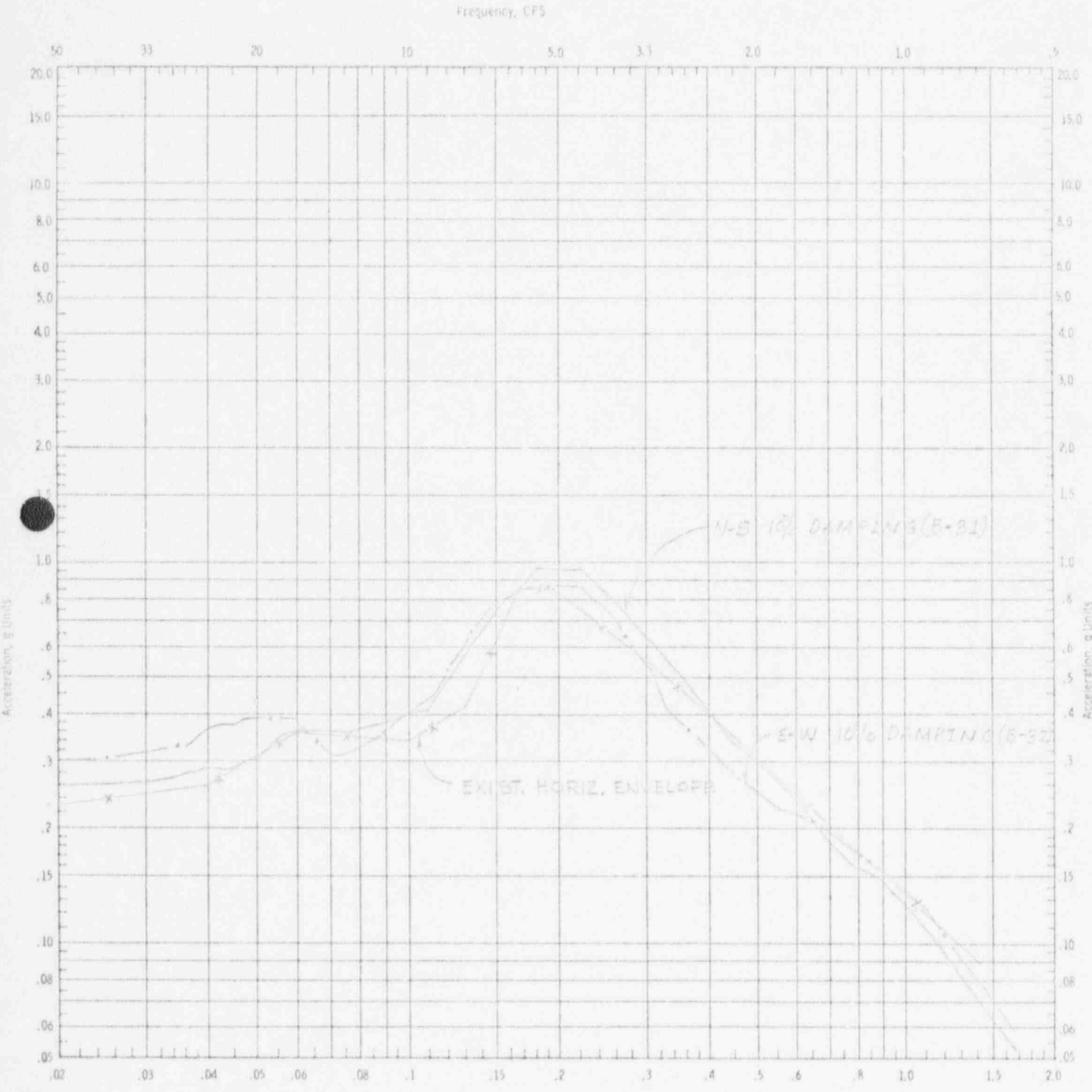
SPECTRA NO 8-29 & 8-30

LOCATION: Rx-Aux. Bldg. Slab

ELEVATION: 583'-6"

SARGENT & LUNDY ENGINEERS CHICAGO	SPECTRA FOR Rx-Aux. Bldg Electrical Cable Tray Hgrs — El. 613'-6"	Calc. No. 6600013
	✓ Safety-Related / Non-Safety-Related	Rev. 0 Date 4-21-81
		Page 52 of

Client D. E. Co.	Prepared by J. Nanda	Date 4/21/81
Project FERMI-2	Reviewed by Tronstantellos	Date 4/22/81
Proj. No. 6139-33 Equip. No.	Approved by	Date



EXCITATION HORIZ. SSE

LOCATION: Rx-Aux. Bldg. Slab

SPECTRA NO B-31 & B-32

ELEVATION: 613'-6"

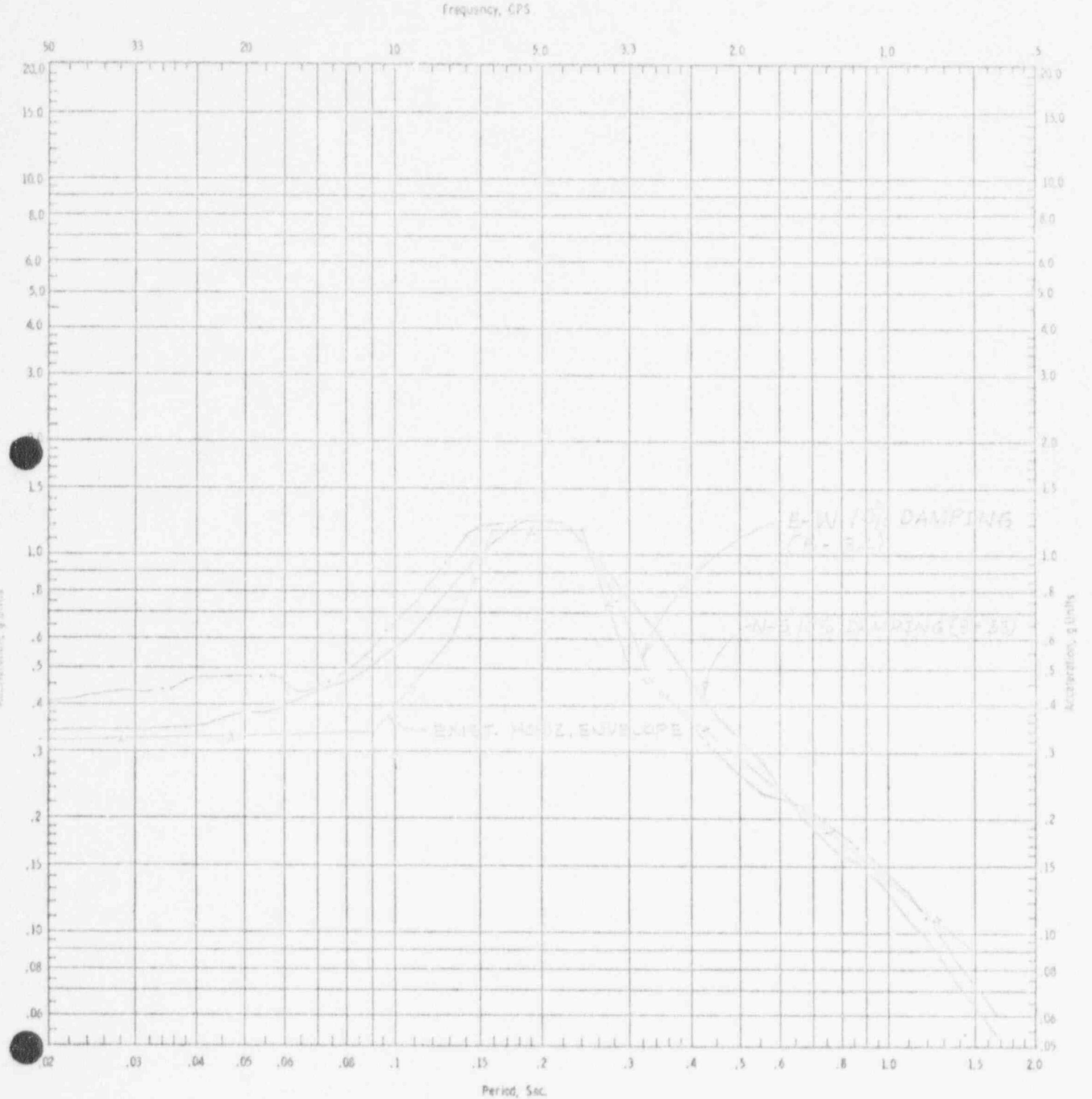
SARGENT & LUNDY
ENGINEERS
CHICAGO

SPECTRA FOR Rx-Aux. BLDG. [illegible]			
Cable Tray Hgrs. - [illegible]			
x	Safety-Related		Non-Safety-Related

Calc. No. [illegible]	
Rev. 0	Date [illegible]/81
Page [illegible] of	

Client D.E. Co.	
Project FERMI-2	
Proj. No. 6139-38	Equip. No.

Prepared by [illegible]	Date [illegible]
Reviewed by T. Konstantellos	Date 4/22/81
Approved by	Date



EXCITATION HORIZ. SSE

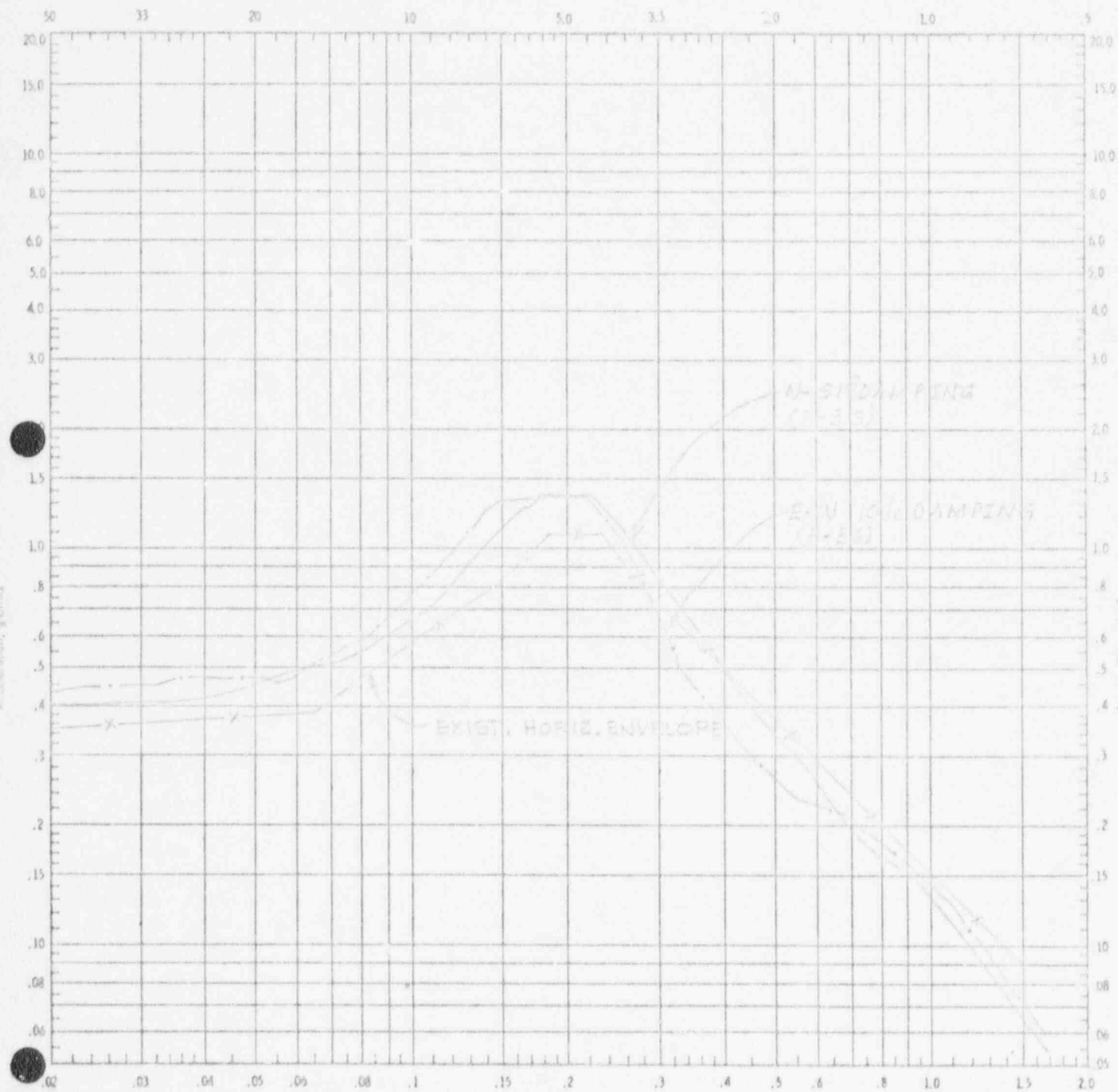
LOCATION: Rx-Aux. BLDG. SLAB

SPECTRA NO B-33 & B-34

ELEVATION: 641'-6"

SARGENT & LUNDY ENGINEERS CHICAGO	SPECTRA FOR Rx-Auxiliary Bldg. Cable Tray Hgrs - El. 659'-6"	Calc. No. [illegible]
	X Safety-Related / Non-Safety-Related	Rev. 0 Date [illegible]
		Page [illegible] of

Client D. E. Co.		Prepared by [illegible]	Date [illegible]
Project FERMI-2		Reviewed by [illegible]	Date [illegible]
Proj. No. 6139-38	Equip. No.	Approved by	Date



EXCITATION Horiz. SSE

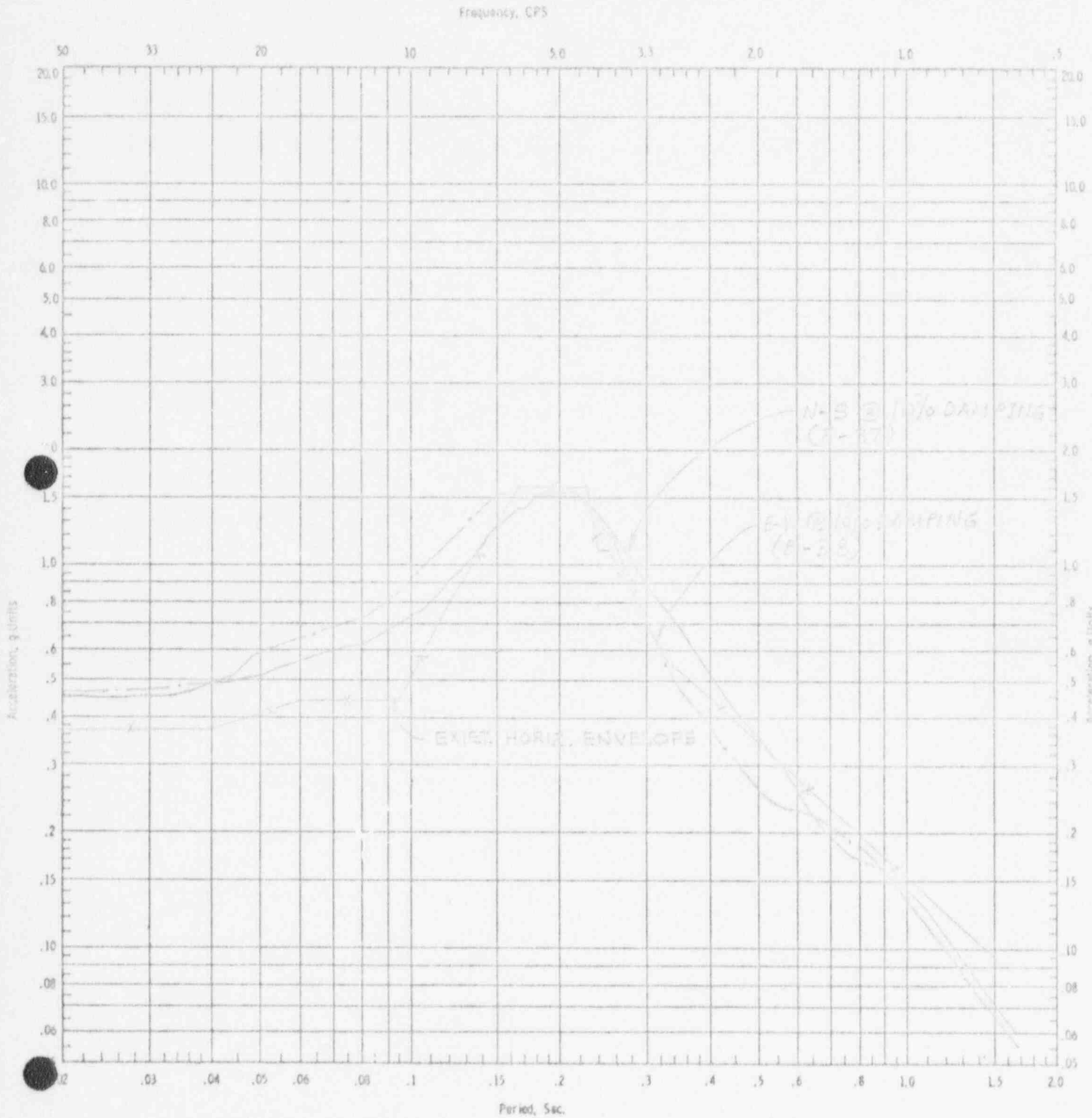
LOCATION: Rx-Aux. Bldg. Slab

SPECTRA NO B-35 & B-36

ELEVATION: 659'-6"

SARGENT & LUNDY ENGINEERS CHICAGO	SPECTRA FOR Rx-Aux. Bldg. Elev. Cable Way Hgrs - Fl. ...	Calc. No.
	X Safety–Related / Non–Safety–Related	Rev. 0 Date
		Page 55 of

Client D. E. Co.		Prepared by	Date
Project FERMI-2		Reviewed by	Date
Proj. No. 6139-38	Equip. No.	Approved by	Date



EXCITATION Horiz. SSE

LOCATION: Rx-Aux. Bldg. Slab

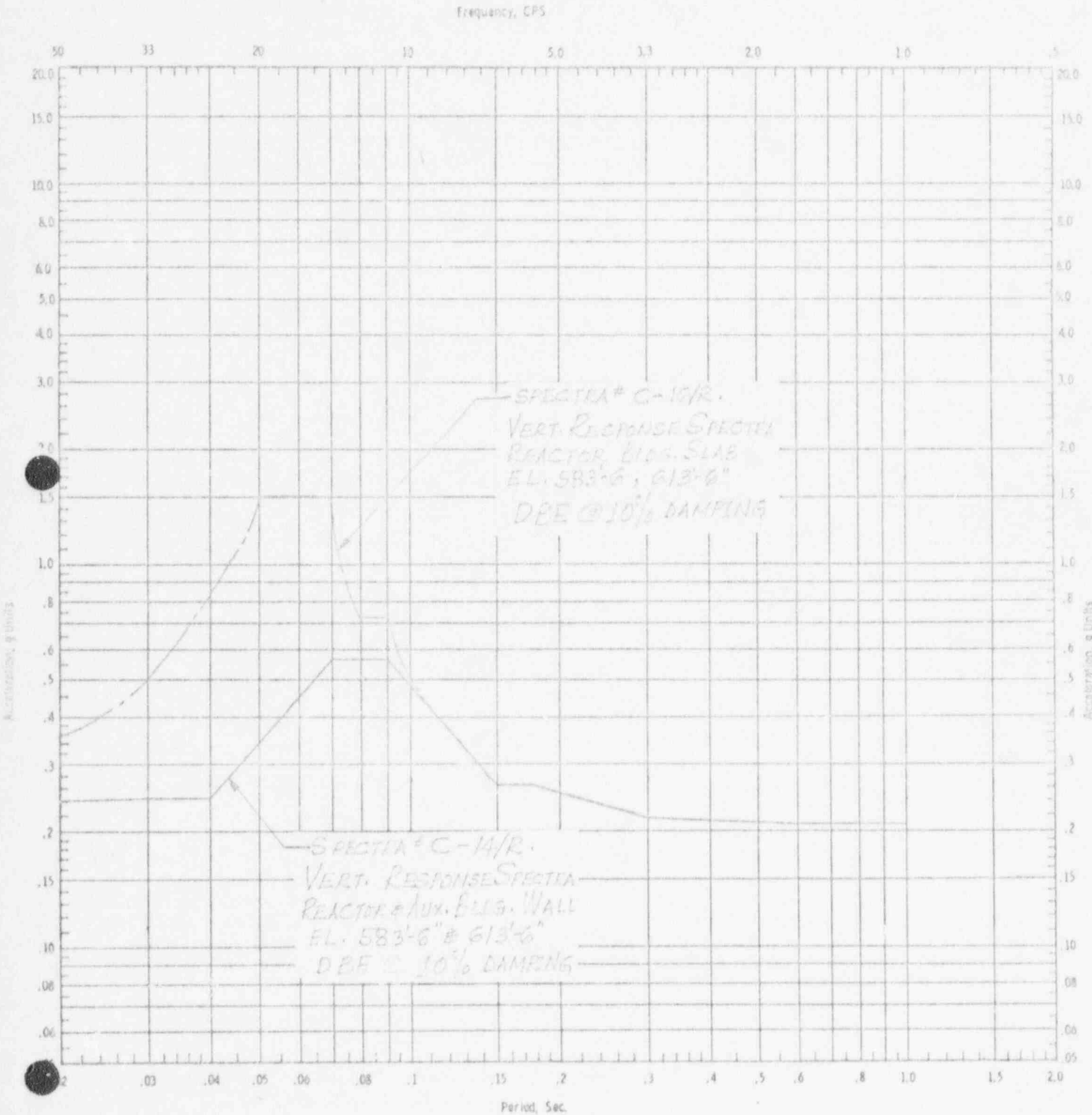
SPECTRA NO B-37 & 38

ELEVATION: 684'-6"

SARGENT & LUNDY ENGINEERS CHICAGO

SPECTRA FOR ELECTRICAL & HEATERS HANGERS IN Rx-AUX. BLDG.		Calc. No. [illegible]
X Safety-Related	Non-Safety-Related	Rev. [illegible] Date [illegible]
		Page [illegible] of

Client DETROIT EDISON CO.		Prepared by [illegible]	Date [illegible]
Project FERMI - 2		Reviewed by [illegible]	Date 5/?/81
Proj. No. 6139-38	Equip. No.	Approved by	Date



EXCITATION DBE VERT. @ 10% DAMPING

LOCATION: Rx-AUX. BLDG WALL & SLAB

SPECTRA NO C-14/R, C-16/R

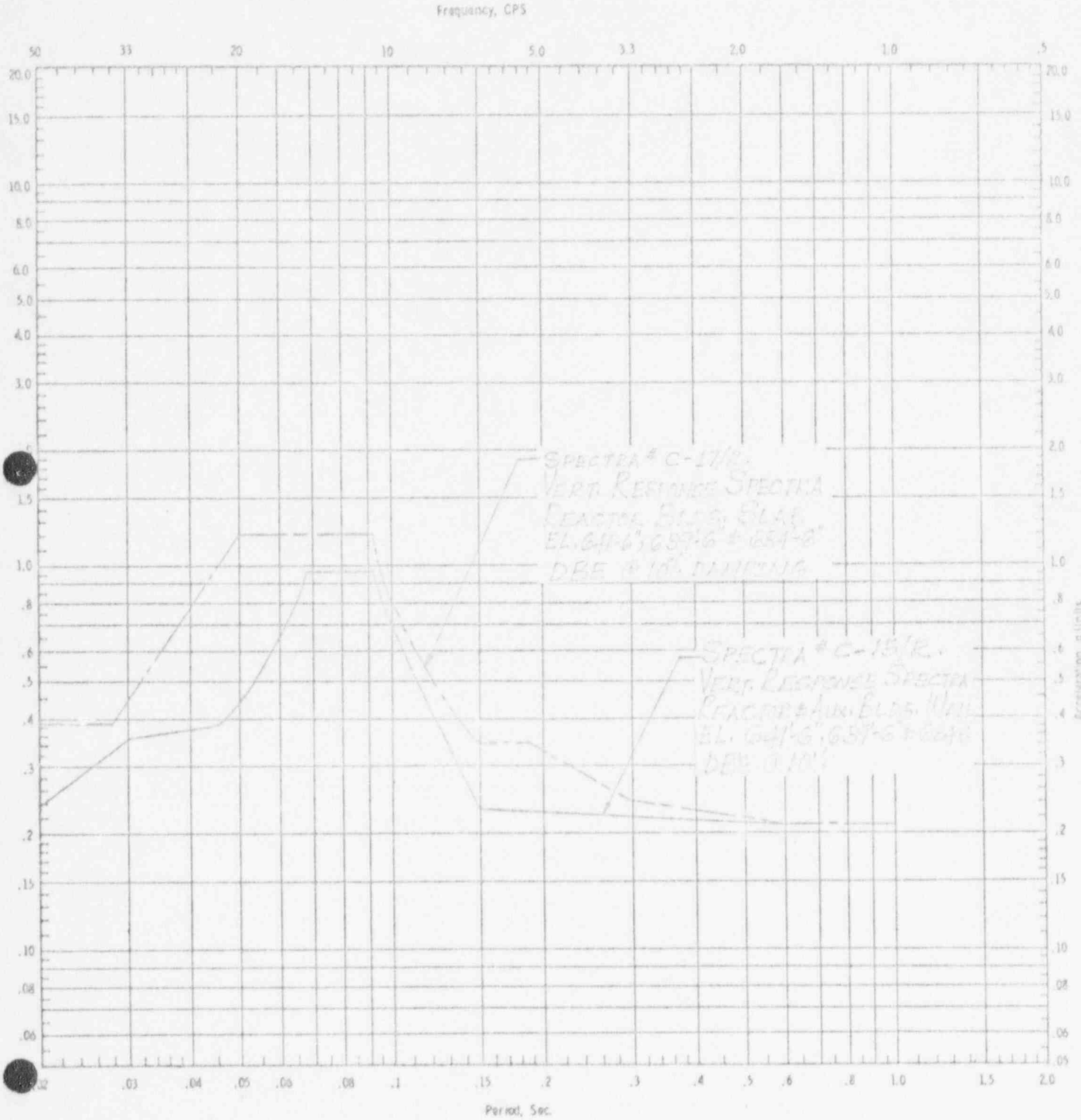
ELEVATION: 583'-6" & 613'-6"

BPD

SARGENT & LUNDY ENGINEERS CHICAGO

SPECTRA FOR ELECTRICAL CABLE TRAY HANGERS IN RX-BUILDING		Calc. No.
X Safety-Related	Non-Safety-Related	Rev.	Date
		Page of FINAL

Client DETROIT EDISON CO.	Prepared by	Date
Project FERMI - 2	Reviewed by T Constantinides	Date 5/29/81
Proj. No. 6139 - Equip. No.	Approved by	Date



EXCITATION DBE VERTICAL @ 10% DAMPING

LOCATION: RX-AUX. BLDG. WALL & SLAB

SPECTRA NO C-15/R, C-17/R

ELEVATION: 641-6", 659-6" & 684-6"

BPD

SARGENT & LUNDY ENGINEERS

DESIGN CONTROL SUMMARY
DESIGN VERIFICATION

PAGE 1

PROJECT NAME Enrico Fermi
PROJECT NO. 6370-53 UNIT NO. 2
CLIENT Detroit Edison Co.
CALC. NO. & DESCRIPTION SS 001
CALC. FOR Reactor Bldg - [illegible] Seismic Reevaluation

☒ SAFETY RELATED ☐ NON SAFETY RELATED

COMMENT NO. | QA SERIAL NUMBER

SIGNATURE & DATE FOR REV. 0		IDENTIFICATION OF PAGES PREPARED/REVISED/VOIDED & REVIEW METHOD
PREPARER	P. Davis 5-15-81	p. 1-39
REVIEWER	M. Khoyyata 5-18-81	
APPROVER	N.V. Hingorani 5-18-81	REVIEW METHOD Review of representative [illegible]

SIGNATURE & DATE FOR REV __		IDENTIFICATION OF PAGES PREPARED/REVISED/VOIDED & REVIEW METHOD
PREPARER		
REVIEWER		
APPROVER		REVIEW METHOD

SIGNATURE & DATE FOR REV __		IDENTIFICATION OF PAGES PREPARED/REVISED/VOIDED & REVIEW METHOD
PREPARER		
REVIEWER		
APPROVER		REVIEW METHOD

SIGNATURE & DATE FOR REV __		IDENTIFICATION OF PAGES PREPARED/REVISED/VOIDED & REVIEW METHOD
PREPARER		
REVIEWER		
APPROVER		REVIEW METHOD

SARGENT & LUNDY ENGINEERS

DESIGN CONTROL SUMMARY
DESIGN ELEMENT INDEX
PROJECT NO. 6139-65 UNIT NO. 2

PAGE 2 OF CALCULATIONS NO. SC 0001 REV. 0 DATE [illegible]

Revision

Rx/Aux Bldg Superstructure Steel

Contents

FORM SAS 22.3 Rev. 0

SARGENT & LUNDY
ENGINEERS

DESIGN CONTROL SUMMARY
DESIGN INPUT DOCUMENTS

PROJECT NO. [illegible] UNIT NO. 2

PAGE 1 OF CALCULATIONS NO. [illegible] REV. 0 DATE [illegible]

FORM SAS 22.7 Rev. 0

NO.	INPUT DOCUMENT			REVISION NUMBER OR DATE ON LINE INDICATED								USED IN DESIGN OF	C N T	C / N
	DESCRIPTION	SOURCE	IDENT	DATE	S	DATE	S	DATE	S	DATE	S			
1.	SEISMIC Reanalysis for 7% Damping SITE SPECTRA Rx/Aux Bldg	S&L	[illegible]	Rev. 0										
2.	Steel Construction Manual	AISC	7th Ed.	1970										
3.	Reactor Bldg. Column Bracing Rows & Det's	S&L	B-172	Rev J										
4.	Reactor Building Roof Framing Plan South Area	S&L	B-184	Rev F										
5.	Reactor Building Roof Framing Plan North Area	S&L	B-185	Rev C										
6	Reactor Building Column Schedule	S&L	B-190	Rev H										
7	360 MPH TORNADO — SIDING BLOWN OFF	S&L	402.08	Rev. 0										

CNT - Continued CN - Comment Number S - Status

SARGENT & LUNDY
ENGINEERS

DESIGN CONTROL SUMMARY
ASSUMPTIONS AND PROCEDURES
PROJECT NO. [illegible] UNIT NO. [illegible]

PAGE 3 OF CALCULATIONS NO. [illegible] REV. 0 DATE 5-5-81

To determine the adequacy of the superstructure steel for the increased SSE loading, the following procedure will be used to survey the superstructure members:

1) Compare the loads in the members of the vertically braced Rows as calculated from the SSE shears given in the seismic Report with the design loads shown on the structural drawings.

2) Compare the loads in the Roof truss members as calculated when the SSE shears are applied to the truss with the design loads in the members as shown on the structural drawings.

3) Make a frequency check of critical members at the Roof elevation based on the SSE response spectra.

The assumption is that if the axial or transfer forces shown on the structural drawings can be shown to be greater than those calculated using the revised SSE loadings, then the members should be adequate since they were originally designed for the governing case between OBE, SSE & Tornado.

The detailed summary of design procedure is given on page 1 & 2 of calculation sheets.

FORM SAS-22.4 Rev. 0

Revision

SARGENT & LUNDY ENGINEERS CHICAGO	Calcs. For Refuel. Bldg. Superstruc. steel for Seismic Results from [illegible]	Calc. No. [illegible]
	X Safety-Related / Non-Safety-Related	Rev. 0 Date / Page 1 of 22

Client [illegible]	Prepared by [illegible]	Date [illegible]
Project Fermi-2	Reviewed by M. K. [illegible]	Date [illegible]
Proj. No. [illegible]-88 Equip. No.	Approved by	Date

Summary

I Vertical Bracings and Column Rows

The seismic shear in each braced row was determined from the computer output which showed the force in each spring with a different spring for each row. This load was then applied to each vertically braced bay at the roof level - even tho it could be assumed that for the two braced bays in each row that one-half the load is resisted by each bay. The loads that are calculated in the bracing members are then reduced to a working stress level and compared to the working stress level member loads shown on the structural drawings. Since the loads on the structural drawings reflect what the members were originally designed for because the shop detailer had to design the connections for the loads shown. Therefore, since our very conservative approach shows that the original member design forces are equal to or greater than what we now calculate, the members and connections must be OK. And since the original horizontal shears must be larger, the axial loads in the columns and anchor bolts due to the moment of loads on the vertical truss must be larger. Therefore, the column & anchor bolts should be OK.

II Horizontal Roof Trusses & Roof Members

Only the upper roof trusses were considered, the lower roof trusses laterally supporting the lower flange of the girders and located in the interior bays only were ignored. This is a conservative assumption since these trusses will also act to help resist the lateral loads and make the roof act as a shear pl. and distribute the loads more evenly. The total roof shear in one direction was divided in half and applied to the truss resisting this shear at the truss panel pts. The calculated member loads were then reduced to a working stress level and

Form GO 3.08.1 Rev. 2

SARGENT & LUNDY ENGINEERS CHICAGO	Calcs. For Reactor Bldg Superstruc. Steel for Seismic Reevaluation for New Response Spectra	Calc. No. SS-001
	X Safety-Related / Non-Safety-Related	Rev. 0 Date
		Page 2 of 38

Client DECo	Prepared by [illegible]	Date 5-7-81
Project Fermi-2	Reviewed by [illegible]	Date 5-14-81
Proj. No. 6139-38 Equip. No.	Approved by	Date

II Horizontal Roof Trusses & Roof Members cont

compared to the working stress level member loads shown on the drawings, since these are the orig loads the member connections were designed for and the members must be used for. It was found that the calculated loads were equal to or less than the drawings loads for most of the members and only slightly higher for the rest. Therefor, the connections should be ok.
The frequency of the individual members were then calculated and the loads in the main roof girders and one truss diagonal were calculated. The two sample members checked out ok both for their individual load cases and when the truss loads were added to them. The remainder of the members had their frequencies looked at and where they fell on the response spectra curves. In most cases the g levels were less than 1.0 occasionally going to 1.5. It therefor appears that they should also be ok or only slightly overstressed.

CRANE ELEVATION

From all the calculations it can be seen that the tornado loading originally designed for still governs the design. It can be assumed therefore, that this would also hold true for the crane elevation steel. Under an SSE condition the crane is parked by the 17 row — this still holds true. The crane girder was designed for total exposure to tornado pressure.

Form GO-3.08.1 Rev. 2

SARGENT & LUNDY ENGINEERS CHICAGO	Calcs. For Reflux Bldg. Superstructure steel for seismic reevaluation [illegible]	Calc. No. [illegible]
	X Safety-Related / Non-Safety-Related	Rev. 0 Date / Page [illegible] of [illegible]

Client DECo		Prepared by [illegible]	Date [illegible]
Project Fermi - 2		Reviewed by M. Khazzaka	Date 5/14/81
Proj. No. 5135-28	Equip. No.	Approved by	Date

Reference:

Preliminary Response Spectra & Time history analysis from Specialists Group dated 4-16-81, see p. 118 - 120; see drwg's 13-192

This is now incorporated in calc book No. SDO-SECO-0003.

MAXIMUM RESPONSES OF SLABS (from p. 118)

	SLAB COMP	DISPLACEMENT FT	TIME SEC	ACCELERATION G	DYNAMIC FORCE KP	TIME SEC	
	1	.13920	4.670	.189	7386.007	2.440	
	2	.16002	4.600	-.288	-6918.120	2.550	
	3	.16181	4.650	-.400	-8081.232	2.550	
X-Comp.	4	.16225	4.660	-.432	-7768.810	2.550	
	5	.16281	4.660	-.462	-9531.954	2.550	
	6	.17095	4.730	-1.186	-811.365	2.970	Roof
	7	.15911	4.670	-.301	-243.584	2.530	
	8	.16056	4.570	-.315	-144.256	2.520	
	9	.16208	4.680	.504	4572.491	2.630	
	10	.12690	6.300	.203	7950.501	2.440	
	11	.12809	6.310	.259	6216.611	4.980	
	12	.12956	6.310	.349	7064.448	2.480	
Y-Comp.	13	.12954	6.310	.396	7123.071	2.480	
	14	.12999	6.310	.438	9025.572	2.480	
	15	.14811	6.330	1.216	831.786	5.020	Roof
	16	.12721	6.310	.208	108.485	2.440	
	17	.12749	6.310	.205	94.033	2.440	
	18	.13042	6.310	.503	4593.323	4.980	
	19	-.00001	2.700	.000	113780.387	2.700	
	20	-.00002	5.000	.001	141502.098	2.760	
	21	-.00003	2.700	.001	153816.828	2.700	
	22	-.00004	5.000	-.001	-156353.115	5.090	
θ-Comp.	23	-.00004	5.000	-.002	-193450.937	5.050	
	24	-.00018	2.700	.006	14499.304	2.700	
	25	.00008	2.520	-.006	-1361.206	2.520	
	26	.00008	2.520	-.007	-829.039	2.510	
	27	-.00004	5.000	-.002	-32864.127	5.090	

MAXIMUM STORY MOMENTS

SLAB NO	MAX THETA-Y KP-FT	THETA-X KP-FT	TIME SEC	THETA-Y KP-FT	MAX THETA-X KP-FT	TIME SEC
0	-4582101.937	-2407697.219	2.550	26985.500	4267652.687	4.980
1	-2757501.281	-1412719.422	2.550	1225563.109	2589147.500	2.480
2	-1648997.187	-826682.852	2.550	816043.914	1574261.156	2.480
3	-811256.906	-391119.645	2.550	439826.895	793862.941	2.480
4	-418171.762	-190813.699	2.550	242329.637	412337.941	2.480
5	-88748.200	-41795.345	2.560	744.174	84017.711	5.000
6	.000	.000	.000	.000	.000	.000
7	-2757501.281	-1412719.422	2.550	1225663.109	2589147.500	2.480
8	-1851602.909	-933384.898	2.550	890747.095	1759924.828	2.480
9	-30828.073	-4953.019	2.970	9295.990	31607.863	5.020

The following calc's are an approx CHECK of the superstructure steel for the NEW Response spectra. If the loads generated by the new spectra can be shown to be about the same as the orig design loads on the drwg's then the members should be OK for the new seismic loads. Orig. governing design loads were based on tornado load.

Form GQ-3.08.1 Rev. 2

SARGENT & LUNDY
ENGINEERS
CHICAGO

Calcs. For		Calc. No.	
[illegible]		Rev.	Date
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Client	Prepared by	Date
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Proj. No. Equip. No.	Approved by	Date

MAXIMUM SHEAR FORCES OF X-SPRINGS (from p.119)

SPRING NO	START SLAB	END SLAB	SHEAR FORCE KP	TIME SEC
101	0	1	8176.339	2.550
103	0	1	1909.667	2.550
105	0	1	-1555.518	2.550
107	0	1	1555.518	2.550
109	0	1	1666.853	2.550
111	0	1	1669.853	2.550
113	0	1	2402.051	2.550
115	0	1	9045.672	2.550
701	0	7	[illegible]	[illegible]
10201	1	2	15.451	2.550
10203	1	2	2888.310	2.550
10205	1	2	6071.251	2.550
10207	1	2	759.073	2.550
10209	1	2	1255.545	2.550
10211	1	2	-8673.023	2.720
10213	1	2	-3262.082	2.720
10215	1	2	-1370.533	2.720
10217	1	2	-1279.047	2.720
10701	1	7	86.240	2.530
20301	2	3	7143.992	2.550
20303	2	3	918.300	2.550
20305	2	3	634.109	2.560
20307	2	3	1019.755	2.560
20309	2	3	3002.297	2.550
20311	2	3	8210.053	2.550
20313	2	3	1920.921	2.550
20315	2	3	2082.875	2.550
20317	2	3	2415.626	2.550
20319	2	3	8254.033	2.550
20801	2	8	-118.183	2.470
30401	3	4	6576.102	2.550
30403	3	4	646.509	2.550
30405	3	4	2748.690	2.550
30407	3	4	3308.774	2.550
30409	3	4	-1140.980	2.630
30411	3	4	933.576	2.720
30413	3	4	2551.568	2.720
30415	3	4	529.512	2.770
30417	3	4	1569.233	2.550
30501	3	5	1837.953	2.720
40501	4	5	4294.581	2.560
40503	4	5	521.854	2.550
40505	4	5	4213.947	2.550
40507	4	5	1811.360	2.550
40509	4	5	1456.373	2.720
40511	4	5	2058.935	2.720
40513	4	5	524.475	2.720
9 · 50601	5	6	-561.802	2.880
12 50603	5	6	-357.639	3.060
50901	5	9	478.127	2.560
50903	5	9	-923.162	2.640
50907	5	9	-887.622	2.640

E-W DIRECTION

9 [illegible]

12 [illegible]

Form GQ-3.08.1 Rev. 2

SARGENT & LUNDY ENGINEERS CHICAGO	Calcs. For [illegible]	Calc. No. [illegible]
	✓ Safety-Related / Non-Safety-Related	Rev. 0 Date
		Page [illegible] of [illegible]

Client DECo	Prepared by	Date [illegible]
Project Fermi 2	Reviewed by M. Khayyat	Date [illegible]
Proj. No. [illegible] Equip. No.	Approved by	Date

MAXIMUM SHEAR FORCES OF Y-SPRINGS (from p.120)

SPRING NO	START SLAB	END SLAB	SHEAR FORCE KP	TIME SEC
102	0	1	-7525.842	2.480
104	0	1	-1361.743	2.480
106	0	1	-1361.743	2.480
108	0	1	-1602.510	4.980
110	0	1	-1602.510	4.980
112	0	1	-1203.431	4.990
114	0	1	-7407.491	4.900
116	0	1	-2496.944	4.990
118	0	1	-5266.444	4.990
120	0	1	-1515.672	4.990
10202	1	2	-6747.374	2.480
10204	1	2	-517.323	4.990
10206	1	2	-497.789	4.990
10208	1	2	-1800.930	4.990
10210	1	2	-1119.461	4.990
10212	1	2	-2066.605	4.990
10214	1	2	-3682.240	4.990
10216	1	2	-8588.413	4.990
10702	1	7	-678.863	2.500
10704	1	7	-699.162	2.480
20302	2	3	-5257.900	2.480
20304	2	3	-895.571	2.480
20306	2	3	-9821.027	2.480
20308	2	3	-3255.861	4.990
20310	2	3	-4023.287	4.990
20312	2	3	-2803.079	4.990
20314	2	3	-1555.710	2.480
20802	2	8	582.492	2.480
20804	2	8	562.742	2.500
30402	3	4	-1968.081	2.480
30404	3	4	-2565.659	2.480
30406	3	4	-3407.108	2.480
30408	3	4	-3259.922	4.990
30410	3	4	-1939.495	4.990
30412	3	4	-293.560	4.990
30414	3	4	-3742.607	2.480
30416	3	4	-4550.376	2.480
40502	4	5	777.716	3.010
40504	4	5	-992.243	4.990
40506	4	5	-1980.112	4.990
40508	4	5	-1401.557	4.990
40510	4	5	-2518.491	4.990
40512	4	5	-1339.912	4.990
40514	4	5	-1672.709	2.480
40516	4	5	-2351.511	2.480
40518	4	5	-944.510	2.480
40520	4	5	-1127.185	2.480
50602	5	6	-385.137	5.030 A Row SPRING
50604	5	6	-414.117	5.030 F Row SPRING
50902	5	9	-833.968	2.480

N-S direction

N-S A F

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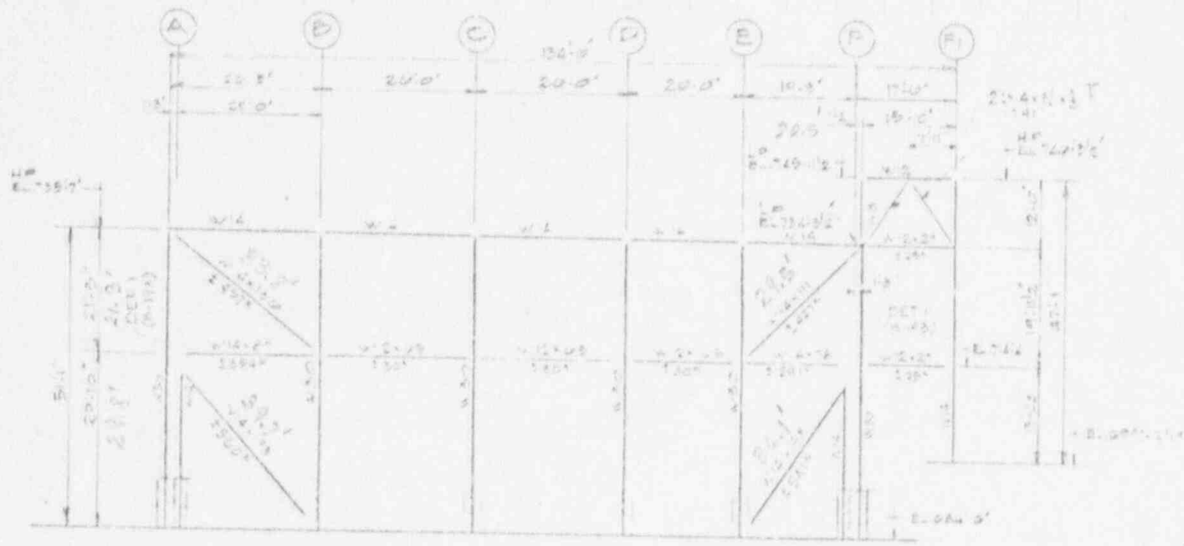
Client DECo		Prepared by [illegible]	Date [illegible]
Project Fermi 2		Reviewed by [illegible]	Date [illegible]
Proj. No. 6139-38	Equip. No.	Approved by	Date

CHECK Row 9



COLUMN ROW 9

Column Row 9 is modeled as spring no. 50601
SPRING LOAD from Response Spectra (p.119) = 361.8^k

Since the loads shown on the vertical bracings above, i.e. loads diagonals & connections are designed for, are approximatly equal for bays A-B and E-F it could be assumed that the stiffness of each bay is equal.
FIRST CHECK if only one braced bay is caplable of RESISTING SHEAR since this is the most CONSERVATIVE

	BAY A-B	BAY E-F
SHEAR @ EL. 684'-6	$\frac{26.3}{39.7}(560) = 371^k$	$\frac{20.5}{36.1}(541) = 306^k$
SHEAR @ EL. 714'-4	371^k	306^k
SHEAR @ EL. 735'-7	$\frac{26.3}{33.8}(457) = 356^k$	$\frac{20.5}{29.5}(427) = 295^k$

SARGENT & LUNDY ENGINEERS CHICAGO	Calcs. For Reactor Bldg. Superstruc. Steel for Seismic Reevaluation for new Response Spectra	Calc. No. SS-0001
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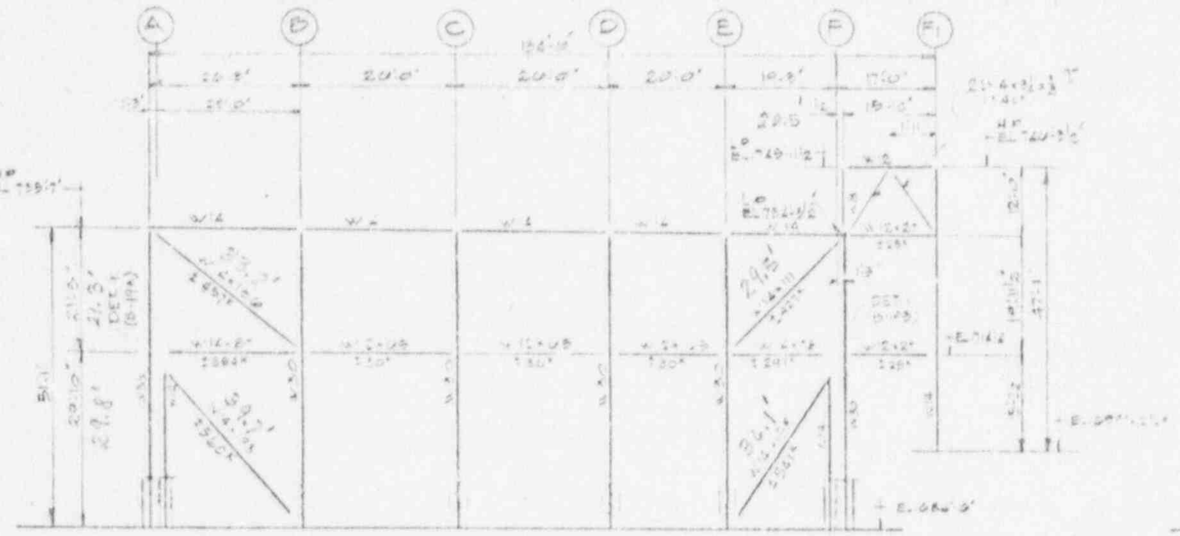
Client DECo	Prepared by	Date
Project Fermi - 2	Reviewed by	Date 5/13/81
Proj. No. 6139-38 Equip. No.	Approved by	Date

CHECK ROW 9



COLUMN ROW - 9

Column Row 9 is modeled as spring no. 50601

SPRING LOAD from Response Spectra (p. 119) = 361.8^k

Since the loads shown in the vertical bracings above, i.e. loads diagonals & connections are designed for, are approximatly equal for bays A-B and E-F it could be assumed that the stiffness of each bay is equal. FIRST CHECK if only one braced bay is capable of RESISTING SHEAR since this is the most CONSERVATIVE

	BAY A-B	BAY E-F
SHEAR @ EL. 684'-6	$\frac{26.3}{39.7}(560) = 371^k$	$\frac{20.5}{36.1}(541) = 306^k$
SHEAR @ EL. 714'-4	371^k	306^k
SHEAR @ EL. 735'-7	$\frac{26.3}{33.8}(457) = 356^k$	$\frac{20.5}{29.5}(427) = 295^k$

SARGENT & LUNDY ENGINEERS CHICAGO

Calcs. For R/Aux Bldg Superstruc. Steel and Seismic Reevaluations for New Response Spectra		Calc. No. SS-201
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Client DECo	Prepared by	Date 4-24-81
Project Fermi-2	Reviewed by	Date 5/12/81
Proj. No. 6139-38 Equip. No.	Approved by	Date

Check Col Row 9 cont

Reduce spring load to working stress level in order to compare with loads shown in vertical bracings which are at working stress level.

$361/1.6 = 226^k$

Therefore, by comparing total shear load in Row 9 of 226^k with design shear loads from vertical bracing it can be seen that even the lowest design shear load for only one bay is greater than the seismic shear load for the Row.

1 bay min shear cap. $= 295^k > 226^k$ OK

Row 9 min shear capacity 2 bays $= 356^k + 295^k = 651^k \gg 226^k$ OK

If the horizontal component is larger @ all elevations than the seismic shear load from the spring then the vertical component must also be larger. Therefore, the columns & anchor bolts should be OK

Therefore, Column Row 9 columns, anchor bolts and bracings are OK for new seismic loads.

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SARGENT & LUNDY ENGINEERS CHICAGO	Calcs. For Rx/Aux Bldg Superstruc Steel for Seismic Reevaluation for New Response Spectra	Calc. No. SS 2021
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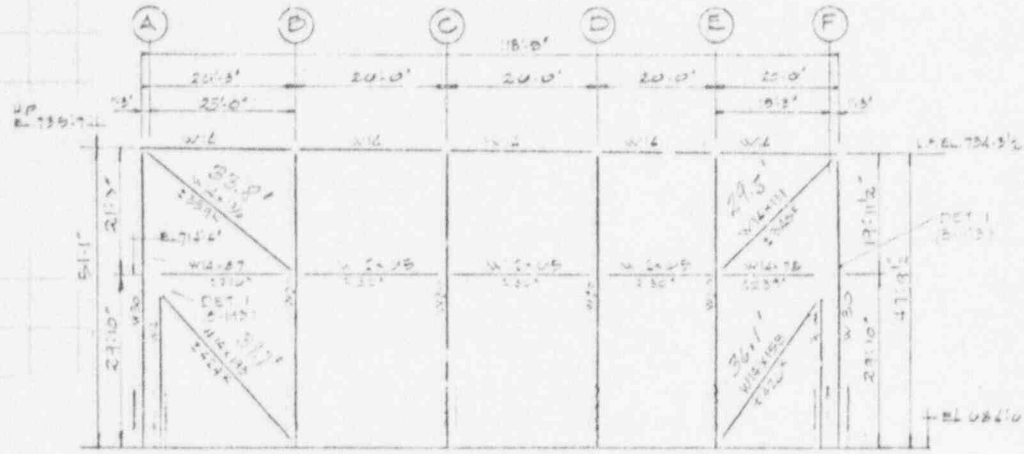
Client DECo	Prepared by G. Dixon	Date 4-24-81
Project Fermi-2	Reviewed by M. Khayyatan	Date 5/14/81
Proj. No. 6123-38 Equip. No.	Approved by	Date

CHECK ROW 17



Column Row 17 is modeled as spring no. 50603
spring load from output (p.119) = 357.7 ^{K}

Determine SHEAR FORCE in ROW @ EACH ELEVATION IN EACH BRACED bay

	BAY A-B	BAY E-F
SHEAR @ EL. 684'-6	$\frac{26.3}{39.7}(424^k) = 281^k$	$\frac{20.5}{36.1}(426^k) = 242^k$
SHEAR @ EL. 714'-4	281^k	242^k
SHEAR @ EL. 735'-7	$\frac{26.3}{33.8}(359) = 279^k$	$\frac{20.5}{29.5}(348^k) = 242^k$

Reduce Spring Load to working stress level for comparing

$358/1.6 = 224^k$

Form GQ-3.08.1 Rev. 2

SARGENT & LUNDY ENGINEERS CHICAGO	Calcs. For Reactor Bldg. Superstructure Steel for Seismic Reevaluation for Spring Response Spec.	Calc. No. 530201
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Client DECo	Prepared by J. Dean	Date 4-24-81
Project Fermi - 2	Reviewed by M. Chuggate	Date 5/14/81
Proj. No. 6138-58 Equip. No.	Approved by	Date

Check Col Row 17 con't

Compare spring loads with design shear loads
capacity of any one bay is larger than spring load as well as the entire capacity is much greater than spring load

$$1 \text{ bay min load} = 242^k > 224^k \quad \text{OK}$$
$$\text{min. Row 17 capacity} = 279^k + 242^k = 521^k >> 224^k \quad \text{OK}$$

Since horizontal components of vert. bracing are larger than seismic shears then the vertical components are larger. Therefore, the columns & anchor bolts are OK

Therefore, Col. Row 17 columns, anchor bolts and bracings are OK for new seismic loads

SARGENT & LUNDY ENGINEERS CHICAGO	Calcs. For Reactor Bldg. Superstructure Steel for Seismic Reevaluation for the Pressure Section	Calc. No. 53 0001
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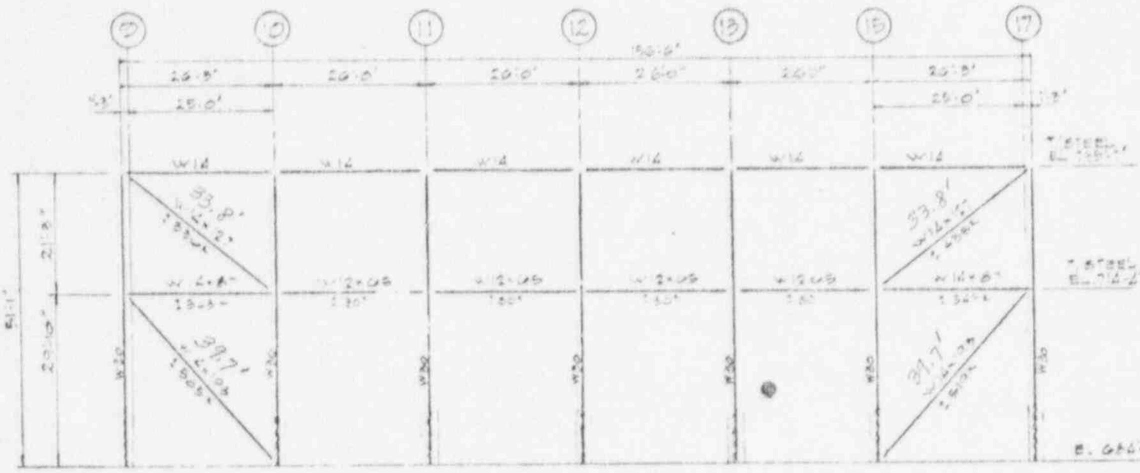
Client DECo	Prepared by	Date
Project Fermi - 2	Reviewed by M. Shimojita	Date 5/4/81
Proj. No. 5139-28 Equip. No.	Approved by	Date

Check Col. Row A



COLUMN ROW-A

Column Row A is modeled as springs in 50602
spring load from output (p.120) = 385.1^k

DETERMINE SHEAR IN ROW @ EACH ELEVATION

	Bay 9-10	Bay 15-17
SHEAR @ EL. 684'-6	$\frac{26.3}{39.7}(585) = 388^k$	$\frac{26.3}{39.7}(519) = 344^k$
SHEAR @ EL. 714'-4	388^k	344^k
SHEAR @ EL. 735'-7	$\frac{26.3}{33.8}(336) = 261^k$	$\frac{26.3}{33.8}(435) = 338^k$

Reduce spring load to working stress level for comparing

$385.1/1.6 = 241^k$

SARGENT & LUNDY ENGINEERS CHICAGO	Calcs. For Rx. Bldg. Superstructure Steel for Seismic Reevaluation for new Response Spectra	Calc. No. SS 0201
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Client DECo	Prepared by [illegible]	Date 4-25-81
Project Fermi - 2	Reviewed by M. Khayyata	Date 5/14/81
Proj. No. 5133-38 Equip. No.	Approved by	Date

Check Col Row A cont

Compare spring load with design shear loads
capacity of any one bay is larger than spring load as well as the entire capacity is much greater than spring load

$$1 \text{ bay min load} = 261^k > 241^k \quad OK$$
$$\text{min Row A capacity} = 261 + 338 = 599^k >> 241^k \quad OK$$

Since horizontal components of vertical bracings are larger than seismic shears, then the vertical components are larger. therefor, the columns and anchor bolts are OK

Therefore, column Row A columns, anchor bolts, beams, conn's and bracings are OK for new seismic loads

SARGENT & LUNDY ENGINEERS CHICAGO	Calcs. For Rx. Bldg. Superstruc. Steel for Seismic Reevaluations for New Response Spectra	Calc. No. SS0001
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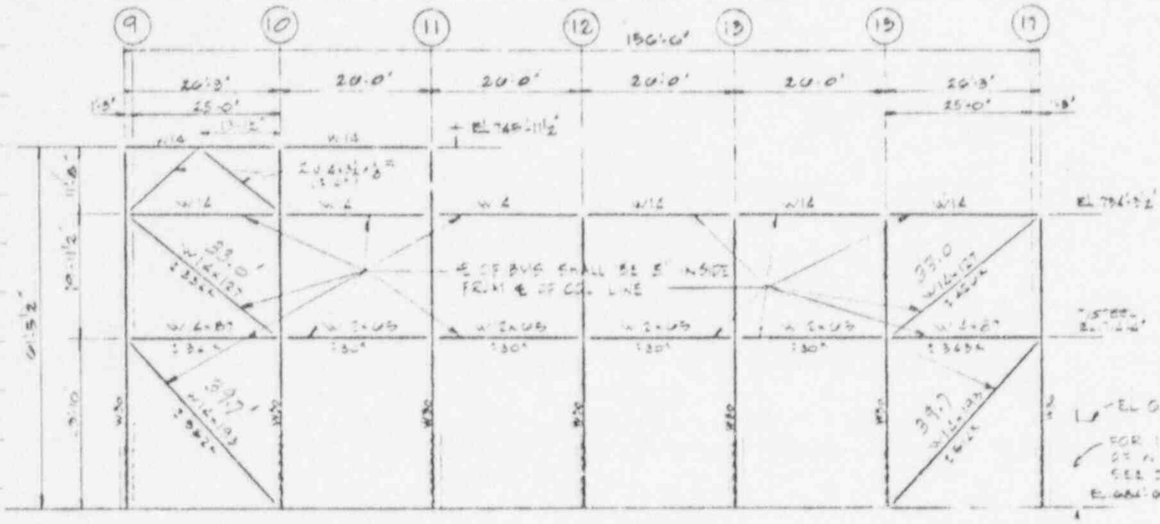
Client DEC	Prepared by	Date
Project Fermi-2	Reviewed by	Date .14.81
Proj. No. 6139-38 Equip. No.	Approved by	Date

CHECK Col Row F



COLUMN ROW-F

Column Row F is modeled as spring no. 50604
spring load from output (p.120) = 414.1 k

DETERMINE SHEAR IN ROW @ EACH ELEVATION

	BAY 9-10	BAY 15-17
SHEAR @ El. 684'-6	$\frac{26.3}{39.7}(582^k) = 386^k$	$\frac{26.3}{39.7}(512^k) = 339^k$
SHEAR @ El. 714'-4	386^k	339^k
SHEAR @ El. 735'-7	$\frac{26.3}{33.0}(334) = 266^k$	$\frac{26.3}{33.0}(426^k) = 339^k$

Reduce spring load to working stress level for comparing

$414.1/1.6 = 259^k$

SARGENT & LUNDY ENGINEERS CHICAGO	Calcs. For R/Aux. Bldg. Superstructure Steel for Seismic Reevaluation for New Response Spectra	Calc. No. SS0001
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Client DECo	Prepared by	Date 4-23-81
Project Fermi-2	Reviewed by M. Khazayeta	Date 5/12/81
Proj. No. 6135-38 Equip. No.	Approved by	Date

Check Col. Row F con't

Compare spring load with design shear loads
capacity of any one bay is larger than spring load, as well as the entire capacity is much greater than spring load

1 bay min. load = 266^k OK
min. Row A capacity = 266^k + 339^k = 605^k OK

Since horizontal components of vertical bracings are larger than seismic shears, then the vertical components are larger. Therefore, the columns & anchor bolts are OK

Therefore the column Row F columns, anchor bolts, beams, conn's & bracings are OK for new seismic loads.

Summary

Col. Row	Spring No.	Spring load	min. design shear of 1 bay @ any level	min design shear of Row @ any level	Remarks
9	50601	226^k	255^k	651^k	
17	50603	224^k	242^k	521^k	
A	50602	241^k	261^k	599^k	
F	50604	259^k	266^k	605^k	

By inspection it can be seen that the column Rows are designed for much larger horizontal loads and that any one bay of vert. bracing is capable of resisting the entire shear of the Row as obtained from the spring model. Therefore, the superstructure col. Rows should be OK for the higher seismic shears.

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SARGENT & LUNDY ENGINEERS CHICAGO	Calcs. For Edition Bldg. Superstructure Steel for Seismic Requalification for New Reactor ...	Calc. No. SS 0001
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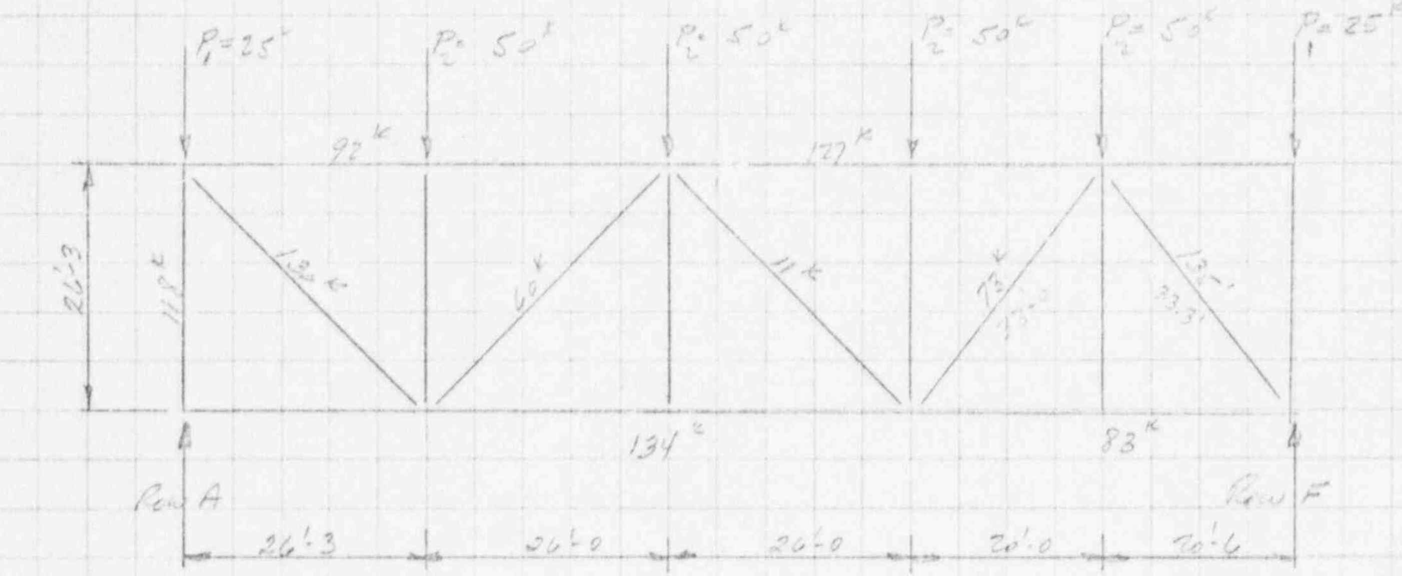
		Page 11 of 31

Client DECo	Prepared by J. Dean — Date 4.25.91
Project Fermi - 2	Reviewed by M. Kanyotha — Date 5.12.91
Proj. No. 5127-38 Equip. No.	Approved by — Date

Check Roof Truss N-S direction loading



$R_1 = 121^k$ (from spring) — (from spring) $R_2 = 130^k$

$R_1 = 118^k$ (calc.) — (calculated) $R_2 = 132^k$

Row A (SPRING NO. 50602)

SHEAR FORCE $= 385^k \Rightarrow$ v.s. $= \frac{385}{1.6} = 241/2 = 121^k$

Row F (SPRING NO. 50604)

SHEAR FORCE $= 414^k \Rightarrow$ v.s. $= 414/1.6 = 259^k/2 = 130^k$

Disregard lower Roof Truss - assume it does not act.

Assume 2 Roof trusses in each direction resist ½ the load

Assume $P_1 = \frac{P_2}{2}$ for approximation of loading

$R_1 + R_2 = 241/2^k + 259/2^k = 250^k$

$2P_1 + 4P_2 = 2\left(\frac{P_2}{2}\right) + 4P_2 = 5P_2 = 250^k$

$P_2 = 50^k$

$P_1 = 50/2 = 25^k$

$R_1 = 25 + \frac{20.5(50) + 40.5(50) + 66.5(50) + 92.5(50)}{26.25 + 2\times 26 + 20 + 20.5} = 118^k$

$118^k \approx 121^k$ from computer, therefore, approximation is ok

Form GQ-3.08.1 Rev. 2

Check Roof Truss N-S direction loading

DESIGN loads from struc drwg's shown on top calculated shown on bottom

By inspection & comparison it can be that the truss members are OK

The diagonals @ the ends are slightly overstressed but since there are trusses at these panels in the opposite direction this truss will actually span between the two opposite direction trusses & they will take the reaction. Therefore, it seems that the tornado loads the trusses were originally designed for still seem to govern. The original analysis modeled the upper & lower trusses in the computer and applied the loads at the joints which accounted for a frame type action.

SARGENT & LUNDY ENGINEERS CHICAGO	Calcs. For R/Aux Bldg Superstruc. Steel for [illegible]	Calc. No. [illegible]
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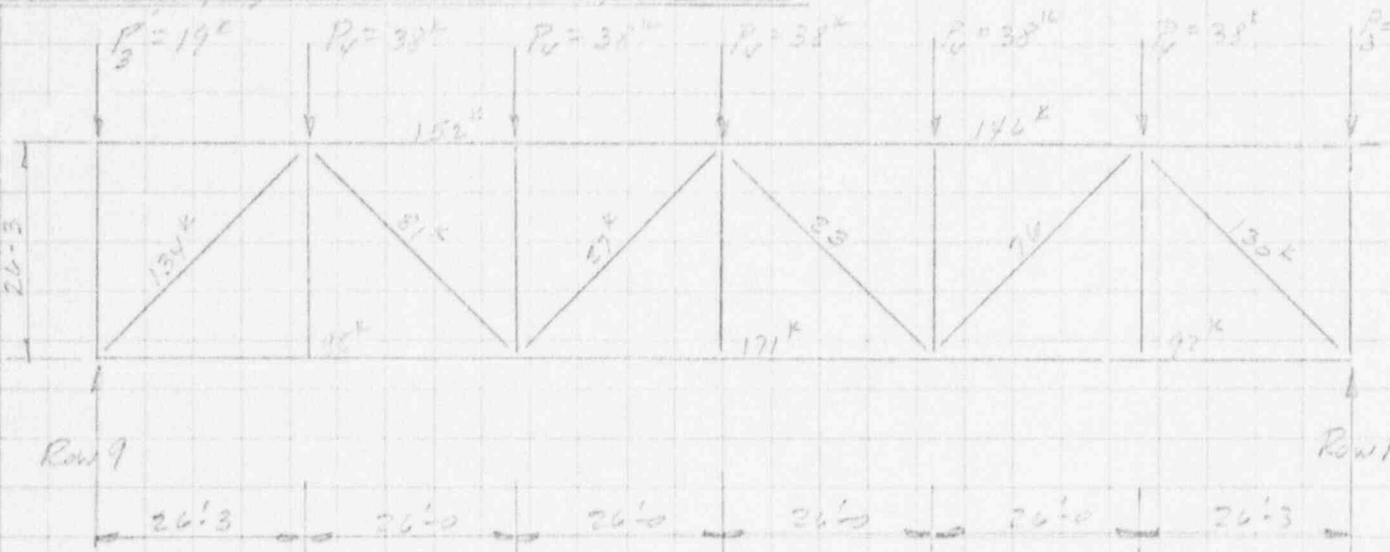
Client DECo		Prepared by [illegible]	Date [illegible]-81
Project Fermi - 2		Reviewed by M. Khayyate	Date [illegible]-81
Proj. No. 6135-38	Equip. No.	Approved by	Date

CHECK ROOF TRUSS E-W direction loading



$R_3 = 113^k$ (from computer)

$R_3 = 114^k$ (calculated)

(from computer) $R_4 = 112^k$

(calculated) $R_4 = 111^k$

Row 9 (spring no. 50601)

SHEAR FORCE $= 361.8^k \Rightarrow$ W.S. $= \frac{361.8}{1.6} = 226^k/2 = 113^k$

Row 17 (spring no. 50603)

SHEAR FORCE $= 357.6^k \Rightarrow$ W.S. $= \frac{357.6}{1.6} = 224^k/2 = 112^k$

Disregard lower roof truss - Assume it does not act

Assume 2 Roof trusses in each direction resist ½ the load

Assume $P_3 = P_4/2$ for approx loading

$R_3 + R_4 = 113 + 112 = 225^k$

$2P_3 + 5P_4 = 2\left(\frac{P_4}{2}\right) + 5P_4 = 6P_4 = 225^k$

$P_4 = 38^k$

$P_3 = 38/2 = 19^k$

$R_3 = 19 + \dfrac{38\,(26.25 + 52.25 + 78.25 + 104.25 + 130.25)}{26.25 + 4\times26 + 26.25} = 114^k$ $R_4 = 225 - 114 = 111$

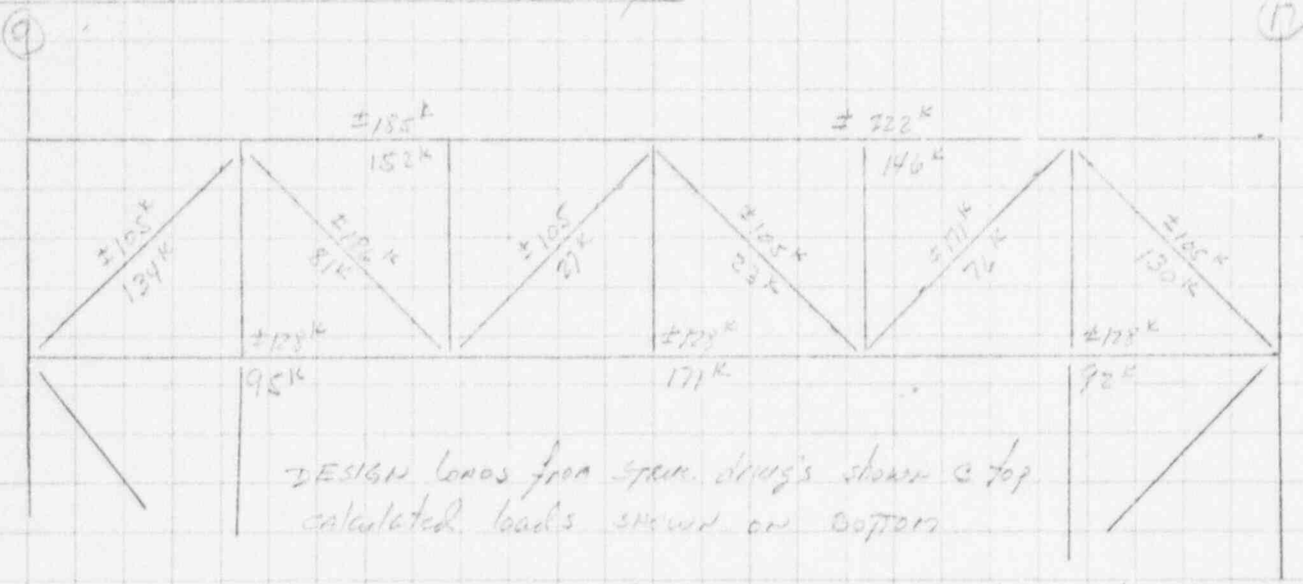
Approximation is ok

Form GO.3.08.1 Rev. 2

SARGENT & LUNDY ENGINEERS CHICAGO	Calcs. For Reactor Bldg. Superstruc. Steel for Seismic Reevaluation for New Response Spectra	Calc. No. SS 0001
	X Safety-Related / Non-Safety-Related	Rev. 0 Date
		Page 17 of 39

Client DECo	Prepared by J. Dinn	Date 4-28-81
Project Fermi-2	Reviewed by N. Khanyata	Date 5.16.81
Proj. No. 5128-38 Equip. No.	Approved by	Date

Check Roof Truss E-W Direction loading



DESIGN loads from steel drwg's shown @ top
calculated loads shown on bottom

By inspection + comparison it can be seen that the truss members are OK

The diagonals @ the ends are part of a truss system in the opposite direction and even tho they are slightly overstressed it is OK since the E-W truss can actually span between the two end trusses and the E-W truss Reactions will be taken by the opposite spaning trusses. This same logic will reduce the chord load in the lower chord.

Therefore, the truss should be OK.

The truss along the EAST side of the Roof should also be OK as can be seen by comparing the design loads shown on the drwg's - they approx. the same when the difference of angle is considered for the diagonals due to the truss being slightly shallower. Therefore, the original tornado design can be seen to still govern.

Summary

IT CAN be seen from calc. book 4.02.08, part 1, page titled "360 mph Tornado - siding blown-off", that the tornado loads are larger than the seismic loads calculated above. Therefore, OK

Form GQ-3.08.1 Rev. 2

SARGENT & LUNDY ENGINEERS CHICAGO	Calcs. For R/Hum Bldg Superstruc. Steel for Seismic Reevaluation for New Response Spectra		Calc. No. SS-002	
	✓ Safety-Related	Non-Safety-Related	Rev. 0	Date
			Page 16 of 27	

Client DECo	Prepared by J. Owen	Date 4-29-81
Project Fermi - 2	Reviewed by M. Khurgustan	Date 5-14-81
Proj. No. 6735-38 Equip. No.	Approved by	Date

Summary cont

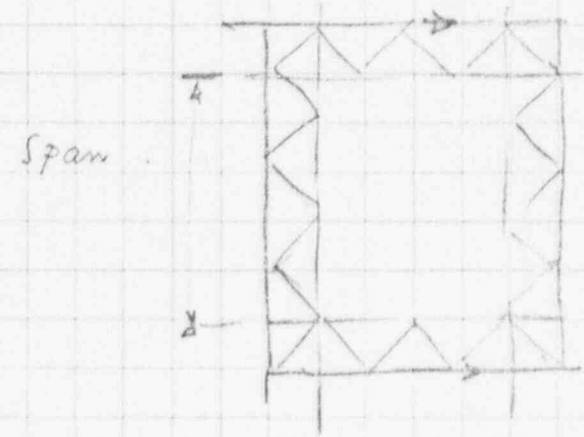
Also see calc book no. 4.02.08, part 1, page showing "probable maximum shear force between slab 5 and slab 6." From this page of calc's it can be seen that the seismic shears originally calculated are greater than the current shears.

Also see calc book no. 4.02.08, part 2, page showing "SEISMIC FORCES FOR UNIT 2 & 3".

APPROVER'S COMMENT:

EVEN THOUGH THE DESIGN SHEARS DUE TO TORNADO WERE HIGHER THAN REEVALUATION SHEARS DUE TO NEW SEISMIC THE SKETCH ON PAGE 17 SHOWS FORCE IN SOME MEMBERS HIGHER DUE TO PRESENT SEISMIC FORCES.

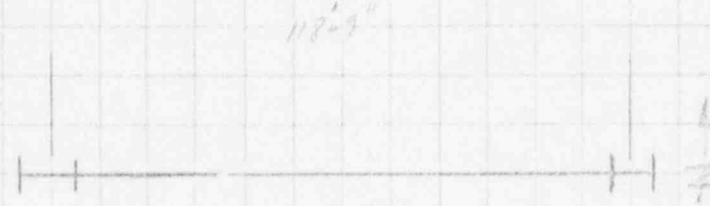
THE ORIGINAL FORCES WERE BASED ON A COMPLETE STRUDL MODEL OF HORIZONTAL TRUSS SYSTEM E-W & N-S Combined WHICH SHOWED THAT THE TRUSS SPANS ARE REALLY BETWEEN PERPENDICULAR TRUSSES RATHER THAN OVERALL SPANS TO THE EXTERNAL COL. ROWS AS TAKEN IN THESE COMPUTATIONS.



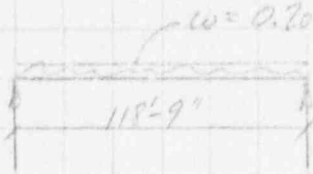
SARGENT & LUNDY ENGINEERS CHICAGO	Calcs. For Rx/Aux Bldg Superstruc. steel for seismic Reevaluation for new Response Spectra	Calc. No. SS0221
	X Safety-Related / Non-Safety-Related	Rev. 2 Date
		Page 19 of 29

Client DECo	Prepared by [illegible]	Date 5-7-81
Project Fermi - 2	Reviewed by [illegible]	Date 5-14-81
Proj. No. 6139-38 Equip. No.	Approved by	Date

CHECK GIRDER RBG 34

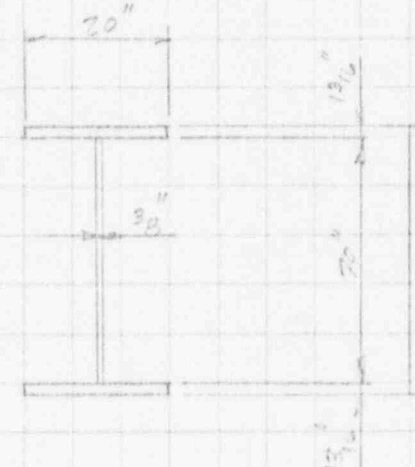


VERTICAL

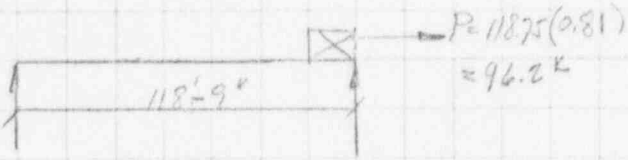


$$f_n = \frac{3.55}{\sqrt{\frac{5 w L^4}{384 E I_x}}}$$

$$= \frac{3.55}{\sqrt{\frac{5(0.81/12)(12 \times 118.75)^4}{384(29,000)(51,462.6)}}} = 2.3 \text{ cps}$$



Horizontal (axial)



$$A = 2\left[20 \times \tfrac{13}{16}\right] + 70\left(\tfrac{3}{8}\right) = 58.75 \text{ in}^2$$

$$I_x = \frac{(3/8)(70)^3}{12} + \frac{20\left[(70 + 2 \times \frac{13}{16})^3 - (70)^3\right]}{12}$$

$$= 51,462.6 \text{ in}^4$$

$$f_n = \frac{1}{2\pi}\sqrt{\frac{AEg}{PL}}$$

$$= \frac{1}{2\pi}\sqrt{\frac{(58.75)(29,000)(32.2 \times 12)}{(96.2)(118.75 \times 12)}}$$

$$= 11.0 \text{ cps}$$

$$I_y = 2\left[\frac{(13/16)(20)^3}{12}\right] + \frac{70(3/8)^3}{12}$$

$$= 1083.6$$

see next page for Horiz (weak axis)

weight = 2(55.3) + 89.3 = 199.9 #/'

19 pairs of stiff's pl 5/16" × 4" × 70" = 19($\frac{5}{12}$ × 74.4) = 942.4#

942.4/118.75' = 7.9 #/'

TOTAL WT. = 199.9 + 7.9

= 207.8 #/'

Assume REACTION of W12×65 as AN REM. & to account for lower steel framing

(17) beams (8' × 0.1625 k/') = 71.8 k/118.75' = [illegible]

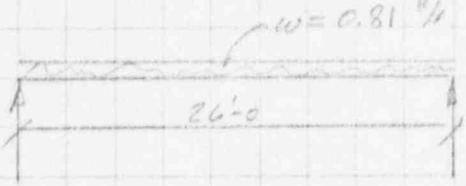
SARGENT & LUNDY ENGINEERS CHICAGO	Calcs For R/Aux Bldg Superstructure Steel for Seismic Reevaluation for New Response Spectra	Calc. No. SS 0001
	X Safety–Related / Non–Safety–Related	Rev. 2 Date
		Page 80 of 97

Client DECo	Prepared by J. Davis	Date 5-7-81
Project Fermi-2	Reviewed by M. Khazgolen	Date 5-14-81
Proj. No. 6139-38 Equip. No.	Approved by	Date

CHECK GIRDER RB6 34 con't

HORIZONTAL (weak axis)

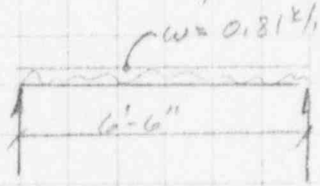
consider girder braced @ truss panel pts. (col rows) only



$$f_n = \frac{3.55}{\sqrt{\frac{5(0.81/12)(12\times 26)^4}{384(29,000)(1083.6)}}}$$

$= 6.9$ cps

consider girder braced @ purlin pts.



$$f_n = \frac{3.55}{\sqrt{\frac{5(0.81/12)(12\times 6.5)^4}{384(29,000)(1083.6)}}}$$

$= 110.3$ cps

Form GQ.3.08.1 Rev. 2

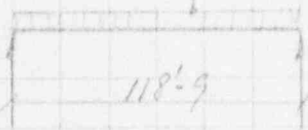
SARGENT & LUNDY ENGINEERS CHICAGO	Calcs. For Reflux Bldg. Superstruc. Steel for Seismic Reevaluation for new Response Spectra	Calc. No. SS-0001
	X Safety–Related / Non–Safety–Related	Rev. 0 Date
		Page 21 of 39

Client DECo	Prepared by [illegible]	Date 5-7-81
Project Fermi – 2	Reviewed by [illegible]	Date 9-14-81
Proj. No. 6135-38 Equip. No.	Approved by	Date

CHECK GIRDER RBG 34 con't (50 ksi STEEL)

Vertical

$\omega = 0.81\ ^k/_l + 0.38(0.81) = 1.12\ ^k/_l$



if $f < 33$ cps then multiply $g \times 1.5$

$f_n = 2.3$ cps $\Rightarrow g = 0.25 \times 1.5 = 0.38$

$M_x = \dfrac{(1.12)(118.75)^2}{8} = 1974.2\ 'k$

$f_{bx} = \dfrac{12(1974.2)\left(\frac{71.25}{2}\right)}{51,462.6} = 16.5$ ksi

$\dfrac{f_{bx}}{F_{bx}} = \dfrac{16.5}{47.5} = 0.35$

$r_t = \sqrt{I/A} = \sqrt{\dfrac{(^{13}/_{16})(20)^3/12}{(^{13}/_{16})(20)}} = 5.8''$

$l/r_t = 12 \times 26 / 5.8 = 53.8$

$F_b = 33.3 - \dfrac{(53.8)^2}{612} = 28.6$ ksi

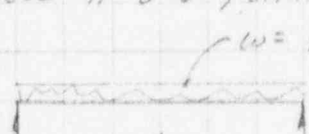
$F_b = \dfrac{12 \times 10^3}{12 \times 26 \times \frac{71.25}{20 \times \frac{13}{16}}} = 8.7$ ksi

$F_{all} = 1.67 \times 28.6 = 47.7 < .95(50) = 47.5$

HORIZ. (WEAK AXIS)

N-S direction

$\omega = 3.0(0.81) = 2.4\ ^k/_l$



$f_n < 33$ cps $\Rightarrow g \times 1.5$

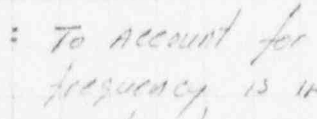
$f_n = 6.9$ cps $\Rightarrow g = 2.0 \times 1.5 = 3.0$

$M_y = \dfrac{(2.4)(26)^2}{8} = 202.8\ 'k$

$f_{by} = \dfrac{12(202.8)(10)}{1083.6} = 22.5$ ksi

CHECK IF 6'-6 purlin spacing is assumed as braced pt.

$\omega = 1.2(0.81) = 0.97\ ^k/_l$



$f_n = 110.3$ cps $\Rightarrow g = 1.2$

$M_y = \dfrac{(0.97)(6.5)^2}{8} = 5.1\ 'k$

$f_{by} = \dfrac{12(5.1)(10)}{1083.6} = 0.57$ ksi

NOTE: To account for higher modes & amplification effects when the frequency is in the flexible zone (ie. less than 33 cps) the acceleration will be multipled by 1.5. This is an approximate value only for use in expediting the survey.

Form GQ-3.08.1 Rev. 2

SARGENT & LUNDY ENGINEERS CHICAGO

Calcs. For Reactor Bldg Superstruc. Steel for Seismic Reevaluation for New Response Spectra

X Safety-Related | Non-Safety-Related

Calc. No. SS-0201 | Rev. 2 | Date

Client DECo

Project Fermi - 2

Proj. No. 6129-38 Equip. No.

Prepared by | Date 5-7-81

Reviewed by | Date 5/14/81

Approved by | Date

CHECK Girder RBG 34 con't

Horiz. (axial)

N-S direction

$P \rightarrow$ $P = 2.1\,(96.2^k) = 202.0^k$

$118'$-9

unbraced for 26'-0

$f_n < 33\,cps \Rightarrow g \times 1.5$

$f_n = 11.6\ cps \Rightarrow g = 1.4 \times 1.5 = 2.1$

$f_a = 202.2/58.75 = 3.4\ ksi$

Note: Since no response spectra was generated for the roof elevation, the response spectra at the crane elevation has been used and has been scaled down by comparing the acceleration values of the nodes at each elevation. This is typ. for all members checked.

$r_y = \sqrt{I/A} = \sqrt{\frac{1083.6}{58.75}} = 4.3''$

$r_x = \sqrt{I/A} = \sqrt{\frac{51246.6}{58.75}} = 29.6$

$l/r_y = \frac{12 \times 26}{4.3} = 72.3$

$l/r_x = \frac{12 \times 118.75}{29.6} = 48.1$

$l/r_y = 72.3 \Rightarrow F_a = 20.5\,(1.6) = 32.8$

CHECK for seismic w/o Truss Action

$$\frac{f_a}{F_a} + \frac{f_{bx}}{F_{bx}} + \frac{f_{by}}{F_{by}} < 1.0$$

$$\frac{3.4}{32.8} + \frac{16.5}{47.5} + \frac{22.5}{47.5} = 0.104 + 0.347 + 0.474 = 0.925$$

Form GQ-3.08.1 Rev. 2

SARGENT & LUNDY ENGINEERS CHICAGO

Calcs. For Refuel Bldg Superstruc. Steel for Seismic Reevaluation for New Response Spectra	Calc. No. SS-0001	
X Safety-Related / Non-Safety-Related	Rev. 0	Date
	Page 23 of 37	

Client DE Co	Prepared by [illegible]	Date 5-7-87
Project Fermi-2	Reviewed by M. Khaypeth	Date 5-14-87
Proj. No. 6138-38 Equip. No.	Approved by	Date

Check Girder RB6 con't

CHECK for seismic w/ Truss Action

seismic in N-S direction loading

truss spans E-W

from previous pages max axial load = 134^k (required)

$$\times 1.6$$

$$214.4^k$$

$$\therefore f_a = \frac{214.4}{58.75} = 3.6 \text{ ksi}$$

$$\frac{f_a}{F_a} + \frac{f_{bx}}{F_{bx}} + \frac{f_{by}}{F_{by}} \leq 1.0$$

$$\frac{(3.6+3.4)}{32.8} + \frac{16.5}{47.5} + \frac{22.5}{47.5} \leq 1.0 \qquad \frac{f_a}{F_a} \approx 0.15$$

$$0.213 + 0.347 + 0.474 = 1.03 \approx 1.0 \quad \text{ok}$$

ALTHO TRUSS IS PART OF BOTH E-W & N-S TRUSSES WE HAVE used the load from the bending of the E-W spanning truss. As stated previously this truss spans from truss to truss NOT col. Row to col. Row. Therefore, the axial load in the girder due E-W truss bending will be less but an additional amount of axial load from the N-S spanning truss will be added since the N-S spanning truss will also span from E-W truss to E-W truss. Therefore, we have used the the axial load from the E-W spanning truss as an approximation of the total axial load

Form GQ-3.08.1 Rev. 2

SARGENT & LUNDY ENGINEERS CHICAGO	Calcs. For R/Aux Bldg Superstruc. Steel Frame Reevaluation for New Response Spectra	Calc. No.	Rev.	Date	Page of
	✓ Safety–Related / Non–Safety–Related				

Client DECo		Prepared by	Date 5-8-81
Project Fermi - 2		Reviewed by H. Khazzaghi	Date
Proj. No. 4435-31	Equip. No.	Approved by	Date

Calculat frequency of members

Use SDED REPORT No. 50, "NATURAL FREQUENCIES OF BEAMS & SLABS" by H. Sindu & H. Singh to determine natural frequencies REV. 1 (1-2-80)

Roof Purlin W12X27

metal deck + insulation
DL = 15 psf

Vertical

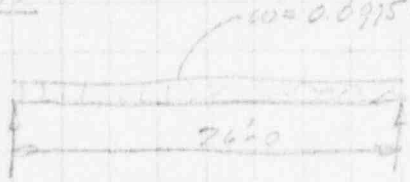
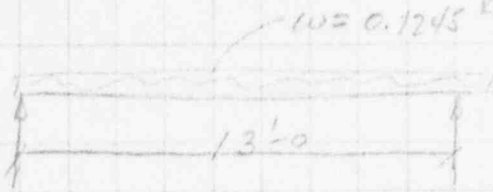


table value = 2.69 — (SDED Report No. 50)

$$f_n = \frac{2.69}{\sqrt{0.1245}} = 7.6 \text{ cps}$$

Horiz. E-W (weak axis)



contributory area = 6'-6"
Roof DL = (6.5)(15) = 97.5 #/l

table value = 3.23

$$f_n = \frac{3.23}{\sqrt{0.1245}} = 9.2 \text{ cps}$$

Horiz N-S (axial)

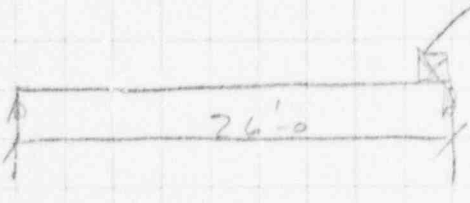


table value = 85.13

$$f_n = \frac{85.13}{\sqrt{3.24}} = 47.3 \text{ cps}$$

Form GQ-3.08.1 Rev. 2

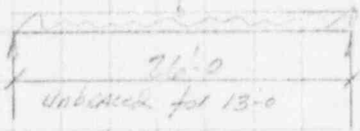
SARGENT & LUNDY ENGINEERS CHICAGO

Calcs. For Rx/Aux. Bldg. Superstructure Steel for Seismic Reevaluation for New Response Spectra		Calc. No. [illegible]
X Safety-Related	Non-Safety-Related	Rev. 0 Date
		Page 65 of [illegible]

Client DECo	Prepared by J. Daum	Date 5-7-81
Project Fermi-2	Reviewed by M. [illegible]	Date 5-18-81
Proj. No. 6139-38 Equip. No.	Approved by	Date

Roof Purlin W12x27 con't

Vertical



$w = 0.1245^{k/l} + 0.9(0.1245) = 0.24$

$f_n < 33\,cps \Rightarrow g \times 1.5$

$f_n = 7.6\,cps \Rightarrow g = 0.6 \times 1.5 = 0.9$

$\frac{f_{b_x}}{F_{b_x}} = \frac{7.1}{27.5} = 0.258$

$M = \frac{(0.24)(26)^2}{8} = 20.3'^k$

$f_{b_x} = \frac{12 \times 20.3}{34.2} = 7.1\,ksi$

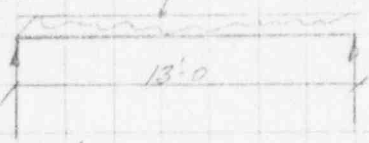
$l/r_t = \frac{12 \times 13}{1.74} = 89.7$

$F_{b_x} = 24 - \frac{(89.7)^2}{1181} = 17.2\,ksi \times 1.6 = 27.5$

$F_{b_x} = \frac{12 \times 10^3}{12 \times 13 \times 4.6} = 16.7$

Horizontal (weak axis)

E-W



$w = (0.1245)2.3 = 0.29^{k/l}$

$f_n = 9.2\,cps \rightarrow g = 1.5 \times 1.5 = 2.3$

$M = \frac{(0.29)(13)^2}{8} = 6.1'^k$

$f_{b_y} = \frac{12 \times 6.1}{5.63} = 13.1\,ksi$

$F_{b_y} = 0.95 \times 36 = 34.2\,ksi$

Form GC-3.08.1 Rev. 2

SARGENT & LUNDY ENGINEERS CHICAGO

Calcs. For Reactor Bldg Superstruc. Steel for Seismic Reevaluation for New Response Spectra		Calc. No.
x Safety-Related	Non-Safety-Related	Rev. 0 Date
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Client DECo	Prepared by G. Dinas	Date 5.9.81
Project Fermi - 2	Reviewed by H. Khuzyite	Date 5.14.81
Proj. No. 6139-38 Equip. No.	Approved by	Date

Roof Purlin W12x27 cont'd

Horiz (axial)

N-S

$P = (3.24^k)1.2 = 3.9^k$

26'-0

considered for 13-0

$f_n = 47.3$ cps $\Rightarrow g = 1.2$

$f_a = 3.9/7.95 = 0.49$ ksi

$\frac{l}{r_y} = \frac{12 \times 13}{1.52} = 102.6$

$\frac{l}{r_x} = \frac{12 \times 26}{5.07} = 61.5$

$F_a = 12.65$ ksi $\times 1.6 = 20.2$ ksi

$$\frac{f_a}{F_a} + \frac{f_{bx}}{F_{bx}} + \frac{f_y}{F_{by}} \leq 1.0$$

$$\frac{0.49}{20.2} + \frac{7.1}{27.5} + \frac{13.1}{34.2} \leq 1.0 \qquad 0.036\ \frac{f_a}{F_a} < 0.15$$

$$0.034 + 0.215 + 0.351 = 0.60 < 1.0 \quad \text{OK}$$

Form GO-3.08.1 Rev. 2

SARGENT & LUNDY ENGINEERS CHICAGO	Calcs. For Rx/Aux Bldg. Superstruc. Steel For Seismic Reevaluation for New Reverse Loading	Calc. No. S-2501
	× Safety-Related / Non-Safety-Related	Rev. 0 / Date
		Page 27 of 37

Client DECo	Prepared by D. Owen	Date 5-7-81
Project Fermi - 2	Reviewed by M. Khosrow-shahi	Date 5-15-81
Proj. No. 6139-30 Equip. No.	Approved by	Date

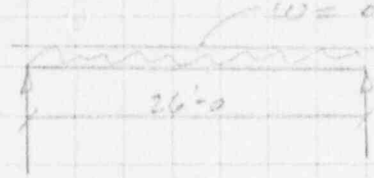
Roof Purlin W12×65

metal deck + insulation

DL = 15 psf

VERTICAL

$w = 0.0975 + 0.065 = 0.1625\ ^{k}/_{1}$



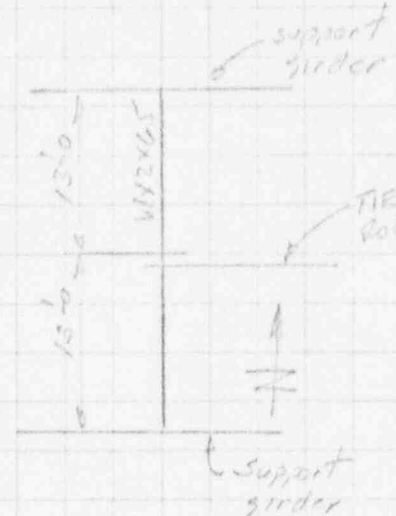


table value = 4.35

$$f_n = \frac{4.35}{\sqrt{0.1625}} = 10.8 \text{ cps}$$

HORIZ. E-W (WEAK AXIS)

contributory area = 6'-6"

$DL = 6.5(15) = 97.5\ ^{\#}/_{1}$

$w = 0.1625\ ^{k}/_{1}$

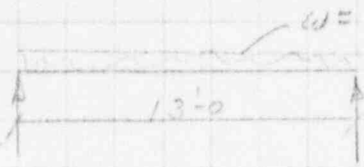


table value = 9.98

$$f_n = \frac{9.98}{\sqrt{0.1625}} = 24.8 \text{ cps}$$

HORIZ. N-S (axial)

$P = 26'(0.1625\ ^{k}/_{1}) = 4.23^{k}$

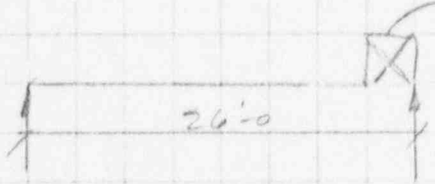


table value = 131.95

$$f_n = \frac{131.95}{\sqrt{4.23}} = 64.2 \text{ cps}$$

Form GO-3.08.1 Rev. 2

SARGENT & LUNDY ENGINEERS CHICAGO	Calcs. For R. Wax Bldg Superstruc. Steel for Seismic Reevaluation for new Process System	Calc. No. 23.0001
	X Safety-Related / Non-Safety-Related	Rev. 2 Date
		Page 29 of 39

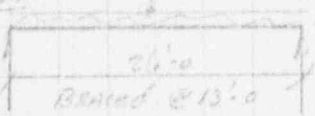
Client Deco	Prepared by J. Dinner	Date 5-5-81
Project Fermi-2	Reviewed by M. Khazzoli	Date 5-15-81
Proj. No. 4135-38 Equip. No.	Approved by	Date

Roof Purlin W12x65 cont

Vertical

down

$\omega = 0.1625 + 2.1(0.1625) = 0.51$ k/'



$f_n < 33$ cps $\Rightarrow$ g x 1.5

$f_n = 10.8$ cps $\Rightarrow g = 1.35 \times 1.5 = 2.1$

$\frac{f_b}{F_b} = 5.9/34.2 = 0.17$

$M_x = \frac{(0.51)(26)^2}{8} = 43.1$ 'k

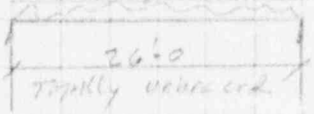
$f_{b_x} = \frac{12(43.1)}{88.0} = 5.9$ ksi

$F_{b_x} = 0.6(36)(1.6) = 34.6$

$F_{b_x} = 0.95(36) = 34.2$ ksi

up

$\omega = 0.1625 - 2.1(0.1625) = 0.18$ k/'



$M_x = \frac{(0.18)(26)^2}{8} = 15.2$ 'k

$f_b = \frac{12(15.2)}{88.0} = 2.1$ ksi

$\frac{l}{r_y} = \frac{12(26)}{3.31} = 94.3$

$F_b = 24 - \frac{(94.3)^2}{1144} = 16.5$

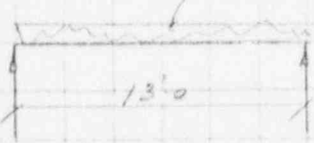
$F_b = \frac{12 \times 10^3}{12 \times 26 \times 1.0?} = 23.0 \Rightarrow F_b = .6(36) = 21.6$

$F_b = 0.95(36) = 34.2$ ksi

$\frac{f_b}{F_b} = \frac{2.1}{34.2} = 0.06$

Horizontal (weak axis)

$\omega = 2.0(0.1625) = 0.33$ k/'



$f_n < 33$ cps $\Rightarrow$ g x 1.5

$f_n = 24.8$ cps $\Rightarrow g = 1.3 \times 1.5 = 2.0$

$M_y = \frac{(0.33)(13)^2}{8} = 7.0$ 'k

$f_{b_y} = \frac{12(7.0)}{29.1} = 2.9$ ksi

$F_{b_y} = 0.95(36) = 34.2$

Form GQ 3.08.1 Rev. 2

SARGENT & LUNDY ENGINEERS CHICAGO	Calcs. For Reftor Bldg. Superstructure Steel for Seismic Reevaluation for New Response Spectra	Calc. No. [illegible]221
	☒ Safety–Related ☐ Non–Safety–Related	Rev. 2 Date
		Page 29 of 39

Client DECo	Prepared by G. Wrum	Date 5-9-81
Project Fermi - 2	Reviewed by M. Khaypal	Date 5-15-81
Proj. No. 4139-38 Equip. No.	Approved by	Date

Roof Purlin W12x65 con't

Horizontal (axial)

$f_n = 131.55$ cps $\Rightarrow g = 1.2$

$f_a = 5.1/19.1 = 0.27$ ksi

$l/r_y = \frac{12 \times 13}{3.02} = 51.7$

$l/r_x = \frac{12 \times 26}{5.28} = 59.1 \Rightarrow F_a = 17.5$ ksi

$F_a = 1.6(17.5) = 28.0$ ksi

$$\frac{f_a}{F_a} + \frac{f_{bx}}{F_{bx}} + \frac{f_{by}}{F_{by}} < 1.0$$

$$\frac{0.27}{28.0} + \frac{5.9}{34.2} + \frac{2.9}{34.2} < 1.0$$

$0.027 = f_a/F_a < 0.15$

$0.010 + 0.173 + 0.083 = 0.266 < 1.0$ OK

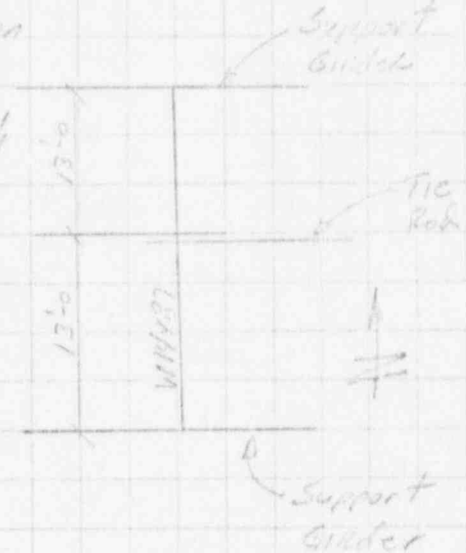
By inspection axial load from truss will increase axial stress but will still be OK

SARGENT & LUNDY ENGINEERS CHICAGO	Calcs. For Reactor Bldg. Superstructure Steel for Seismic Reevaluation for New Response Spectra	Calc. No. SS-221
	X Safety-Related / Non-Safety-Related	Rev. 0 Date
		Page 30 of 39

Client DECo		Prepared by J. Wang	Date 5-3-81
Project Fermi-2		Reviewed by M. Khayyate	Date 5-15-81
Proj. No. 6175-38	Equip. No.	Approved by	Date

Roof Purlin W 14×87

metal deck + insulation
DL = 15 psf



contributory area = 6'-6
DL = 6.5 (15) = 97.5 #/'

Vertical

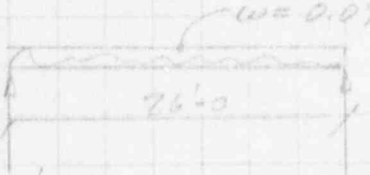


table value = 5.86

$$f_n = \frac{5.86}{\sqrt{0.1845}} = 13.6 \text{ cps}$$

Horiz. E-W (weak axis)

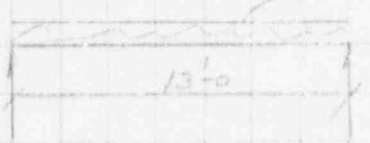


table value = 14.11

$$f_n = \frac{14.11}{\sqrt{0.1845}} = 32.8 \text{ cps}$$

Horiz N-S (axial)

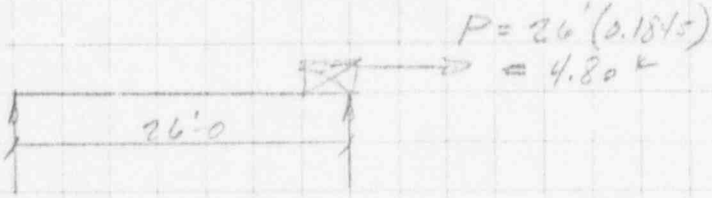


table value = 152.76

$$f_n = \frac{152.76}{\sqrt{4.80}} = 69.7 \text{ cps.}$$

Form GQ.3.08.1 Rev. 2

SARGENT & LUNDY ENGINEERS CHICAGO	Calcs. For Reactor Bldg Superstr. Steel For Seismic Reevaluation for New Response Spectra	Calc. No.	Rev. 0	Date	Page 31 of 39
	X Safety-Related / Non-Safety-Related				

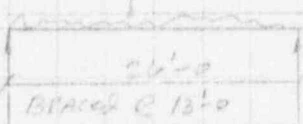
Client DECo	Prepared by	Date 5-?-81
Project Fermi-2	Reviewed by M.K.	Date 5.18.81
Proj. No. 6135-38 Equip. No.	Approved by	Date

Roof Purlin W 14×37 con't

VERTICAL

down

$w = 0.1845 + 2.55(0.1845) = 0.66$



$f_n < 33\text{ cps} \Rightarrow g \times 1.5$

$f_n = 13.6\text{ cps} \Rightarrow g = 1.7 \times 1.5 = 2.55$

$M_x = \frac{(0.66)(26)^2}{8} = 55.8'k$

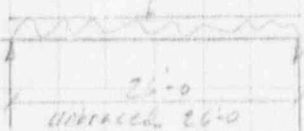
$f_{b_x} = \frac{12(55.8)}{138} = 4.9\text{ ksi}$

$F_{b_x} = 24\text{ ksi} \times 1.6 = 38.4$

$\rightarrow F_{b_x} = .95(36) = 34.2$

UP

$w = 0.1845 - 2.55(0.1845) = 0.29$



$M_x = \frac{(0.29)(26)^2}{8} = 24.5'k$

$f_{b_x} = \frac{12 \times 24.5}{138} = 2.1\text{ ksi}$

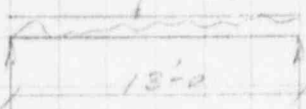
$\frac{f_{bx}}{F_{bx}} = \frac{4.9}{34.2} = 0.143$

$F_{b_x} = 21.6 \times 1.6 = 34.6$

$\rightarrow F_{b_x} = .95(36) = 34.2\text{ ksi}$

HORIZONTAL (weak axis)

$w = (0.1845)2.0 = 0.37^{k/'}$



$f_n < 33\text{ cps} \Rightarrow g \times 1.5$

$f_n = 32.8\text{ cps} \Rightarrow g = 1.3 \times 1.5 = 2.0$

$M_y = \frac{(0.37)(13)^2}{8} = 7.8'k$

$f_{b_y} = \frac{12(7.8)}{48.2} = 2.0\text{ ksi}$

$F_{b_y} = 0.95(36) = 34.2\text{ ksi}$

Form GQ-3.08.1 Rev. 2

SARGENT & LUNDY ENGINEERS CHICAGO	Calcs. For Reactor Bldg. Superstructure Steel For Seismic Evaluation for New Response Spectra	Calc. No. SS-0??1
	X Safety-Related / Non-Safety-Related	Rev. 0 Date
		Page 32 of 79

Client DECo	Prepared by C.J. ... Date 5-18-81
Project Fermi-2	Reviewed by M. Kennedy Date 5-18-81
Proj. No. 6135-38 Equip. No.	Approved by Date

Roof Purlin W14×87 cont

Horizontal (axial)

$f_a = 68.7$ cps $\Rightarrow g = 1.2$

$f_a = 5.8 / 25.6 = 0.23$

$\ell / r_y = \frac{12 \times 13}{3.72} = 42.2$

$\ell / r_x = \frac{12 \times 26}{6.15} = 50.7$

$F_a = 18.3 \times 1.6 = 29.3$ ksi

$f_a = 0.23$

CHECK w/o TRUSS action

$f_a / F_a = \frac{0.23}{29.3} = 0.008$

$f_a / F_a + \frac{f_{bx}}{F_{bx}} + \frac{f_{by}}{F_{by}} \le 1.0$

$\frac{0.23}{29.3} + \frac{4.9}{34.2} + \frac{2.0}{34.2} \le 1.0$

$0.008 + 0.143 + 0.058 = 0.209 < 1.0$ OK

CHECK w/ TRUSS action

TRUSS load $= 134^k \times 1.6 = 214^k$ $\quad f_a = \frac{214}{25.6} = 8.4$ ksi

$\frac{(8.4 + 0.23)}{29.3} = 0.295$

$\frac{(5.2 + 0.23)}{34.2} + \frac{4.9}{34.2} + \frac{2.0}{34.2} \le 1.0$ OK

$\frac{(5.2 + 0.23)}{29.3} + \frac{4.9}{(1 - \frac{5.43}{57.41})34.2} + \frac{2.0}{(1 - \frac{5.43}{84.65})34.2} \le 1.0$ OK

$0.159 + 0.143 + 0.058 = 0.360 < 1.0$ $\quad 0.185 + 0.158 + 0.062 = 0.405 \le 1.0$

See the note about approximating axial load from truss action that is with calc's for check of RB6 34 Roof girder

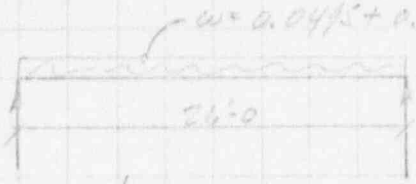
SARGENT & LUNDY ENGINEERS CHICAGO	Calcs. For Turbine Bldg Superstructure Steel For Seismic Reevaluation for New Response Spectra	Calc. No. SS-220-1
	X Safety-Related / Non-Safety-Related	Rev. 2 Date
		Page 53 of 39

Client DECo	Prepared by G. Oraam	Date 5-3-88
Project Fermi-2	Reviewed by M. Khayyate	Date 5-15-88
Proj. No. 6235-38 Equip. No.	Approved by	Date

Roof Eave Purlin W14x119

Vertical

metal deck + insulation
DL = 15 psf



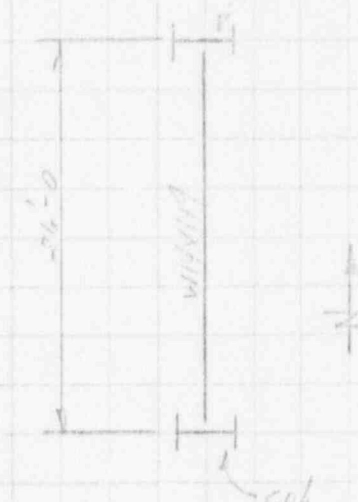


table value = 6.98

$$f_n = \frac{6.98}{\sqrt{0.6685}} = 8.5 \text{ cps}$$

Horiz. E-W (weak axis)

contributory area = $\frac{6'-6"}{2}$ = 3'-3"
3.3'(15) = 49.5 #/l

metal siding is also supported from W14x119 see B-157
DL = 50'(10 #/□') = 500 #/l

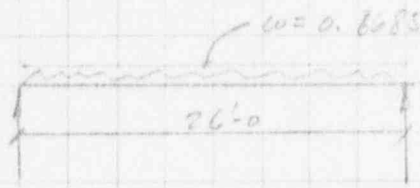


table value = 4.18

$$f_n = \frac{4.18}{\sqrt{0.6685}} = 5.1 \text{ cps}$$

Horiz. N-S (axial)

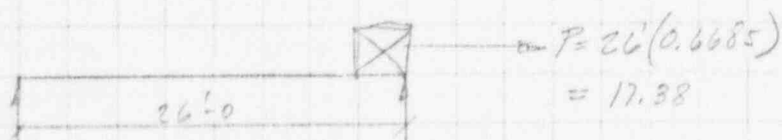


table value = 178.62

$$f_n = \frac{178.62}{\sqrt{17.38}} = 42.8 \text{ cps}$$

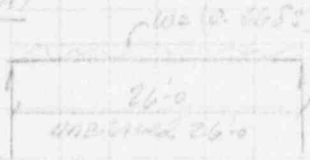
Form GQ-3.08.1 Rev. 2

SARGENT & LUNDY ENGINEERS CHICAGO	Calcs. For Reactor Bldg Superstructure Steel for Seismic Requalification for New Response Spectra	Calc. No. 5201
	X Safety–Related / Non–Safety–Related	Rev. 0 Date / Page 1 of 79

Client DECo	Prepared by [illegible] — Date 5-14-87
Project Fermi - 2	Reviewed by M. Khazzgate — Date 5-15-87
Proj. No. 6135-28 Equip. No.	Approved by — Date

Roof Eave Purlin W14x119

Vertical



$f_n < 33cps \Rightarrow g \times 1.5$

$f_n = 8.5cps \Rightarrow g = 0.8 \times 1.5 = 1.2$

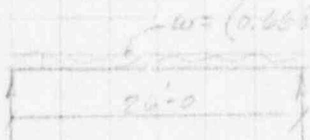
$M_x = \frac{(1.5)(26)^2}{8} = 126.8'^k$

$f_{b_x} = \frac{12(126.8)}{189} = 8.1\,ksi$

$F_{b_x} = 0.6(36)(1.6) = 34.6$

$F_{b_x} = 0.95(36) = 34.2\,ksi$

Horizontal (weak axis)



$f_n < 33cps \Rightarrow g \times 1.5$

$f_n = 5.1\,cps \Rightarrow g = 6.0 \times 1.5 = 9.0$

$M_y = \frac{(6.0)(26)^2}{8} = 507.0'^k$

$f_{b_y} = \frac{12(507.)}{67.1} = 90.7$

$F_{b_y} = 0.95(36) = 34.2\,ksi$

Horizontal (axial)

$f_n = 42.8\,cps \Rightarrow g = 1.2$

$f_a = 20.9/35.0 = 0.6\,ksi$

$l/r_y = \frac{12 \times 26}{3.75} = 83.2$

$l/r_x = \frac{12 \times 26}{6.26} = 49.8$

$F_a = 15\,ksi \times 1.6 = 24\,ksi$

CHECK w/o TRUSS REACTION

$\frac{0.6}{24.0} + \frac{8.1}{34.2} + \frac{90.7}{34.2} \le 1.0$

$0.03 + 0.24 + 2.65 = 2.92 \nleq 1.0$

CHECK w/ TRUSS REACTION of 117^k

$f_a = 117/35.0 = 5.3\,ksi$ $\quad \frac{\times 1.6}{187}$

$\frac{(5.3+0.6)}{24.0} + \frac{8.1}{(1-\frac{5.9}{57.75})34.2} + \frac{90.7}{(1-\frac{5.9}{21.65})34.2} \le 1.0$

$0.246 + 0.263 + 3.64 = 4.15 \nleq 1.0$

Since not all the siding load will be supported by this beam as assumed the frequency of the beam can be shown to move toward the rigid zone & off the peak in the weak axis loading (most critical) therefore the bending will be considerably lowered. Therefore, the beam should be OK

SEE PRIOR NOTE ABOUT TRUSS LOADING ASSUMPTION

Form GO-3.08.1 Rev. 2

SARGENT & LUNDY ENGINEERS CHICAGO	Calcs. For Rx/Aux Bldg Superstructure Steel for Seismic Reevaluation for New Purge System	Calc. No. [illegible]
	✓ Safety-Related / Non-Safety-Related	Rev. 0 Date
		Page 25 of 39

Client DECo	Prepared by [signature] Date 5-7-81
Project Fermi-2	Reviewed by M. Kiezyster Date 5-18-81
Proj. No. 5139-38 Equip. No.	Approved by Date

Roof Eave Purlin W14x119

VERTICAL

$w = \frac{3(1.6)}{26} + 0.5 + 0.119 = 0.804^{k/'}$

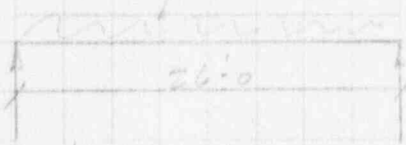
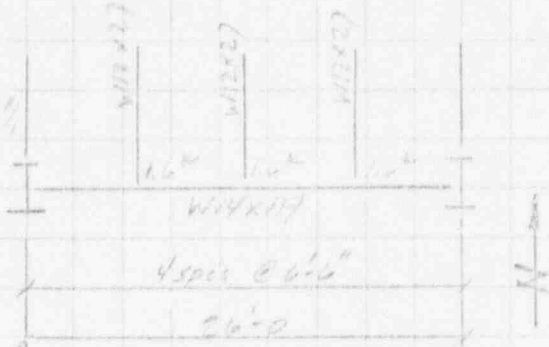


table value = 6.98

$f_n = \frac{6.98}{\sqrt{0.804}} = 7.8$ cps



metal deck + insulation
DL = 15 psf

W12x27 REACTION ON W14

$R = \left(\frac{26}{2}\right)[6.5(0.015) + 0.027] = 1.6^{k}$

HORIZ. N-S (WEAK AXIS)

$w = 0.804^{k/'}$

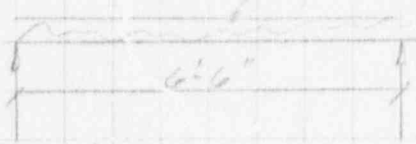
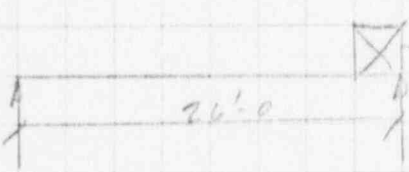


table value ≈ 68.11 ⟨ 7'-0: 57.69; 6'-0: 78.53

$f_n = \frac{68.11}{\sqrt{0.804}} = 76.0$ cps

metal siding is also supported from W14x119 see B-197
DL = 50'(10#/□') = 500#/'

HORIZ. E-W (axial)



$P = 26'(0.804) = 20.9^{k}$

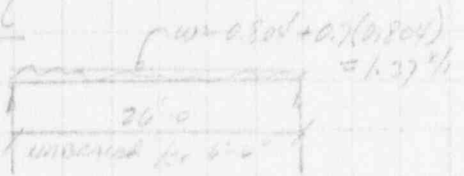
table value = 178.62

$f_n = \frac{178.62}{\sqrt{20.9}} = 39.1$ cps

Form GO-3.08.1 Rev. 2

Roof Eave Purlin W14x119 con't

Vertical



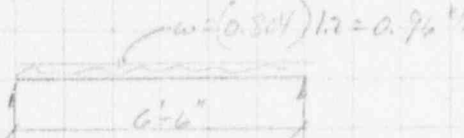
$f_n = 2.8$ cps $\Rightarrow g = 0.7$

$M_x = \frac{(1.37)(26)^2}{8} = 115.8'^k$

$f_{b_x} = \frac{12(115.8)}{187} = 7.4$ ksi

$F_{b_x} = 0.95(36) = 34.2$ ksi

Horizontal (weak axis)



$f_n = 76.0$ cps $\Rightarrow g = 1.2$

$M_y = \frac{(0.96)(6.5)^2}{8} = 5.1'^k$

$f_{b_y} = \frac{12(5.1)}{67.1} = 0.91$ ksi

$F_{b_y} = 0.95(36) = 34.2$

Horizontal (axial)

$f_n = 38.1$ cps $\Rightarrow g = 1.3$

$f_a = 27.2/35.0 = 0.78$ ksi

$l/r_y = 12 \times 6.5/3.75 = 20.8$

$l/r_x = 12 \times 26/6.26 = 49.8$

$F_a = 18.36 \times 1.6 = 29.4$ ksi

$f_a = 0.78$ ksi

Truss load = 152 k $\Rightarrow f_a = 152/35.0 = 4.34$ ksi

$\frac{f_a}{F_a} + \frac{f_{bx}}{F_{bx}} + \frac{f_{by}}{F_{by}} \leq 1.0$ $\qquad \frac{f_a}{F_a} = \frac{4.34 + 0.78}{29.4} = 0.174 \approx .15$

$\frac{4.34+0.78}{29.4} + \frac{7.4}{34.2} + \frac{0.91}{34.2} = 0.174 + 0.216 + 0.027 = 0.417 < 1.0$ OK

Form GO-3.08.1 Rev. 2

SARGENT & LUNDY ENGINEERS CHICAGO

Calcs. For R/Aux. Bldg. Superstruc. Steel for Seismic Reevaluation for New Response Spectra		Calc. No. [illegible]
X Safety-Related	Non-Safety-Related	Rev. 2 Date
		Page [illegible] of 39

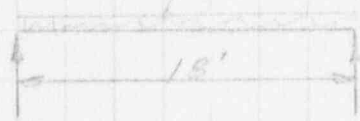
Client DECo		Prepared by [illegible]	Date [illegible]
Project Fermi - 2		Reviewed by [illegible]	Date 5-18-81
Proj. No. 5137-38	Equip. No.	Approved by	Date

CHECK W12x58 w/ ½" cover PL's Roof Truss Diagonal

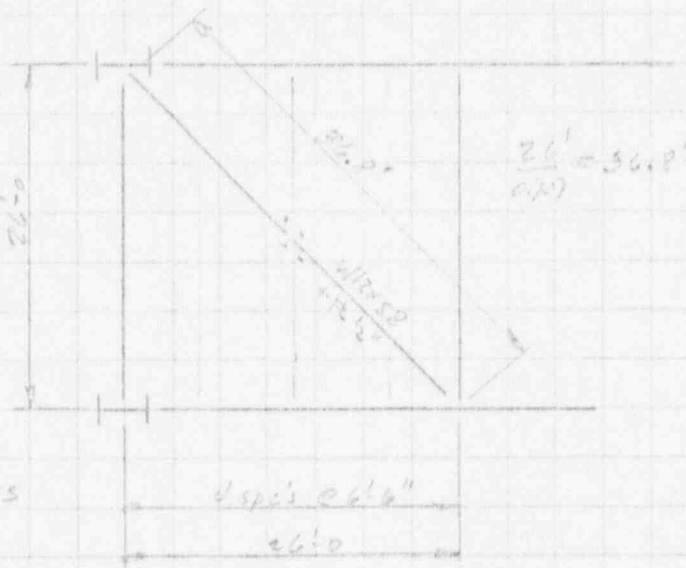
VERTICAL

W12, PL's

$w = 0.058 + 0.037 = 0.095$



18'



26'-0

$\frac{26'}{0.707} = 36.8'$

W12x58 + PL ½"

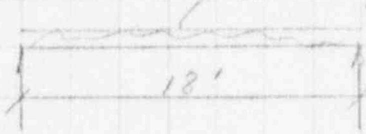
4 spa's @ 6'-6" = 26'-0

$$f_n = \frac{3.55}{\sqrt{\frac{5wL^4}{384 E I_x}}}$$

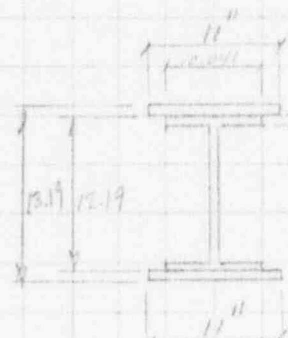
$$= \frac{3.55}{\sqrt{\frac{5(0.095/12)(12\times18)^4}{384(29{,}000)(516.3)}}} = 29.0 \text{ cps}$$

Horiz. (weak axis)

$w = 0.095$



18'



11"

0.440

13.19

12.19

11"

W12x58

$I_x = 476$

$I_y = 107$

$A = 17.1 \text{ in}^2$

$$f_n = \frac{3.55}{\sqrt{\frac{5(0.095/12)(12\times18)^4}{384(29{,}000)(217.9)}}} = 18.8 \text{ cps}$$

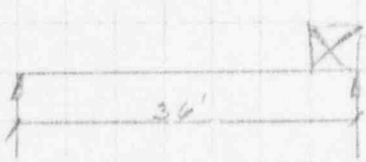
$$I_x = 476 + \frac{11[(13.19)^3 - (12.19)^3]}{12}$$

$$= 516.3 \text{ in}^4$$

$$I_y = 107 + 2\left[\frac{(0.5)(11)^3}{12}\right]$$

$$= 217.9 \text{ in}^4$$

Horiz. (axial)



$P = 36'(0.095)$

$= 3.4^k$

36'

$$A = 17.1 + 2\left(\tfrac{1}{2}\times 11\right)$$

$$= 28.1 \text{ in}^2$$

$$f_n = \frac{1}{2\pi}\sqrt{\frac{AEg}{PL}}$$

$$= \frac{1}{2\pi}\sqrt{\frac{(28.1)(29{,}000)(32.2\times12)}{(3.4)(36\times12)}} = 73.7$$

$$r_y = \sqrt{217.9/28.1} = 2.8$$

SARGENT & LUNDY ENGINEERS CHICAGO	Calcs. For Reactor Bldg Superstructure Steel for Seismic Reevaluation for ...	Calc. No. [illegible]
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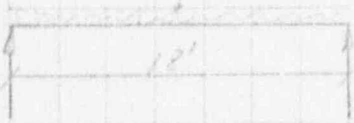
		Page 35 of 39

Client DECo	Prepared by [illegible]	Date 5-3-81
Project Fermi-2	Reviewed by M. Khayyat [illegible]	Date 5-18-81
Proj. No. 4175-18 Equip. No.	Approved by	Date

Check W12×58 w/ ½" cover pl. at Roof Truss Diagonal con't

Vertical

$w = 0.095 + 0.55(0.095) = 0.147\ ^k/_{'}$



$f_n = 22.0$ cps $\Rightarrow g = 0.55$

$M_x = \dfrac{(0.147)(18)^2}{8} = 6.0\ 'k$

$f_{b_x} = \dfrac{12 \times 6\left(\frac{13.19}{2}\right)}{476} = 1.0$ ksi

$\dfrac{f_{bx}}{F_{bx}} = \dfrac{1.0}{47.5} = 0.02$

$r_t = \sqrt{I/A} = \sqrt{\dfrac{\frac{(0.5)(11)^3}{12} + \frac{(0.64)(10.64)^3}{12}}{(0.5)(11) + (0.64)(10.64)}}$

$= 7.14"$

$l/r_t = 12(18)/7.14 = 30.3$

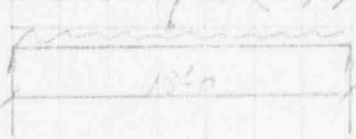
$F_b = 0.6(50) = 30 \times 1.67 = 50.1$

use $F_b = 0.95(50) = 47.5$

Horiz. (weak axis)

N-S direction

$w = (0.095)1.3 = 0.124$



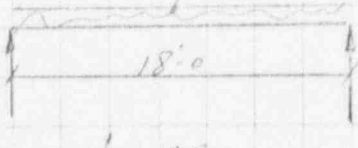
$f_n = 18.8$ cps $\Rightarrow g = 1.3$

$M_y = \dfrac{(0.124)(18)^2}{8} = 5.0\ 'k$

$f_{b_y} = \dfrac{(12)(5)\left(\frac{11}{2}\right)}{217.5} = 1.5$ ksi

E-W direction

$w = (0.095)1.3 = 0.124$



$f_n = 18.8$ cps $\Rightarrow g = 1.3$

$f_{b_y} = \sqrt{(1.5)^2 + (1.5)^2} = 2.1$ ksi

$F_{b_y} = 0.95(50) = 47.5$

$\dfrac{f_{b_y}}{F_{b_y}} = \dfrac{2.1}{47.5} = 0.05$

Form GQ-3.08.1 Rev. 2

SARGENT & LUNDY ENGINEERS CHICAGO	Calcs. For Rx Bldg Superstruc. Steel for Seismic Reevaluation for New Response Spectra	Calc. No. SS-001
	X Safety-Related / Non-Safety-Related	Rev. 0 Date
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Client DECo	Prepared by G. [illegible]	Date 5-2-81
Project Fermi-2	Reviewed by M. Khanjirala	Date 5-18-81
Proj. No. 6135-38 Equip. No.	Approved by	Date

Check W12×58 w/ 1/2" cover PL Roof Truss Diagonal cont

Horiz. (axial)

N-S direction

$P \rightarrow$ ——— $\leftarrow P = 3.4^k(1.2) = 4.1^k$ $\quad f_a = 4.1/_{28.1} = 0.15$

$36'-0$ unbraced for $18'-0$

$f_n = 73.7$ cps $\Rightarrow g = 1.2$

E-W direction

$P \rightarrow$ ——— $\leftarrow P = 3.4^k(1.3) = 4.4^k$ $\quad f_a = 4.4^k/_{28.1} = 0.16$

$36'-0$ unbraced for $18'-0$

$f_a = \sqrt{(0.15)^2 + (0.16)^2} = 0.22$

$f_n = 73.7$ cps $\Rightarrow g = 1.3$ $\quad \frac{l}{r_y} = \frac{12(18)}{2.8} = 77.1 \Rightarrow F_a = 19.6 \times 1.67 = 32.7$ ksi

DIAGONAL IS PART OF TRUSS SPANNING E-W, THEREFORE, ADD AXIAL TRUSS LOAD FROM N-S EVENT.

From previous calc's truss load $= 60^k \times 1.6 = 96^k \Rightarrow f_a = \frac{96}{28.1} = 3.4$

$$\frac{f_a}{F_a} + \frac{f_{bx}}{F_{bx}} + \frac{f_{by}}{F_{by}} \le 1.0$$

$$\frac{(0.22 + 3.4)}{32.7} + \frac{1.0}{47.5} + \frac{2.1}{47.5} \le 1.0$$

$$0.110 + 0.021 + 0.044 = 0.175 < 1.0 \quad \text{OK}$$

By inspection remaining truss diagonals will be OK

Form GO.3.08.1 Rev. 2

SARGENT & LUNDY
ENGINEERS

DESIGN CONTROL SUMMARY
DESIGN VERIFICATION

PAGE 1

PROJECT NAME ENRICO FERMI
PROJECT NO. 6150-33 UNIT NO. 2
CLIENT DETROIT EDISON CO
CALC. NO. & DESCRIPTION 4.21.1
CALC. FOR RHR COMPLEX - BASE MAT-SEISMIC REEVALUATION

☑ SAFETY RELATED ☐ NON SAFETY RELATED

COMMENT NO.
QA SERIAL NUMBER

SIGNATURE & DATE FOR REV.		IDENTIFICATION OF PAGES PREPARED/REVISED/VOIDED & REVIEW METHOD
REV. 0 PREPARER	M. Khayyata 5-15-81	Pg. 1 TO 8
REV. 0 REVIEWER	J. Daan 5-15-81	
REV. 0 APPROVER	N. V. Shingarani 5-16-81	REVIEW METHOD Detailed Review
REV __ PREPARER		
REV __ REVIEWER		
REV __ APPROVER		REVIEW METHOD
REV __ PREPARER		
REV __ REVIEWER		
REV __ APPROVER		REVIEW METHOD
REV __ PREPARER		
REV __ REVIEWER		
REV __ APPROVER		REVIEW METHOD

FORM SAS 22.1 Rev. 0

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ENGINEERS

DESIGN CONTROL SUMMARY
DESIGN ELEMENT INDEX
PROJECT NO. [illegible] UNIT NO. 2

Revision

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1.	ASSUMPTIONS AND CONCLUSIONS	1-2
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3.	LOADING CALCULATIONS	5-6
4.	SHEARS AND MOMENTS CALCULATIONS	7-8

FORM SAS 22.3 Rev. 0

SARGENT & LUNDY ENGINEERS

DESIGN CONTROL SUMMARY
ASSUMPTIONS AND PROCEDURES
PROJECT NO. 6155-83 UNIT NO. 2

PAGE 3 OF CALCULATIONS NO. RHR BASE MAT REEVALUATION REV. 0 DATE 5-15-81

Revision

THE ASSUMPTIONS AND PROCEDURES FOR THE RE-EVALUTION OF THE RHR BASE MAT IS COVERED IN PAGES 1 & 2 OF THIS CALCULATIONS.

FORM SAS-22.4 Rev. 0

FORM SAS 22.7 Rev. 0

NO.	INPUT DOCUMENT			REVISION NUMBER OR DATE ON LINE INDICATED								USED IN DESIGN OF	CNT	C/N
	DESCRIPTION	SOURCE	IDENT	DATE	S	DATE	S	DATE	S	DATE	S			
1	ACI 318-77	—	—											
2	COL. LOAD CALCS.	S&L	1.14.7											
3	BASE MAT. CALCS.	S&L	1.2.1											
4	RHR FND PLAN EL. 555-0"	S&L	DWG. B-257	6-12-78	G									
5	SEISMIC R-ANALYSIS FOR 5% + 7% DAMPING SITE SPECTRA RHR COMPLEX	S&L	SDD-D[illegible] -001	4-18-81										

SARGENT & LUNDY ENGINEERS

DESIGN CONTROL SUMMARY
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PROJECT NO. [illegible] UNIT NO. 2

PAGE 4 OF CALCULATIONS NO. 1-3.[illegible] REV. 0 DATE 5-[illegible]-1981

CNT - Continued CN - Comment Number S - Status

SARGENT & LUNDY ENGINEERS CHICAGO	Calcs. For BASE MAT - RHR COMPLEX RE-EVALUATION	Calc. No. 1.31.1
	X Safety-Related / Non-Safety-Related	Rev. / Date
		Page 1 of 6

Client DETROIT EDISON	Prepared by [illegible] Date 5-5-[illegible]
Project FERMI-2	Reviewed by [illegible] Date [illegible]
Proj. No. 6159-29 Equip. No.	Approved by Date

ASSUMPTIONS AND CONCLUSIONS

THE ASSESSMENT OF THE BASE MAT FOR THE NEW SEISMIC LOADS (7% SITE DAMPING DBE CASE) IS AS FOLLOW:

A 9'-0" COLUMN STRIP IS CHOSEN AS AN EFFECTIVE BEAM WIDTH. THIS BEAM IS CONSIDERED RIGID AND THEN THE SOIL WILL ADJUST ITSELF SO THAT EACH COLUMN LOAD WILL SPREAD ALMOST UNIFORMLY UNDER THE MAT. THE SECTION OF THE MAT IS LOADED WITH THE COLUMNS DEAD LOADS, EQUIPMENT LOADS VERTICAL SEISMIC LOADS, SEISMIC SHEAR WALLS LOADS AND HYDROSTATIC LOAD. (** SEE SHEET 2)

THE COLUMS STRIP LOCATION WAS CHOSEN TO SPAN ALONG THE SHORT LENGTH OF THE BUILDING (IN THE EAST WEST DIRECTION) AND AT THE END OF THE BUILDING (IN THE NORTH SOUTH DIRECTION) SO THAT TO ACCOUNT FOR THE MAX. EFFECT OF THE SEISMIC LOADS FROM THE EAST & WEST SHEAR WALLS. (SEE SKETCH ON SHEET 3).

THE LOADS FROM BOTH WALLS & COLUMNS WAS ASSUMED TO CAUSE A UNIFORM SOIL PRESSURE DISTRIBUTION ON THE MAT. THIS PRESSURE WAS COMBINED WITH THE GROUND WATER PRESSURE & THE NET EFFECT

(CONT.)

Form GO-3.08.1 Rev. 2

WAS USED TO CHECK THE MAT.

AN APPROXIMATE METHOD OF FRAME ANALYSIS USING THE ACI MOMENT & SHEAR COEFFICIENTS WAS USED.

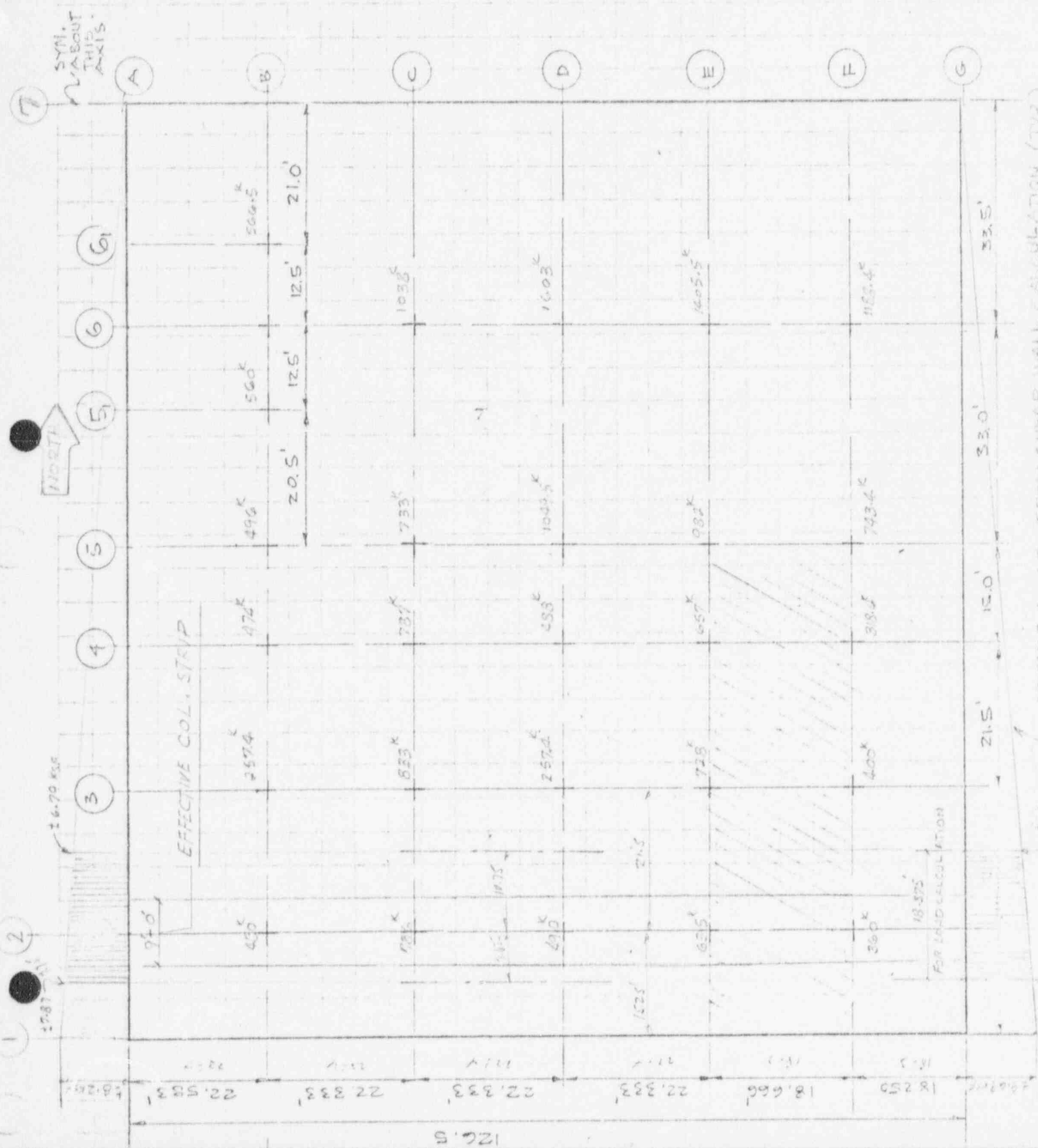
IN BOTH CASES (APPLYING SEISMIC LOADS ACTING UP & DOWN) THE STRUCTURAL INTEGRITY OF THE MAT WAS FOUND TO BE ACCEPTABLE.

** ASUMPTIONS OF UNIFORMLY DISTRIBUTION OF THE SOIL PRESSURE UNDER THE MAT IS APPROXIMATE AND DEPENDS UPON THE VALUE OF THE SUB-GRADE MODULUS OF THE SOIL HOWEVER, THE MOMENTS OBTAINED DUE TO UNIFORM SOIL PRESSURE ARE EXPECTED TO BE HIGHER THAN CALCULATED. ALSO SUFFICIENT MARGIN IN FLEXURAL REINFORCEMENT (SEE SHEET 7) DOES NOT REQUIRE MORE RIGOROUS ANALYSIS.

SARGENT & LUNDY ENGINEERS CHICAGO	Calcs. For BASE MAT - RHR COMPLEX COLUMNS + SEISMIC SHEAR WALLS LOADS	Calc. No.
	X Safety-Related / Non-Safety-Related	Rev. / Date / Page 8 of 8

Client DETROIT EDISON	Prepared by M. Khayyatin	Date 4-30-81
Project FERMI-2	Reviewed by	Date 5-13-81
Proj. No. 6139-38 Equip. No.	Approved by	Date

CALCS. FOR MAT FOUNDATION FOR RHR COMPLEX EL. 555-0



SARGENT & LUNDY ENGINEERS CHICAGO	Calcs. For BASE MAT - RHR COMPLEX	Calc. No. 1.3.1
	X Safety-Related / Non-Safety-Related	Rev. / Date
		Page 4 of 8

Client DETROIT EDISON	Prepared by M. Krawulski — Date 1-23-81
Project FERMI-2	Reviewed by [illegible] — Date [illegible]
Proj. No. 6139-38 Equip. No.	Approved by — Date

WALL LOADS CALCULATIONS - REF. CALCS. BOOK 1.16.7 (COL. LOAD)

LOAD FROM	WALL "A"	WALL "G"
SELF WEIGHT to EL. 590'		1.5x27x18.375x.15 = 112^k
" " to EL. 565'	3.5x63x18.375x.15 = 608^k	3.5x35x18.375x.15 = 338^k
	FROM COL. A2 = 155^k	
* FROM SLAB AT EL. 617'		2.08x18.375 = 38^k
FROM SLAB AT EL. 590'	—	2.01x18.375 = 37^k
SEISMIC	$\pm\left(\frac{7.82+6.29}{2}\right)$3.5x18.375 = ±907^k	$\pm\left(\frac{9.27+7.21}{2}\right)$3.5x18.375 = ±550^k
TOTAL (SEISMIC ↓)	1237^k	1075^k
TOTAL (SEISMIC ↑)	295	(UPLIFT) -25^k

ACI 13.2.1

effective column strip = $0.25\,l_1$ or l_2 ⇒ 0.25(15.25 + 21.5) = 9.1' ≈ 9'-0

* Some load from cooling tower deck & cooling tower should be added at this EL. but it is small in comparison to other loads

Form GQ-3.08.1 Rev. 2

SARGENT & LUNDY
ENGINEERS
CHICAGO

Calcs. For BASE MAT SUB COMPON	Calc. No. 1.3.1.1
X Safety–Related / Non–Safety–Related	Rev. / Date
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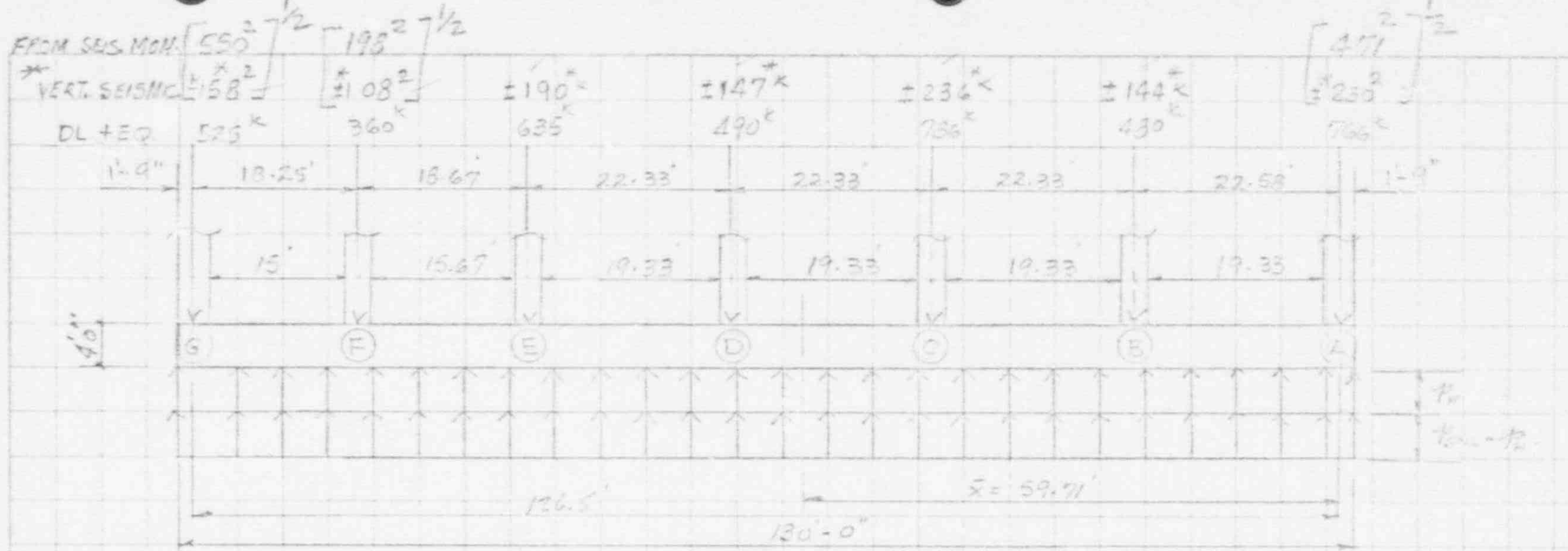
Client DECO		Prepared by	Date
Project FERMI-2		Reviewed by	Date 5-13-…
Proj. No. 4189-38	Equip. No.	Approved by	Date

Form GQ-3.08.1 Rev. 2



FIND ECCENTRICITY OF DL + EQUIPMENT

$\Sigma M_A = 0$

$$\bar{x} = \frac{525(126.5) + 360(108.25) + 635(89.58) + 490(67.25) + 756(44.91) + 430(22.58)}{4042}$$

$= 59.71'$

$e = \frac{126.5}{2} - 59.71 = 3.54'$, IGNORED, UNIFORM SOIL DISTRIBUTION WILL BE ASSUMED

USE AVE. HIGH WATER PRESSURE FROM CALCS 1.2.1 SHT. 4

$p_H = \frac{p_{H1} + p_{H2}}{2} = \frac{2.02 + 2.5}{2} = 2.25\ ^k/_{ft^2}$

$\Sigma P = \Sigma P_{DL+EQ} \pm \Sigma P_{SEIS}$

CASE 1 : $1.0\,DL + 1.0\,p_H + 1.0(7\%\ DBE \downarrow)$ — SLAB WEIGHT EXCLUDED

CASE 2 : $1.0\,DL + 1.0\,p_H + 1.0(7\%\ DBE \uparrow)$ — SLAB WEIGHT INCLUDED

* 0.30 x DL+EQ → 0.3 is vertical acceleration of mat elev. (CONT.)

SARGENT & LUNDY ENGINEERS CHICAGO	Calcs. For BASE MAT - RHR COMPLEX	Calc. No. 1.31.1
	X Safety-Related / Non-Safety-Related	Rev. Date / Page 3 of

Client DECO	Prepared by N. Khanzode	Date 5/15/81
Project FERMI-2	Reviewed by J. Patel	Date
Proj. No. 5199-59 Equip. No.	Approved by	Date

$\Sigma P_{SEIS} = \left[550^2 + 108^2\right]^{\frac{1}{2}} + \left[198^2 + 108^2\right]^{\frac{1}{2}} + 190 + 147 + 236 + 144 + \left[230^2 + 147^2\right]^{\frac{1}{2}}$

$\Sigma P_{SEIS} = \pm 2039^K$

$\Sigma P_{DL+LL} = 525 + 360 + 525 + 490 + 750 + 430 + 962 = 4042^K$

$\Sigma P = 4042 \pm 2039$

CASE 1 :

NET UPWARD PRESSURE EXERTED ON SLAB

$p = \frac{\Sigma P}{AREA} - P_H + P_H \leq P_H$

$p = \frac{4042 + 2039}{9 \times 130} = 5.20 \text{ KSF} > 2.25 \text{ KSF} \quad O.K.$

CASE 2 : (NO NEED TO CHECK SLAB REINFORCING FOR THIS CASE)

$p = \frac{4042 - 2039}{9 \times 130} + 4' \times 0.15$

$= 1.71 + 0.6 = 2.31 \text{ KSF} > 2.25 \text{ KSF}$

NOTE THAT IN THE ABOVE CALCULATIONS THE RESERVOIR IS CONSIDERED EMPTY. HOWEVER DURING SSE CONDITION THERE WILL BE SOME WATER IN THE RESERVOIR.

Form GQ-3.08.1 Rev. 2

SARGENT & LUNDY ENGINEERS CHICAGO

Calcs. For BASE MAT RHR COMPLEX		Calc. No.
X Safety-Related	Non-Safety-Related	Rev. Date
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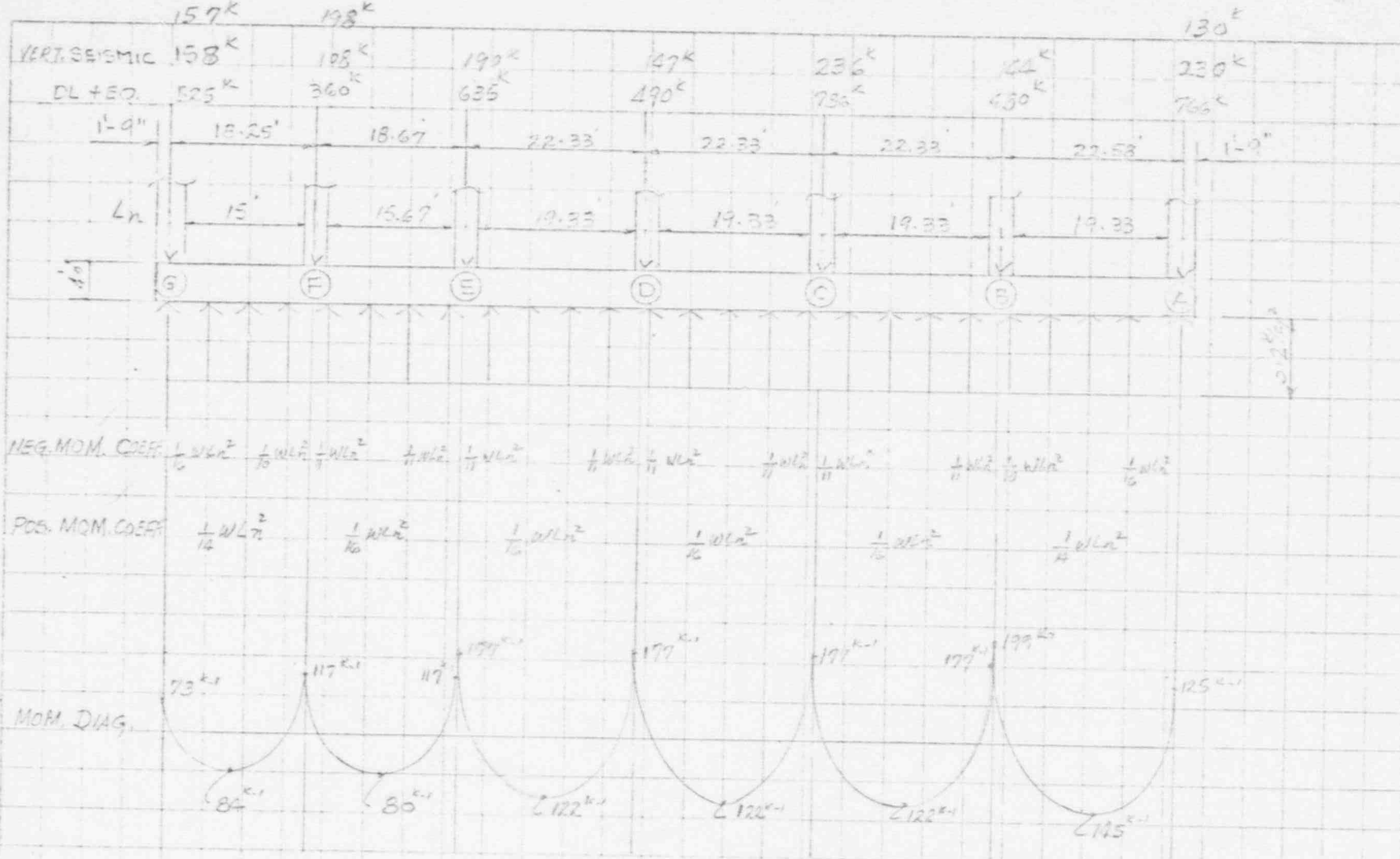
Client DECO	Prepared by M. K...	Date
Project FERMI-2	Reviewed by J. ...	Date
Proj. No.	Equip. No.	Approved by	Date

Form GQ-3.08.1 Rev. 2



$$\text{MAX. SHEAR} = 1.15\,\frac{wL_n}{2} = 1.15\,\frac{5.2 \times 19.58}{2} = 58.54^K$$

$$\text{SHEAR STRESS } v = \frac{58.54 \times 1000}{0.85 \times 12 \times 44.0} = 150.4 \text{ PSI} < 4\sqrt{4000} \quad \text{O.K}$$

$$(A_s)_{REQ} = \frac{M}{\phi F_Y(.85d)} = \frac{199 \times 12}{0.9 \times 60 \times .85 \times 44.0} = 1.13 \text{ SQ.IN} < \#11 @ 12 \text{ FURNISHED TOP \& BOT. O.K}$$

(1.56 sq in)

SARGENT & LUNDY ENGINEERS CHICAGO	Calcs. For BASE MAT - RHR COMPLEX	Calc. No. 1.31.1
	X Safety-Related / Non-Safety-Related	Rev. / Date / Page 8 of

Client DETROIT EDISON	Prepared by [illegible] Date [illegible]
Project EFRMI-2	Reviewed by [illegible] Date [illegible]
Proj. No. 6159-28 Equip. No.	Approved by Date

CHECK PUNCHING SHEAR :

COLUMN D6 :

DL + EQUIPMENT = 1603^k

LIVE LOAD = 453^k

VERT. SEISMIC = 0.3X1603 = 481^k

$d \approx 44''$

$4\sqrt{4000} = 253$ PSI

LOADING COMBINATION :

① $1.4(1603) + 1.7(453) = 3014.3^k$ (GOVERNS)

② $1.0(1603) + 1.0(481) = 2084^k$

$$v_u = \frac{3014.3 \times 1000}{0.85 \times 320 \times 44} = 251 \text{ PSI} < 253 \text{ PSI} \quad O.K.$$

Form GQ-3.08.1 Rev. 2

SARGENT & LUNDY ENGINEERS

DESIGN CONTROL SUMMARY
DESIGN VERIFICATION

PAGE 1

PROJECT NAME [illegible]
PROJECT NO. 6159-38 UNIT NO. 2
CLIENT [illegible] EDISON CO.
CALC. NO. & DESCRIPTION 1.30.1
CALC. FOR [illegible]

☐ SAFETY RELATED ☐ NON SAFETY RELATED

COMMENT NO.

QA SERIAL NUMBER

SIGNATURE & DATE FOR REV.			IDENTIFICATION OF PAGES PREPARED/REVISED/VOIDED & REVIEW METHOD
REV. 0	PREPARER	M. Khurysta 5-15-81	PG 1 TO 23
	REVIEWER	F. Daus 5-18-81	
	APPROVER	N. V. Hingorani 5-18-81	REVIEW METHOD Detailed Check.
REV. __	PREPARER		
	REVIEWER		
	APPROVER		REVIEW METHOD
REV. __	PREPARER		
	REVIEWER		
	APPROVER		REVIEW METHOD
REV. __	PREPARER		
	REVIEWER		
	APPROVER		REVIEW METHOD

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SARGENT & LUNDY ENGINEERS

DESIGN CONTROL SUMMARY
DESIGN ELEMENT INDEX
PROJECT NO. [illegible] UNIT NO. 2

PAGE 2 OF CALCULATIONS NO. 1.30.1 REV. 0 DATE [illegible]

Revision

FORM SAS 22.3 Rev. 0

PURPOSE : THE PURPOSE OF THESE CALCULATIONS IS TO RE-EVALUATE THE SHEAR WALLS FOR THE NEW SEISMIC LOADING.

ASSUMPTIONS: THE SEISMIC FORCES FROM THE FLOOR SLABS RESPONSES ARE TRANSFERED THROUGH THE SHEAR WALLS TO THE MAT. THE SHEAR WALLS ARE CHECKED FOR THE SIMULTANEOUS ACTION OF THE LONGITUDINAL SEISMIC FORCES, THE DYNAMIC LAT. SOIL PRESSURE (FOR PORTION OF WALL BELOW GRADE), VERTICAL SEISMIC AND GRAVITY LOADS.

PROCEDURE : EACH SHEAR WALL IS CHECKED FOR THE MAX. SEISMIC SHEAR + MOMENT EXERTED ON IT. THESE VALUES WERE TAKEN FROM THE SEISMIC RE-ANALYSIS FOR 5% + 7% SITE DAMPING (SEE CALC. SDD-DEC0-001). THE SHEAR CAPACITY OF EACH WALL WAS CHECKED AND THE SHEAR STRESS IN MOST CASES WAS LESS THAN $2\sqrt{f'_c}$.

EACH WALL WAS CHECKED FOR TENSION AT THE CORNERS USING THE FORMULA:

$$\sigma = \frac{Mc}{I} - \frac{N}{A} \leq 0.15\, f_r$$

WHERE

M = IS THE INPLANE SEISMIC MOMENT FT-KIPS

N = NET AXIAL LOAD ON THE WALL KIPS

I = GROSS UNCRACKED MOMENT OF INERTIA FT^4

A = GROSS AREA OF THE WALL FT^2

IN ALL CASES THE TENSILE STRESS AT THE CORNERS WAS FOUND TO BE LESS THAN $0.15 f_r$

WHERE $f_r = 7.5\sqrt{f'_c}$

CONCLUSION: ALL THE SHEAR WALLS IN THE RHR COMPLEX ARE FOUND TO B STRUCTURALLY ADEQUATE.

FORM SAS-22.4 Rev. 0

SARGENT & LUNDY
ENGINEERS

DESIGN CONTROL SUMMARY
DESIGN INPUT DOCUMENTS
PROJECT NO. 6130-38 UNIT NO. 2

PAGE 4 OF CALCULATIONS NO. 1-30-1 REV. 0 DATE 5-15-1981

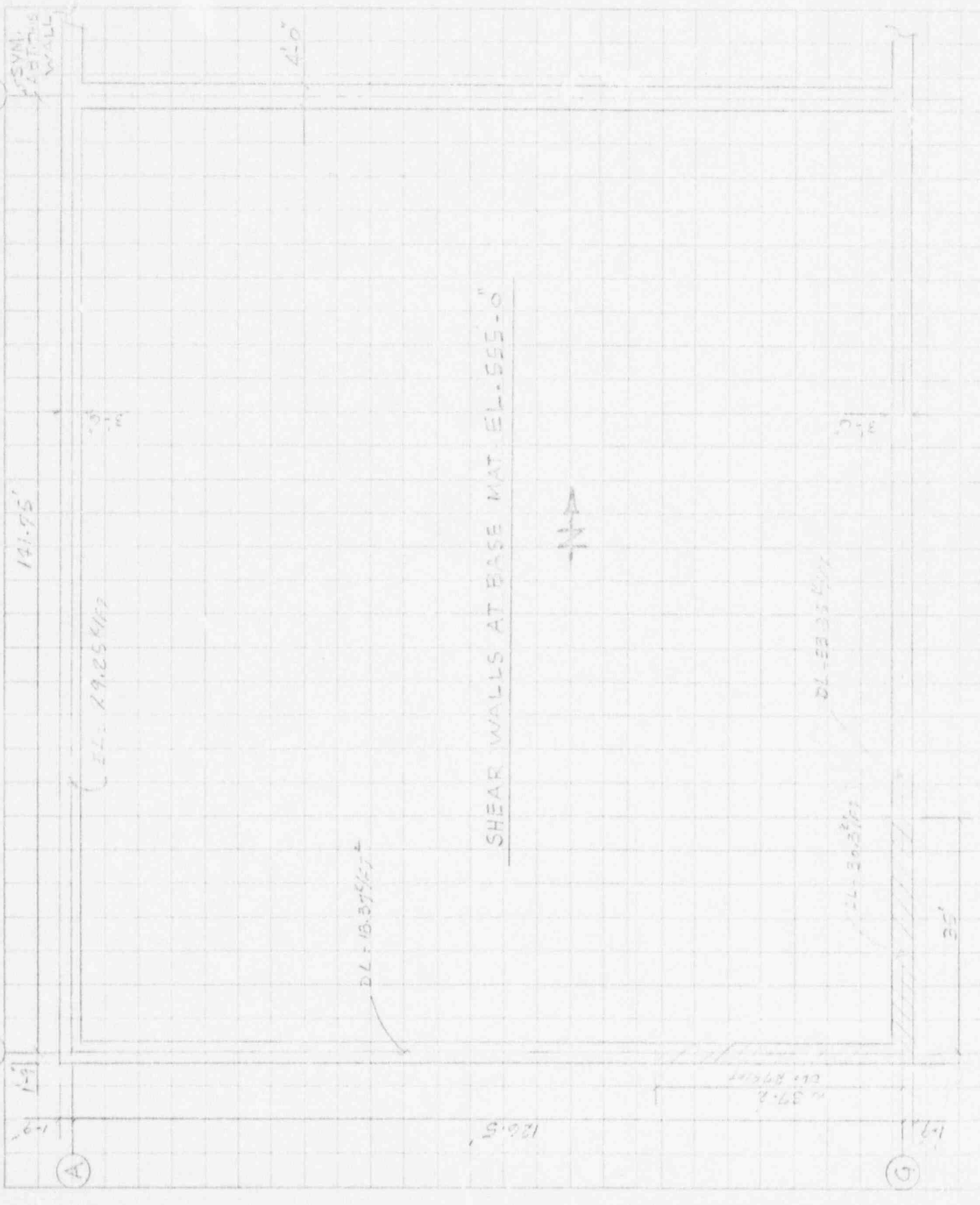
FORM SAS 22.7 Rev. 0

NO.	INPUT DOCUMENT			REVISION NUMBER OR DATE ON LINE INDICATED								USED IN DESIGN OF	CNT	C/N
	DESCRIPTION	SOURCE	IDENT	DATE	S	DATE	S	DATE	S	DATE	S			
1	ACI 313-77													
2	CONC. LAT. LOADS RESISTING SYSTEM FOR NUCLEAR PLANTS	S&L	SDS E31.0											
3	CALCULATION BOOK	S&L	1.15.10											
4	SEISMIC RE-ANALYSIS FOR 5% + 7% DAMPING SITE SPECTRA	S&L	SDD-DEC0.001											

CNT - Continued CN - Comment Number S - Status

SARGENT & LUNDY ENGINEERS CHICAGO	Calcs. For SHEAR WALLS + RBP COMPLEX [illegible]	Calc. No. 1.28.1
	X Safety–Related / Non–Safety–Related	Rev. 0 Date [illegible]
		Page 1 of 23

Client DETROIT EDISON	Prepared by [illegible] Date [illegible]
Project FERMI-2	Reviewed by [illegible] Date [illegible]
Proj. No. 6150-[illegible] Equip. No.	Approved by Date

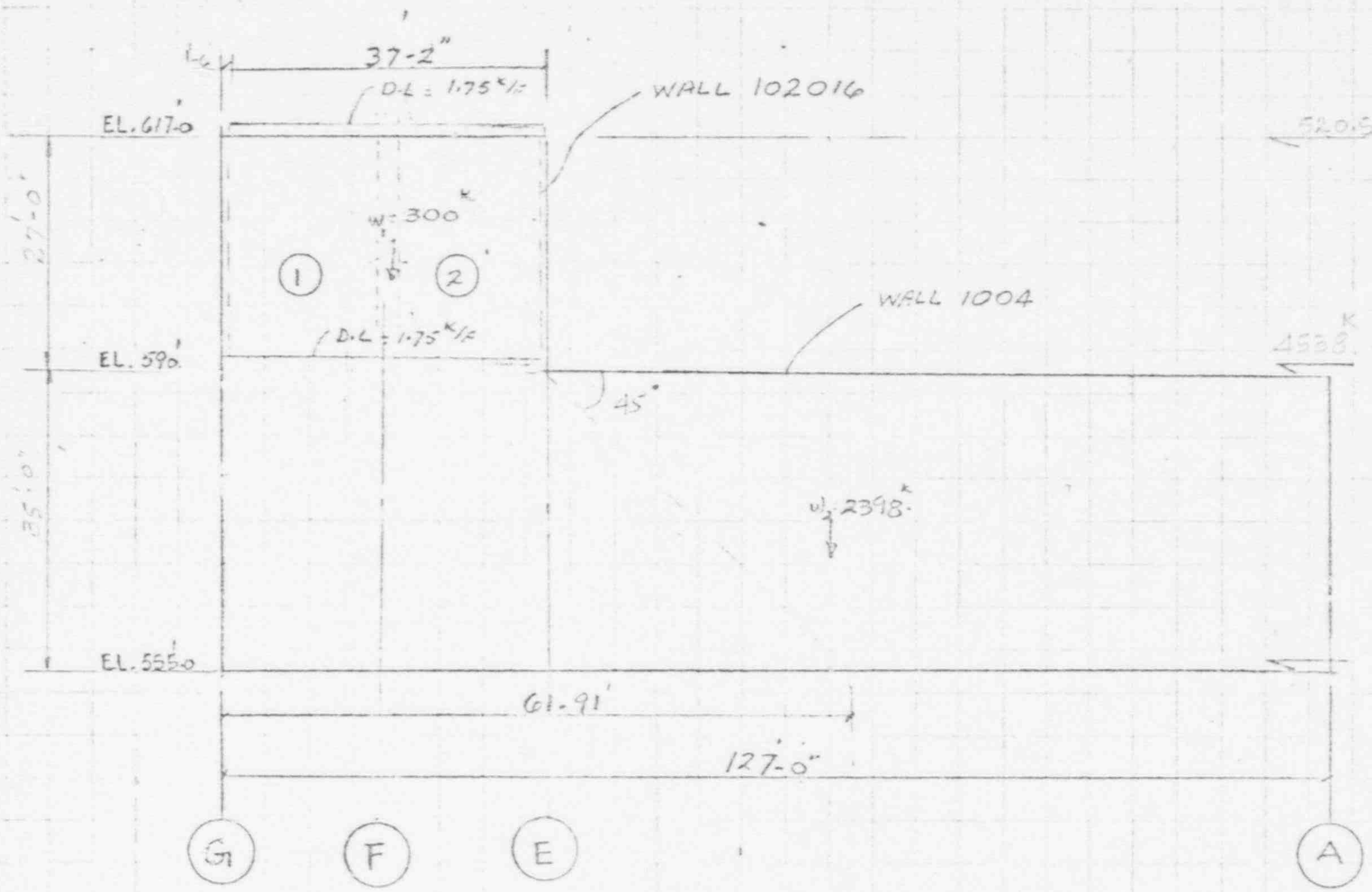


Form GQ.3.08.1 Rev. 2

SARGENT & LUNDY ENGINEERS CHICAGO

Calcs. For SHEAR WALLS ON ROWS "1" & 13" RHR COMPLEX		Calc. No. 1.30.1
X Safety-Related	Non-Safety-Related	Rev. 0 Date 5-5-81
		Page 2 of

Client DECO	Prepared by M. Krayewski	Date 4/16/81
Project FERMI-2	Reviewed by [illegible]	Date 5-?-81
Proj. No. 6139-38 Equip. No.	Approved by	Date



ELEV. OF SOUTH WALL

$w_1 = 37.17(1.75) + 0.225(38.66)\,27' = 300^k$

$w_2 = 37.17(1.75) + .15(3.5)(35)(127) = 2398^k$

SARGENT & LUNDY ENGINEERS CHICAGO	Calcs. For SHEAR WALLS IN RHR COMPLEX	Calc. No. 1.30.1
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ASSESSMENT OF NEW RESPONSE SPECTRA ON ORIGINAL DESIGN

t	WALL (SPECIMEN)	NEW SHEAR KIPS	NEW MOMENT AT BOT. 1-K	DESIGN SHEAR KIPS	DESIGN MOMENT AT BOT 1-K	RATIO NEW/DESIGN SHEAR	RATIO NEW/DESIGN MOMENT
1'-6"	102016	520.5	14053.5	358.4	9658.5	1.45	1.46
	102018	520.5	14053.5	358.4	9658.5	1.45	1.46
3'-6"	1004	4538	172883.5	2430.1	95093.6	1.87	1.82
	1006	4538	172683.5	2430.1	95093.6	1.87	1.82

COMMENTS
Values in chart are based on 5% site response damping & should be lowered for 7% design

CHECK STRESSES - MULTIPLY RATIO BY DESIGN VALUE (REF CALC 1.15.10 SHT. 11)

FOR 102016 $v_u = 1.45 \times 65.6 = 95.12$ PSI $< 2\sqrt{f'c}$ ... 126.8 PSI O.K.

& 102018 $A_v = \dfrac{520.5}{0.7 \times 60 \times 38} = 0.326^{\square''}/\text{ft}$ $\mu = 0.7$ (CONSERVATIVE)

TENSION STEEL IN WALL FLANGES

$\sigma = 1.46 \times 19.54 = 28.5$ KSF $= 197.9$ PSI $> 0.15 f_r = 71$ PSI

EFFECTIVE FLANGE - $9t$ ... $= 13.5'$

$A_f = \dfrac{28.5 \times 13.5}{0.9 \times 60} = 7.125$ SQ.IN

COMBINE SEISMIC & LAT. SOIL PRESSURE & CHECK VERT. STL. FOR [illegible]

$(M)_{soil} = 15.8\ ^{'K}/\text{FT}$ REF. CALCS. 1.15.10 SHT. 13

$A_s = \dfrac{0.326}{2} + 0.192$ $\dfrac{M_u}{\phi f'c\, b d^2} = \dfrac{15.8}{0.9 \times 4 \times (16)^2} = $ [illegible]

$= 0.355^{\square''} < 0.79^{\square''}$ FURNISHED $\quad \rho = 0.0011$

$A_s = 0.0011 \times 12 \times 16$

$= 0.192^{\square''}/\text{ft}$

Form GQ-3.08.1 Rev. 2

SARGENT & LUNDY ENGINEERS CHICAGO	Calcs. For SHEAR WALLS IN RHR COMPLEX	Calc. No. 1.[illegible]
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ASSESMENT OF NEW RESPONSE SPECTRA ON ORIGINAL DESIGN

WALLS 100A & 100G $t = 3'-6"$ $L = 127'-0"$

CHECK INPLANE SHEAR STRESS:

$$v_u = \frac{4538}{0.85 \times 127 \times 3.5} \times \frac{1000}{144} = 93.4 \text{ PSI} \quad 2\sqrt{4000} \quad \text{O.K.}$$

CHECK TRANSVERSE SHEAR STRESS (DUE TO SOIL PRESSURE)

$$v_u = \frac{74.42}{0.85 \times 1.0 \times 3.5} \times \frac{1000}{144} = 173.7 \text{ PSI} > 2\sqrt{4000}$$

$$v_u = \sqrt{53.4^2 + 173.7^2} = 192.7 \text{ PSI} < 12\sqrt{4000} \quad \text{O.K.}$$

$$V_u = \frac{192.7 \times 144 \times .85 \times 3.5}{1000} = 82.55^K$$

$$A_{vf} = \frac{52.55}{0.9 \times 1 \times 60} = 1.94 \text{ in}^2 < 2\text{-}\#11 \quad \text{O.K.}$$

FLEXURAL CHECK:

$$\sigma = \frac{MC}{I} - \frac{N}{A}$$

CHECK WALL AS COMPOSITE SECTION WITH EFF. FLANGE WIDTH EQUAL 6t.

FOR VERTICAL SEISMIC USE DOUBLE PEAK VALUE OF RESPONSE SPECTRA PREPARED IN 1974. (REF. N.Y. HINGORANI)

(CONT.)

Form GQ-3.08.1 Rev. 2

SARGENT & LUNDY ENGINEERS CHICAGO	Calcs For SHEAR WALLS IN RHB	Calc. No. 1-3.1
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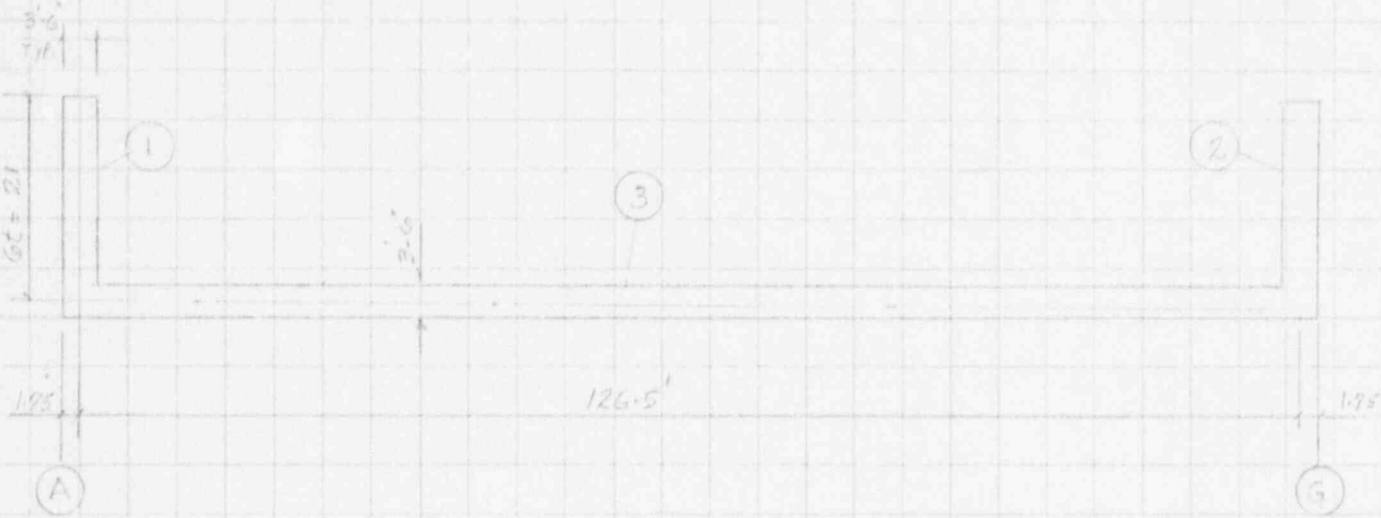
SPRINGS 1004 & 1006



$$I = \frac{3.5(130)^3}{12} + 2\left[19.25\,\frac{(3.5)^3}{12} + 3.5(19.25)\left(\frac{126.5}{2}\right)^2\right] = 1180005\ ft^4$$

$$A_1 = A_2 = 3.5 \times 19.25 = 67.37\ \square'$$

$$A_3 = 3.5 \times 130 = 455\ \square'$$

$$N = 29.25 \times 19.25 + (29 \times 33 + 18.37 \times 71) + 30.3 \times 19.25$$

$$= 563 + 2725 + 583 = 3844^k \quad 3871^k$$

vert g value

$$(N)_{NET} = \underset{3871}{3844} - 0.3\,\underset{3871}{(3844)} = 2690^k \quad 2710^k$$

$$\sigma = \frac{172853 \times 65}{1180005} - \frac{2690\ (2710)}{\underset{455+2\times 67.37}{589.74}}$$

$$= 9.52 - 4.6 = 4.91^{KSF}\ (4.93) < 0.15 f'_c = 10.22\ KSF \quad O.K.$$

∴ NO TENSION AT CORNERS

SARGENT & LUNDY ENGINEERS CHICAGO	Calcs. For SHEAR WALL ON ROW G RHR COMPLEX	Calc. No.	Rev. 0	Date 5-5-81
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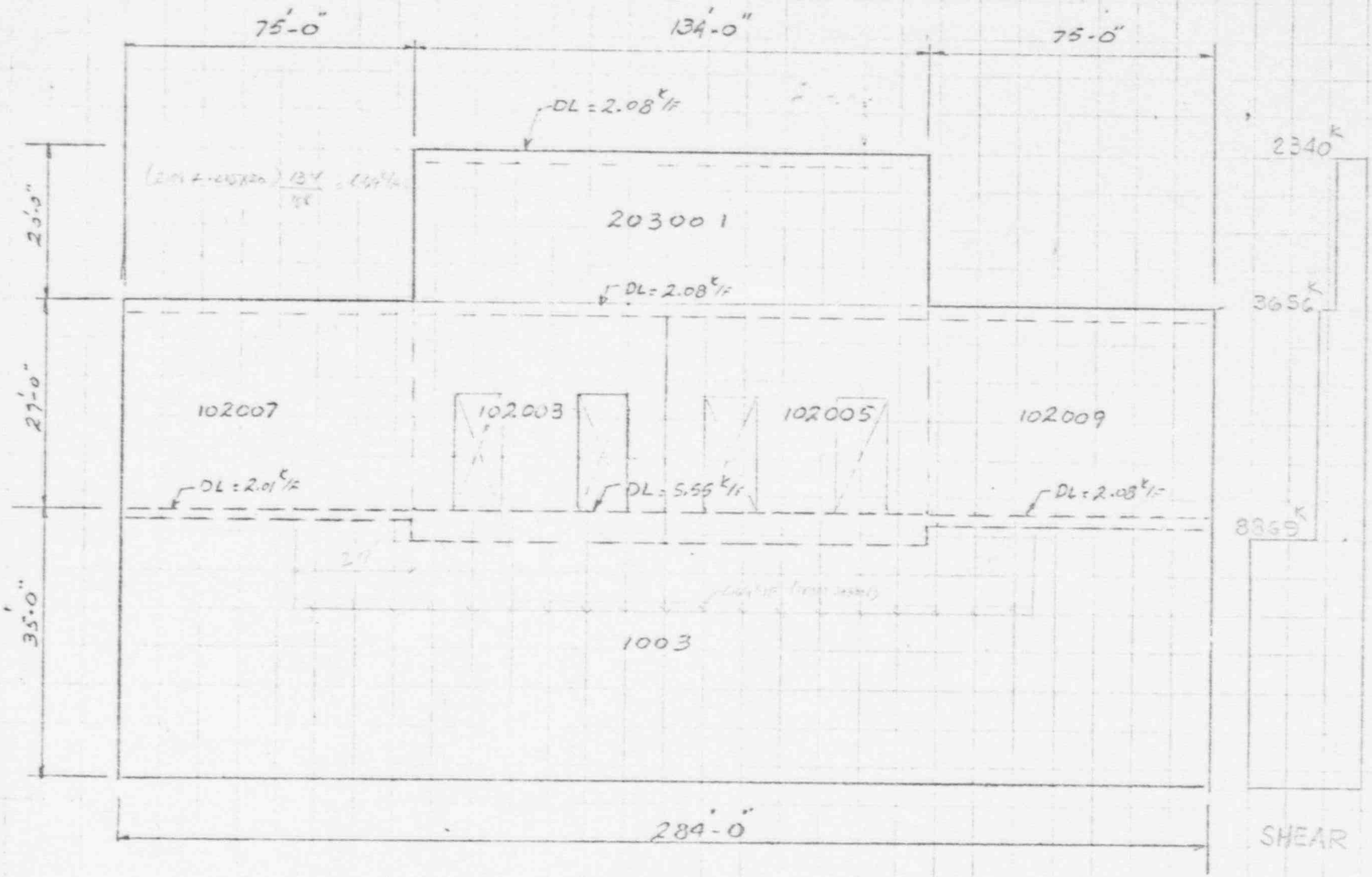

D.L OF SLABS & SUPPORTING WALLS :

	SLAB	WALL
203001	$2.08 \times 134 = 278^k$	$0.225 \times 134 \times 20 = 603^k$ ✓
07 & 102009	$2.08 \times 150 = 312^k$	$0.225 \times 150 \times 27 = 911^k$
003 & 102005	$2.08 \times 134 = 278^k$	$0.225 \times 134 \times 27 = (814^k - 236.5^k) = 577.5^k$
1003	$(2.08)(150) + 5.55(134) = 1055$	$0.525 \times 284 \times 35 = 5218^k$
	1923^k	7309.5 $= 9232.5^k$

OPENING IN MIDDLE SECTION :

$6(6.34 \times 7.17)\ 0.225 = 61^k$

$4(13 \times 15)\ 0.225 = 175.5$

236.5

SARGENT & LUNDY ENGINEERS CHICAGO	Calcs. For SHEAR WALLS IN RHR COMPLEX	Calc. No. [illegible]
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ASSESMENT OF NEW RESPONSE SPECTRA ON ORIGINAL DESIGN

7% SITE DAMPING

these [illegible] are from [illegible] response spectra generations and do not match final results exactly

SPRING NO	SHEAR FORCE KIPS	MOMENT ARM (FT)	MOMENT FT-K	MOMENT AT BOT. WALL	SHEAR STRESS
203001	2340 ✓	20.0	46800 ✓	46800	75 PSI
102003	243 ✓	27.0	6561 ✓		66 PSI
102005	243 ✓	27.0	6561 ✓	145512	66 PSI
102007	1585 ✓	27.0	42795 ✓		127 PSI
102009	1585 ✓	27.0	42795 ✓		127 PSI
1003	8869 ✓	35.0	310415 ✓	455927	73 PSI

INPLANE SHEAR STRESS $= \dfrac{V}{.85 \times L_w \times t} \times \dfrac{1000}{144} \leq 2\sqrt{f'_c} = [illegible]$ PSI

NO ADDITIONAL SHEAR REINFORCEMENTS ARE REQ'D. ✓

Form GQ-3.08.1 Rev. 2

SARGENT & LUNDY ENGINEERS CHICAGO

Calcs. For SHEAR WALLS IN RHR COMPLEX		Calc. No.
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ASSESMENT OF NEW RESPONSE SPECTRA ON ORIGINAL DESIGN

SPRINGS: 102003, 102005, 102007 & 102009 $t = 1'-6"$

USE THE STIFFNESS OF ALL THE SPRINGS ACTING TOGETHER & ALLOW FOR OPENINGS

$L = 2[68+20] = 176'$

$A = 1.5 \times 176 = 264$ SQ.FT

$I = 2\left[\frac{1.5(68)^3}{12} + 1.5(68)(100.95)^2 + \frac{1.5(20)^3}{12} + 1.5(20)(23.5)^2\right]$ SEE CALC HS10 SHT 2(?)

$I = 2471218$ FT^4

FLEXURAL CHECK DUE TO OVERTURNING

IGNOR EFFECT OF VERT. SEISMIC

$\sigma = \frac{145512 \times 142}{2471218} - \left[\frac{2760 - (?)(2760)}{264}\right]$

$\sigma = 8.36 - 7.85 = -0.51$ KSF COMPRESSION

∴ NO ADDITIONAL CORNER STEEL IS REQ'D.

EFFECT OF SLAB MOMENT DUE TO DEAD LOAD = 2.86 "K/FT, (IGNORED)

VERT. STEEL REQ'D DUE TO SHEAR FRICTION:

SPRING 102007 : $A_{VS} = \frac{1585}{0.7 \times 60 \times 68} = 0.55^{\square"} < 1.57^{\square"} - 2\#8$ OK

SPRING 102003 : $A_{VS} = \frac{243}{0.7 \times 60 \times 20} = 0.29^{\square"} < 1.57^{\square"}$ O.K.

* VERT. G VALUE USED

Form GQ-3.08.1 Rev. 2

SARGENT & LUNDY ENGINEERS CHICAGO	Calcs. For SHEAR WALLS IN RHR COMPLEX		Calc. No. [illegible]
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ASSESMENT OF NEW RESPONSE SPECTRA ON ORIGINAL DESIGN

SPRING 203001 $t = 1'-6"$ $L = 134'-0"$

CHECK INPLANE SHEAR STRESS

$V_s = \dfrac{2240}{0.85 \times 134 \times 1.5} \times \dfrac{1000}{144} = 95$ PSI $< 2\sqrt{4000}$

$Av_s = \dfrac{2240}{0.9 \times 60 \times 134} = 0.415$ □"/FT $<$ 2-#5 $A_s = 1.57$□" FURNISHED O.K.

FLEXURAL CHECK

$I = \dfrac{1.5 \times (134)^3}{12} = 300763 \ FT^4$

$N = 278 + 603 - G_V(278 + 603)$ USE $G_V = 0.30$

$= 616.7$

$\sigma = \dfrac{46800 \times 67}{300763} - \dfrac{616.7}{1.5 \times 134} = 7.35$ KSF $< 0.15 f_c$ OK

∴ NO CORNER REINFORCING REQ'D.

TOTAL VERT STEEL REQD $= \dfrac{Av_s}{2} + (A_s)$ DUE TO OUT OF PLANE BENDING

MOMENT FROM FLOOR SLAB $= 15.7$ K-K/FT (FROM CALCS 1.15.10 SHT. 38)

$M_u / \phi f'_c b d^2 = 15.7 / .9(4)(1)(15.5)^2 = 0.018$ $\omega = 0.019$

$\rho = 0.019 \dfrac{4}{60} = 0.00126$

$A_s = 0.00126 \times 12 \times 18 = 0.273$□"/FACE

$(A_s)_{TOTAL} = 0.415/2 + 0.273 = 0.481$□" $<$ 0.77□" FURNISHED O.K.

Form GQ 3.08.1 Rev. 2

SARGENT & LUNDY ENGINEERS CHICAGO

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ASSESMENT OF NEW RESPONSE SPECTRA ON ORIGINAL DESIGN

SPRING 1003 : $L = 284$ $t = 3'-6''$

SHEAR CHECK :

COMBINE TRANSVERSE SHEAR DUE TO SOIL PRESS. WITH SEISMIC

$V_c = \sqrt{V_s + V_t}$

$V_s = \frac{2}{5} wL = \frac{2}{5}(4.22)25 = 47.26^K$ FROM CALC. 1.15.10 SHT. 23

$V_t = \sqrt{\left(\frac{8869}{284}\right)^2 + (47.26)^2} = 56.64^K$

$v_t = \frac{47.26}{.85 \times 1 \times 3.5} \times \frac{1000}{144} = 110.3$ PSI $< 2\sqrt{4000} = 126$

VERT. STEEL BASED ON SHEAR FRICTION

$A_{vs} = \frac{56.64}{0.7 \times 60} = 1.348$ SQ. IN

FLEXURAL CHECK :

$I = \frac{3.5\,(284)^3}{12} = 6681005$ FT^4

$N = 9235 - 0.3(9235) = 6464.5$

$\sigma = \frac{455929 \times 142}{6681005} - \frac{6464.5}{284 \times 3.5}$

$= 9.69 - 6.5 = 3.19$ KSF $< 0.15 f_r$

$\therefore$ NO ADDITIONAL VERT. CORNER STEEL IS REQ'D

Form GQ-3.08.1 Rev. 2

SARGENT & LUNDY
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Calcs. For	SHEAR WALLS IN RHR COMPLEX
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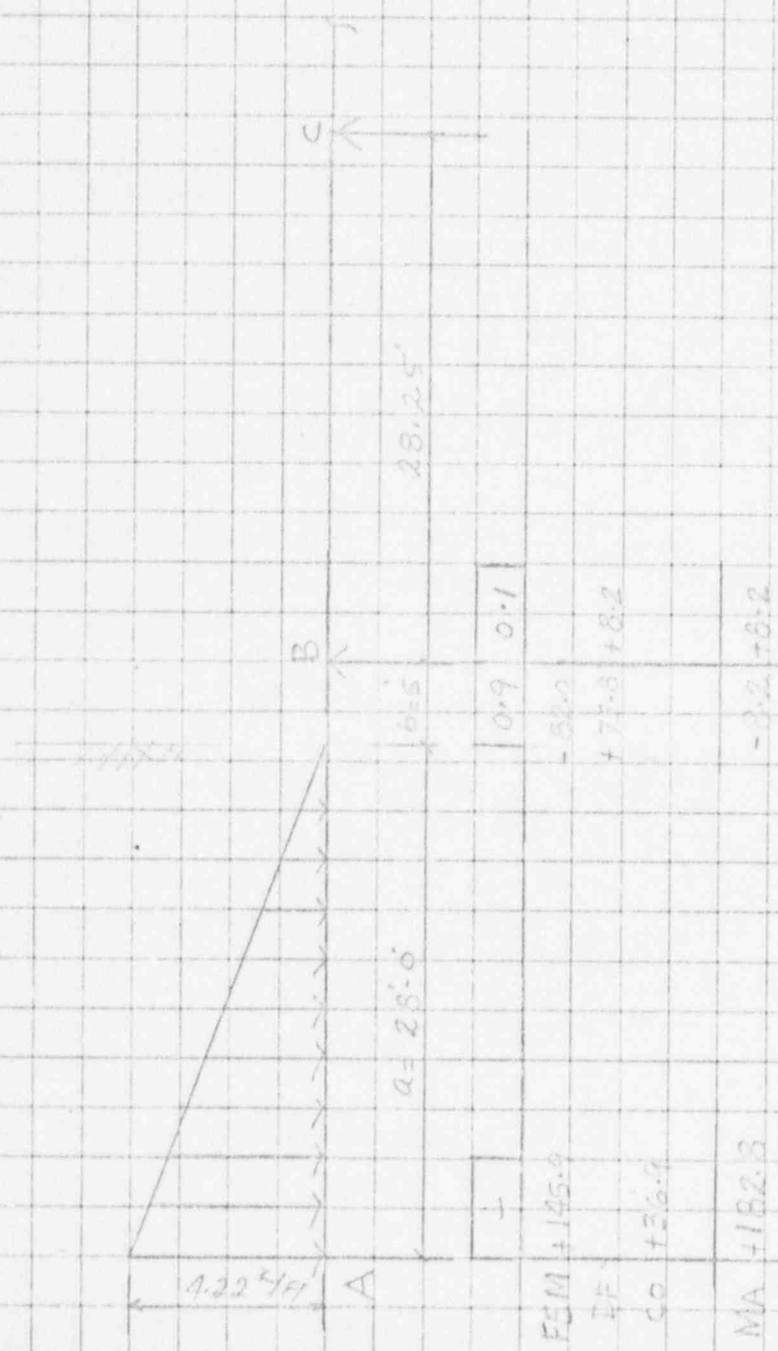
ASSESMENT OF NEW RESPONSE SPECTRA ON ORIGINAL DESIGN

SPRING 100% CONT.

OUT OF PLANE BENDING DUE TO SOIL PRESSURE



VERT. STEEL REQ'D

$Mu/\phi f_c' b d^2$

$\frac{182.8}{0.9(4)(1)39.5^2} = 0.0325$

$\omega = 0.033$

$\rho = 0.033 \frac{4}{60} = 0.0022$

$A_s = 0.0022 \times 12 \times 39.5$

$= 1.042$ SQ.IN / FACE

$(A_s)_{FURNISHED}$ = #11 = 1.56"

SARGENT & LUNDY ENGINEERS CHICAGO

Calcs. For SHEAR WALLS ON ROW "A"		Calc. No. 1-30.1	
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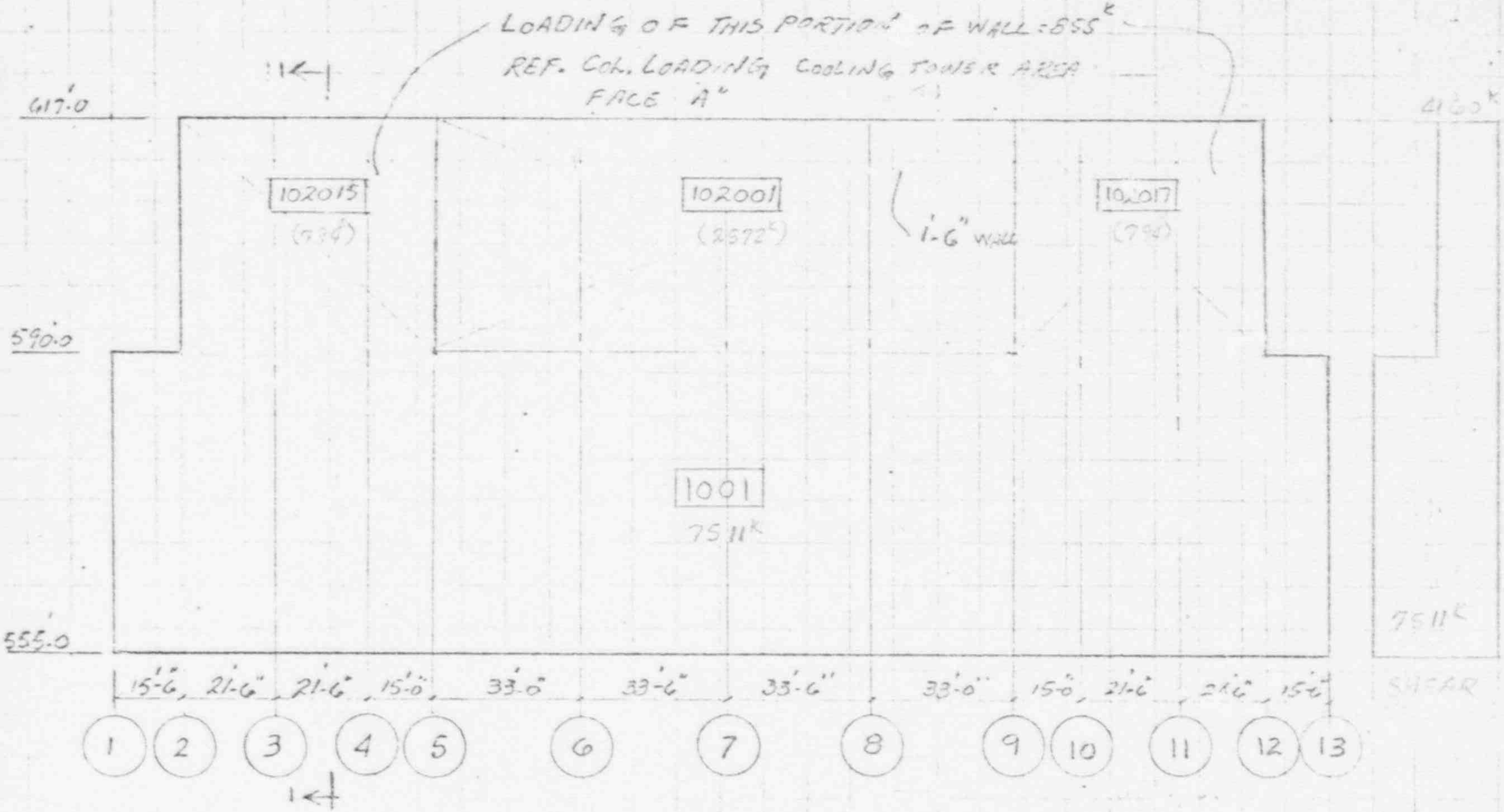

D.L OF WALLS & SLABS

1'-6" SLABS	= 0.225 (11) (133) 2	= 658 k → FOR SLABS @ EL. 590 & 617'
1'-6" WALL	= 0.225 (133) (27)	= 808 k
3'-6" WALL	= 0.525 (284) (35)	= 5218.
	855 x 2	= 1710.
		8394 k

SARGENT & LUNDY ENGINEERS CHICAGO	Calcs. For SHEAR WALLS IN RHR COMPLEX	Calc. No.
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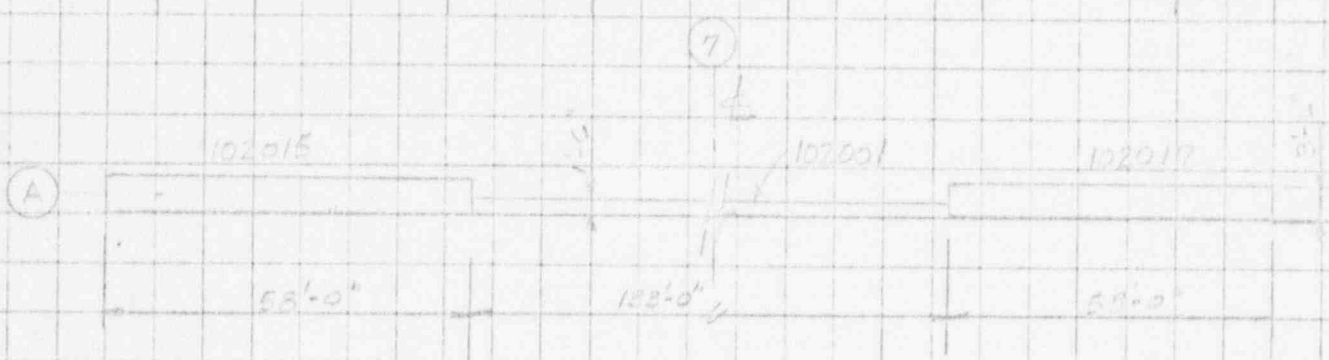
ASSESMENT OF NEW RESPONSE SPECTRA ON ORIGINAL DESIGN

SPRING	t	SHEAR	MOM ARM	MOMENT K-FT	MOM. AT BOT WALL	SHEAR STRESS	L (FT)	(N)
102001	1'-6"	2572^K	27'	69444		105.3 PSI	183	1132
102015	3'-6"	794^K	27'	21438	112320^{K'}	31.95 PSI	58	[illegible]
102017	3'-6"	794	27'	21438		31.95 PSI	58	[illegible]
1001	3'-6"	7511^K	35'	262885	375205^{K'}	62.6 PSI	230	5507



MOMENT OF INERTIA OF THE WHOLE WALL WITHOUT FLANGES

$I = 2\left[\frac{3.5(58)^3}{12} + 3.5(58)(95.5)^2\right] + \frac{1.5(182)^3}{12}$

$= 3816636 + 294079 = 4110716\ FT^4$

CHECK FOR CORNER STEEL REQUIREMENT:

0.3 = g value

$\sigma = \frac{112320 \times 124.5}{4110716} - \left(\frac{2847 - 0.3(2847)}{605.5}\right)$

$A = 1.5 \times 183 + 2 \times 3.5 \times 58 = 605.5\ FT^2$

$= 3.4 - 3.29 = 0.11\ KSF$

∴ NO CORNER STEEL IS REQ'D.

Form GO-3.06.1 Rev. 2

SARGENT & LUNDY ENGINEERS CHICAGO	Calcs. For SHEAR WALLS IN RHR COMPLEX		Calc. No.	
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ASSESMENT OF NEW RESPONSE SPECTRA ON ORIGINAL DESIGN

SPRINGS: 102001, 102015, 102017 CONT.

SHEAR REINFORCING (SHEAR FRICTION) $\mu = 0.7$

FOR 102001 : $A_{s} = \frac{2592}{0.7 \times 133 \times 60} = 0.46^{\square} < 2\text{-}\#8$ O.K.

$A_s = 1.9 \text{ in}^2$

FOR 102015 & 102017 ; $A_{s} = \frac{794}{0.7 \times 55 \times 60} = 0.226^{\square} < 2\text{-}\#11$ O.K.

$A_s = 3.0 \text{ in}^2$

∴ NO FURTHER CHECK IS REQ'D. WALLS ARE O.K.

SPRING: 1001

FLEXURAL CHECK:

$I = \frac{3.5\,(281)^3}{12} = 6402666 \text{ FT}^4$

$\sigma = \frac{325205 \times 40}{6402666} - \left(\frac{8393 - 0.3(8394)}{3.5 \times 281}\right)$

$= 8.2 - 5.11 = 2.29 \text{ KSF} < 0.85 f_r$ O.K.

∴ NO CORNER STEEL IS REQ'D

Form GQ-3.08.1 Rev. 2

SARGENT & LUNDY ENGINEERS CHICAGO	Calcs. For SHEAR WALLS IN RHR COMPLEX	Calc. No. [illegible]
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ASSESMENT OF NEW RESPONSE SPECTRA ON ORIGINAL DESIGN

SPRING : 1001 CONT.

LOCAL BENDING DUE TO SOIL PRESSURE FOR WALL SEC. BETWEEN COL. LINE 2 & 5

MOMENT AT BOT. WALL :

USE MOMENT & SHEAR VALUES FOR SPRING 1003

$M = 182.3^{1-k}$ $(A_S)_{REQ} = 1.042$ SQ. IN /FACE

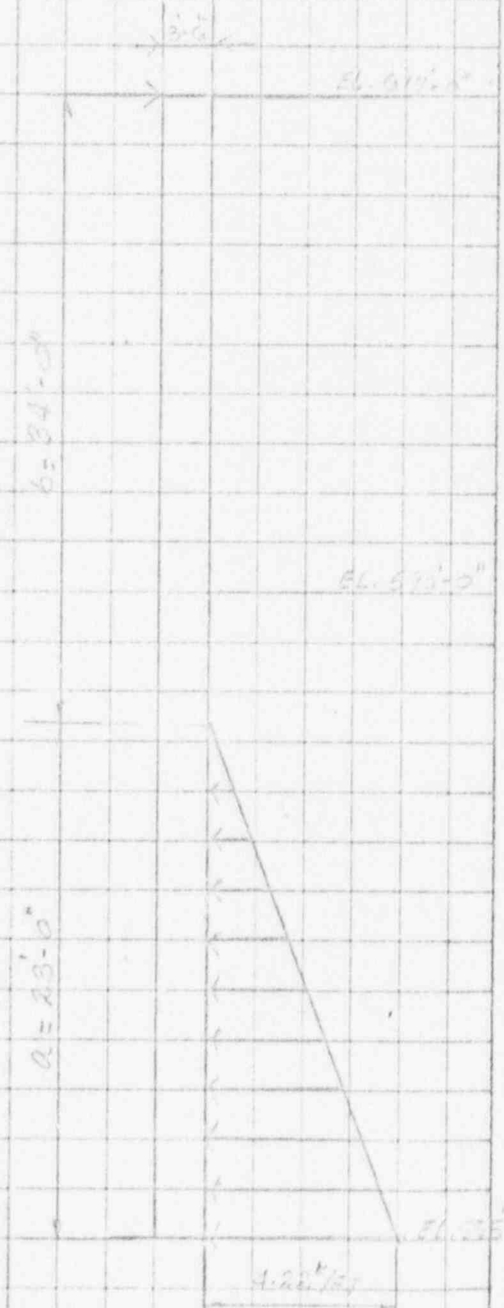
$V_E = 47.26^k$

$V_S = \frac{7511}{280} = 26.82^k$

$V_R = \sqrt{(26.82)^2 + (47.26)^2}$

$= 54.34^k$

$A_{VS} = \frac{54.34}{0.7 \times 60} = 1.29$ SQ IN



Form GQ.3.08.1 Rev. 2

SARGENT & LUNDY
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Calcs. For SHEAR WALLS ON ROW "7"		Calc. No. [illegible]	
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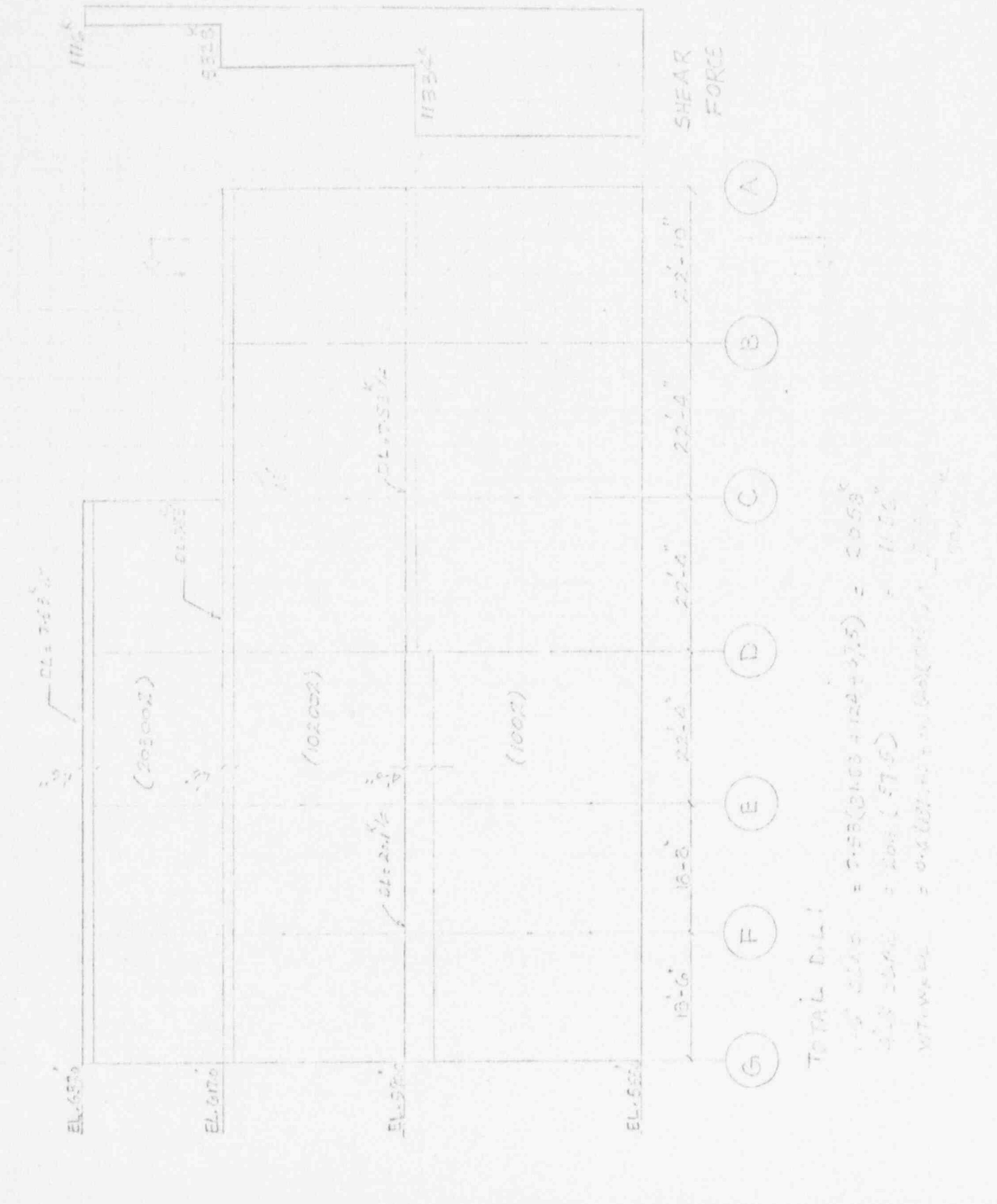

SARGENT LUNDY ENGINEERS CHICAGO

Calcs. For SHEAR WALLS IN EHC COMPLEX

Calc. No. [illegible] 50.1 | Rev. 0 | Date [illegible]

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ASSESSMENT OF NEW RESPONSE SPECTRA ON ORIGINAL DESIGN

SPRING 2030-2 $L = 81.83$ $t = 4'-0"$

$V = 1116^k$, $N = 1273^k$ (FROM CALCS 1.15.10 SHT 58)

$M = 1116 \times 20 = 22320^{'-k}$

$d = 0.8 \times 81.83 = 65.46'$

INPLANE SHEAR STRESS :

$v_s = \frac{1116}{0.85 \times 65.46 \times 4} \times \frac{1000}{144} = 34.82$ PSI $< 2\sqrt{4000}$

$A_{vs} = \frac{1116}{0.9 \times 60 \times 65.46} = 0.41$ SQ.IN $< 2-\#8$ FURNISHED

FLEXURAL CHECK :

$I = \frac{4 (81.83)^3}{12} = 182648$ FT^4

$\sigma = \frac{22320 \times 81.83}{182648 \times 2} - \frac{1273}{4 \times 81.83}$

$\sigma = 5.0 - 3.9 = 1.1$ KSF $< 0.15 f_c' = 10.2$ KSF O.K

Form GQ-3.03.1 Rev. 2

SARGENT & LUNDY ENGINEERS CHICAGO	Calcs. For SHEAR WALLS IN RHR COMPLEX		Calc. No. 1.3??.1	
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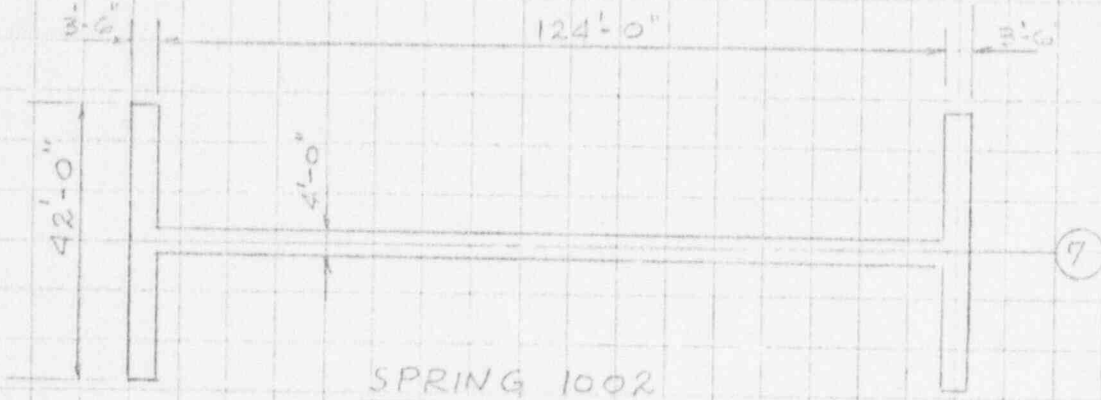
ASSESSMENT OF NEW RESPONSE SPECTRA ON ORIGINAL DESIGN



SPRING 1002

$V = 11334^{k}$

$M = 1116(20) + 5328(27) + 11334(35) = 562866^{'k}$

$A = 4 \times 124 + 2(3.5)42 = 790\ FT^2$

$I = 4\dfrac{(124)^3}{12} + 2\left[3.5 \times 42 \times 63.75^2\right] = 1830375\ FT^4$

$N = 10052^{k}$ (FROM CALCS 1.15.10 SHT. 52)

$\sigma = \dfrac{562866 \times 65.5}{1830375} - \left(\dfrac{10052 - 0.3^{*}(10052)}{790}\right)$ * GV USED

$= 20.14 - 8.90 = 11.24\ KSF > 0.15 f_r = 10.2\ KSF$ 10% OVERSTRESS

∴ REDUSE GV. RESULTS SHOULD BE O.K

INPLANE SHEAR STRESS:

$V_s = \dfrac{11334}{0.85 \times 124 \times 4} \times \dfrac{1000}{144} = 186.68\ PSI < 10\sqrt{4000} = 632\ PSI$ ✓

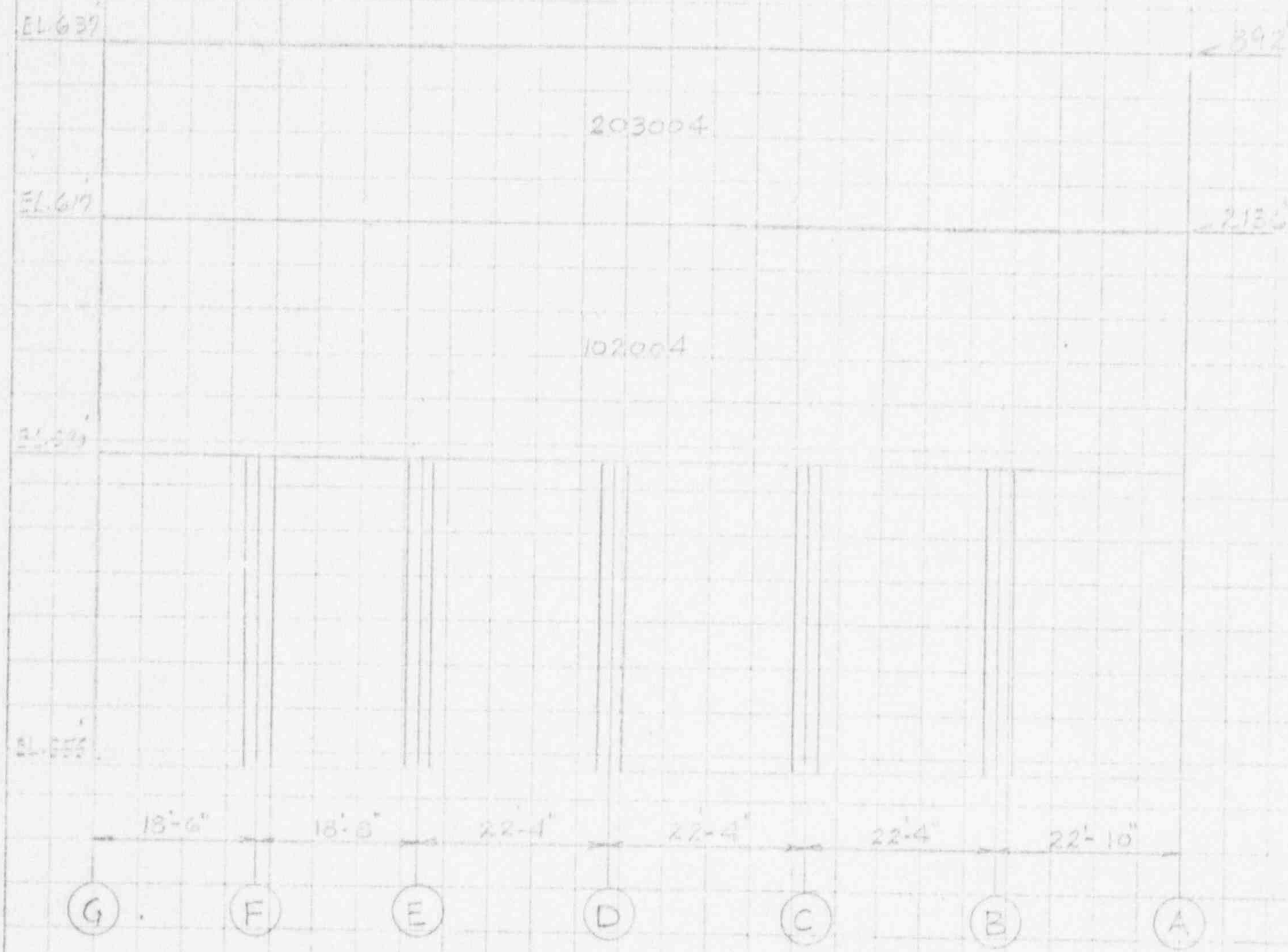
$A_{vs} = \dfrac{11334}{0.9 \times 124 \times .85 \times 60} = 2.56^{\square'} < 3.12^{\square'}$ FURNISHED O.K

Form GO-3.09.1 Rev. 2

SARGENT & LUNDY ENGINEERS CHICAGO	Calcs. For SHEAR WALLS IN EF2 COMPLX	Calc. No. 1.30.1
	X Safety-Related / Non-Safety-Related	Rev. 2 Date 5-5-81
		Page 19 of
Client DECO	Prepared by M. KHAYYAN	Date
Project FERMI-2	Reviewed by	Date
Proj. No. 6171-38 Equip. No.	Approved by	Date

ASSESSMENT OF NEW RESPONSE SPECTRA ON ORIGINAL DESIGN

WALL ON COL. LINE "5" OR "9"



MOMENT AT EL. 590.0 :

$M = 892(20) + 2136(27) = 75512^{\text{'k}}$

INPLANE SHEAR STRESS $= \dfrac{2136}{0.85 \times 127 \times 1.5'} \times \dfrac{1000}{144} = 91.6$ PSI $< 2\sqrt{4000}$ O.K.

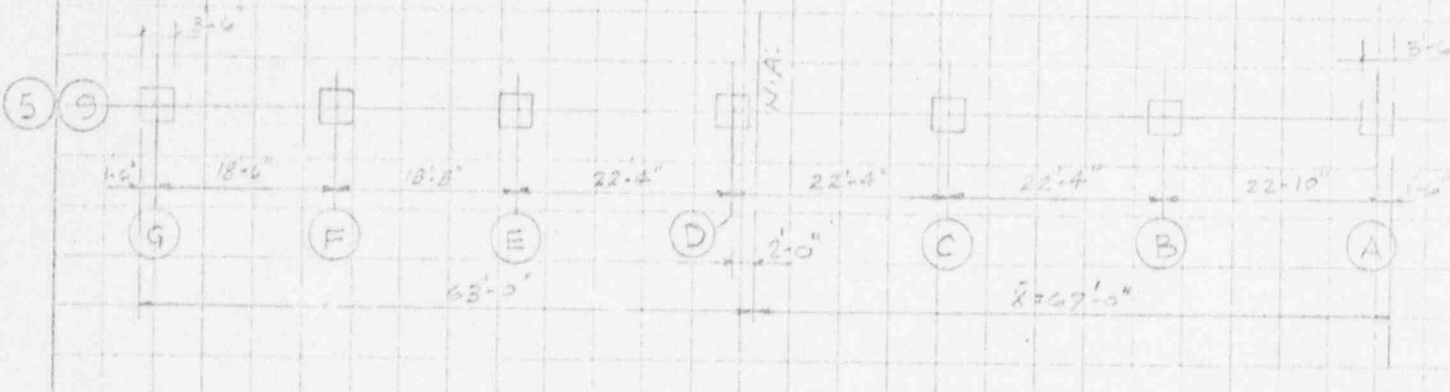
Form GO-3.03.1 Rev. 2

SARGENT & LUNDY ENGINEERS CHICAGO	Calcs For SHEAR WALLS IN RHR COMPLEX	Calc. No. 1.30.1
	X Safety-Related / Non-Safety-Related	Rev. 0 Date 5-15-81
		Page 20 of

Client DECO	Prepared by M. KHAYYMIZ	Date 5/15/81
Project FERMI-2	Reviewed by [illegible]	Date 5-14-81
Proj. No. 6130-33 Equip. No.	Approved by	Date

ASSESSMENT OF NEW RESPONSE SPECTRA ON ORIGINAL DESIGN

3'x3' COL. AREA = 9□' , END COLS AREA = 3x3.5 = 10.5□'



TOTAL AREA = 2(10.5) + 5(9) = 66□'

$$\bar{X} = \frac{10.5(127) + 9(110 + 91.33 + 69 + 46.67 + 24.33) + 10.5(1.5)}{66}$$

$$= 66.988' \quad \text{SAY } 67'$$

$$I_{(COL)} = \frac{3(3)^3}{12} = 6.75\,FT^4 \quad I_{(3\times3.5)} = \frac{3\,(3.5)^3}{12} = 10.71\,FT^4$$

$$I_{N.A} = 10.71 + 10.5(61.5)^2 + 5(6.75) + 9\left(43^2 + 24.33^2 + 2^2 + 20.34^2 + 40.67^2\right) + 10.71 + 10.5(63.5)^2$$

$$= 122721.8\ FT^4$$

IGNOR COL. LOADS + FIND UPLIFT ON EACH COL DUE TO SEISMIC MOMENT

$$\sigma_G = \frac{75512 \times 63}{122721.8} = 38.76\ KSF \qquad \sigma_C = \frac{75512 \times 20.34}{122721.8} = 12.51\ KSF$$

$$\sigma_F = \frac{75512 \times 43}{122721.8} = 26.45\ KSF \qquad \sigma_B = \frac{75512 \times 42.67}{122721.8} = 26.25\ KSF$$

$$\sigma_E = \frac{75512 \times 24.33}{122721.8} = 14.97\ KSF \qquad \sigma_A = \frac{75512 \times 67}{122721.8} = 41.22\ KSF$$

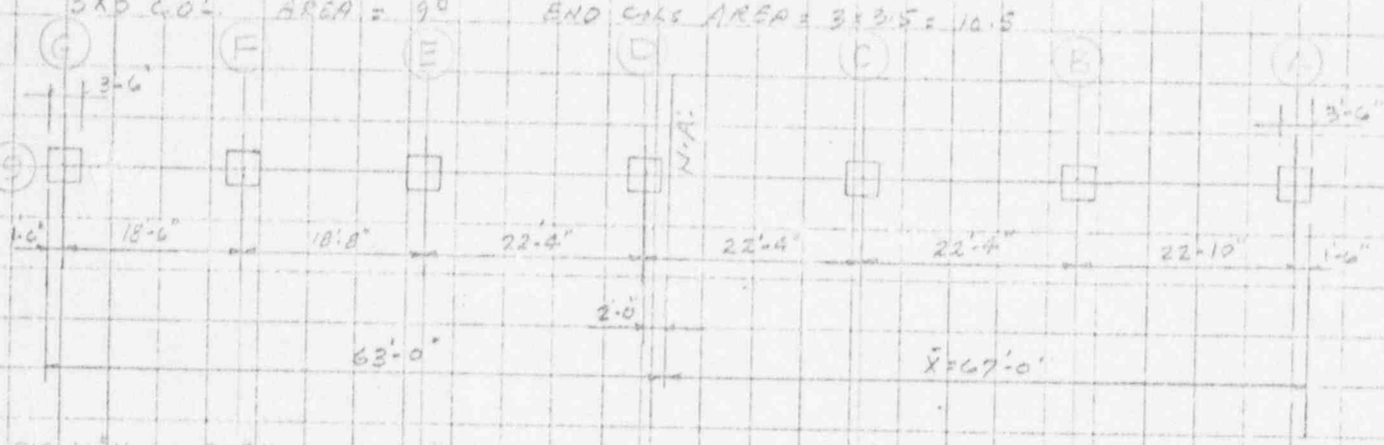
Form GQ-3.08.1 Rev. 2

SARGENT & LUNDY ENGINEERS CHICAGO	Calcs. For SHEAR WALLS IN RHR COMPLEX	Calc. No. 1.3.1
	X Safety-Related / Non-Safety-Related	Rev. 0 Date
		Page 21 of 5-16-81

Client DECO	Prepared by M. Khayyata Date 4/1/81
Project FERMI-2	Reviewed by Date
Proj. No. G139-38 Equip. No.	Approved by Date

ASSESSMENT OF NEW RESPONSE SPECTRA ON ORIGINAL DESIGN

3'x3' COL. AREA = 9□' END COLS AREA = 3×3.5 = 10.5



	G	F	E	D	C	B	A
NO. BAR	16 #11	20 #11	20 #11	20 #11	20 #11	20 #11	16 #11
As	24.96□"	31.2□"	31.2□"	31.2□"	31.2□"	31.2□"	24.96□"
CAP.	1422k	1778k	1778k	1778k	1778k	1778k	1422k
SEISMIC T×A	1407k	1239k	1131.7k	NEGLIGIBLE	± 112.6k	± 296.3k	± 632.6k
(COL.)DL		966.4k	381.2	955.3k	568.6k	403.1k	

TO GET THE NET UPLIFT/COL, THE COL. LOAD SHOULD BE DEDUCTED FROM THE TENSILE FORCE T

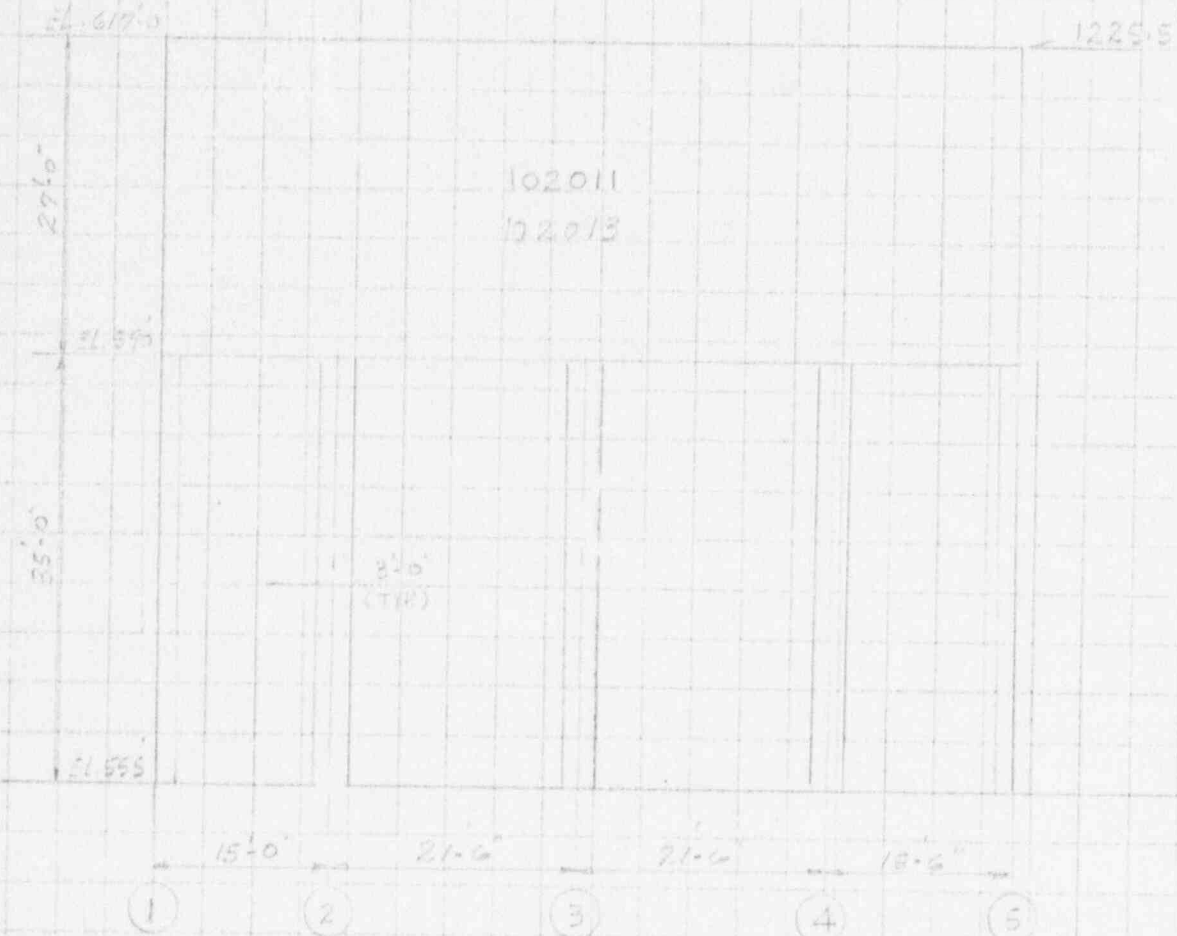
FROM THE ABOVE IT CAN BE SEEN THAT THERE IS NO UPLIFT ON THE COLUMNS.

VERTICAL SEISMIC IS IGNORED

SARGENT & LUNDY ENGINEERS CHICAGO	Calcs. For SHEAR WALLS IN CAR COMPT. ON ROW "F"	Calc. No. 1.30.1
	X Safety-Related / Non-Safety-Related	Rev. 0 Date 5-15-81
		Page 22 of

Client DECO	Prepared by M. KAKOVATIC	Date [illegible]
Project FERMI-2	Reviewed by G. Ahuja	Date 5-[illegible]-81
Proj. No. 5179-38 Equip. No.	Approved by	Date

ASSESSMENT OF NEW RESPONSE SPECTRA ON ORIGINAL DESIGN



(COL) DL $\quad 747^k \quad 837^k \quad 768.6^k \quad 881.2^k$

$M = 1225.5 \times 27 = 33088.5^{'-k}$

$I = I_{col} + ad^2$

$= 5(6.75) + 2 \times 9\,(36.5^2 + 21.5^2) = 32334.75\ Ft^4$

(CONT.)

SARGENT & LUNDY ENGINEERS CHICAGO

Calcs. For SHEAR WALLS IN RHR COMPTS ON E.W. ... | Calc. No. 1.30.1 | Rev. 0 | Date ...
X Safety-Related | Non-Safety-Related | Page 23 of FINAL

Client DECO | Prepared by M. ... | Date ...
Project EF2/MI-2 | Reviewed by ... | Date ...
Proj. No. 6130-?0 Equip. No. | Approved by | Date

ASSESSMENT OF NEW RESPONSE SPECTRA ON ORIGINAL DESIGN

SPRINGS 102011, 102013 CONT. (IGNORE VERT. SEISMIC)

NET COL UPLIFT $= A_{COL}\left[\frac{M \times C}{I}\right] -$ COL. LOAD

$= \frac{9(33088.5 \times 36.5)}{32334.95} - 881.2$

FOR COL. E5 $= 336 - 881.2 = -545^{K}$ COMP.

FROM THE ABOVE, IT CAN BE SEEN THAT ALL COLS ARE IN COMPRESSION

INPLANE SHEAR STRESS ASSUMING UNIFORM WALL THICKNESS:

$t = 1'-6"$

$V_s = \frac{1225.5}{0.85 \times 73 \times 1.5} \times \frac{1000}{144} = 91.43$ PSI $< 2\sqrt{4000}$ O.K.

EFFECT OF SEISMIC MOMENT ON COL. F2 TO BE CONSIDERED WITH BASE MAT CALCULATIONS

$F_2 = \frac{9(33088.5 \times 21.5)}{32334.95} = 198^{K}$

Form GQ-3.08.1 Rev. 2

SARGENT & LUNDY
ENGINEERS

DESIGN CONTROL SUMMARY
DESIGN VERIFICATION

PAGE 1

PROJECT NAME ENRICO F[illegible] MI
PROJECT NO. [illegible] UNIT NO. 2
CLIENT DETROIT EDISON CO
CALC. NO. & DESCRIPTION [illegible]
CALC. FOR [illegible] TRAY HANGERS [illegible]

☒ SAFETY RELATED ☐ NON SAFETY RELATED

COMMENT NO.
QA SERIAL NUMBER

SIGNATURE & DATE FOR REV.			IDENTIFICATION OF PAGES PREPARED/REVISED/VOIDED & REVIEW METHOD
REV. 0	PREPARER	J. Nandi 5-5-81	Prepared pages 1.1, 1.3, 1.4, 1.5 & 1.7
	REVIEWER	TKonstantellos 5/16/81	
	APPROVER	N.V. Hingorani 5-16-81	REVIEW METHOD Detail Review Method
REV 0	PREPARER	TKonstantellos 5/16/81 W. Allen Walker 5/16/81	Prepared pages 1.2, 1.8 & 1.9
	REVIEWER	J. Nandi 5-5-81	
	APPROVER	N.V. Hingorani 5-16-81	REVIEW METHOD Detail Review Method
REV 0	PREPARER	W. Allen Walker 5/16/81	Prepared pages 1.6
	REVIEWER	TKonstantellos 5/16/81	
	APPROVER	N.V. Hingorani 5-16-81	REVIEW METHOD DETAIL REVIEW METHOD
REV 1	PREPARER	J. Nandi 5-21-81	Page 3 Rev. 1 of Design Control Summary
	REVIEWER	TKonstantellos 5-21-81	
	APPROVER	N.V. Hingorani 5-21-81	REVIEW METHOD DETAILED

FORM SAS 22.1 Rev. 0

SARGENT & LUNDY ENGINEERS

DESIGN CONTROL SUMMARY
DESIGN ELEMENT INDEX
PROJECT NO. [illegible] UNIT NO. 2

PAGE 0 OF CALCULATIONS NO. [illegible] REV. 0 DATE [illegible]

Revision

FORM SAD 22.3 Rev. 0

Revision

All the assumptions, procedures, loading and review method are same as outlined in Calculation # EE 0013 for Reactor-Auxilliary Building except for seismic response spectra.

Response spectra were obtained from Calc. Book # SDD-DEC0-001 & enveloped at 10% damping for SSE condition with 7% damping site spectra for RHR Complex.

Summary of the sample 11% of approximate 450 hangers shows that about 94% of the hangers are within the allowable stress limits.

The loads on RHR cable trays is much less than the design load. It is expected that the with actual tray loads it should be possible to qualify the remaining hangers. 1

FORM SAS 22.7 Rev. 0

SARGENT & LUNDY ENGINEERS

DESIGN CONTROL SUMMARY
DESIGN INPUT DOCUMENTS
PROJECT NO. [illegible] UNIT NO. 2

PAGE 4 OF CALCULATIONS NO. EE 0014 REV. 0 DATE 5-15-1981

NO.	INPUT DOCUMENT			REVISION NUMBER OR DATE ON LINE INDICATED								USED IN DESIGN OF	CNT	C/N
	DESCRIPTION	SOURCE	IDENT	DATE	S	DATE	S	DATE	S	DATE	S			
1.	SEISMIC RE-ANALYSIS FOR 5% & 7% DAMPING SITE SPECTRA RHR COMPLEX	S&L	REPORT # SLI-DEED-00[illegible]											
2.	SPEC. FOR DESIGN OF COLD-FORMED STEEL STRUCTURAL MEMBERS	AISI	–	1968 EDITION										
3.	UNISTRUT GENERAL ENGINEERING CATALOG	UNISTRUT CORP.	CATALOG # 9	1978										
4.	S&L COMPUTER PROGRAM "SEISHANG"	S&L	PROGRAM # 09718121 Ø	1981										

CNT - Continued CN - Comment Number S - Status

SARGENT & LUNDY ENGINEERS CHICAGO	Calcs. For RHR Complex Std. Cable Tray Hgrs. Review for New Response Spectra	Calc. No. EE 0014
	X Safety-Related / Non-Safety-Related	Rev. 0 Date 5/6/81
		Page 1 of

Client D. E. Co.		Prepared by [illegible]	Date 5/6/81
Project FERMI-2		Reviewed by [illegible]	Date 5/7/81
Proj. No. 6139-38	Equip. No.	Approved by	Date

SUMMARY

ELEV.	NO. OF HGRS. REVIEWED	NO. OF HANGERS O.K.	NO. OF HANGERS OVERSTRESSED	PERCENT OVERSTRESS 0-10%	10-25%	25% & OVER
617'-0"	34	33	1	-	-	1
637'-0"	16	14	2	1	1	—
TOTAL	50	47	3	1	1	1

NOTE :- 1. ABOVE IS A SUMMARY OF SAMPLE STANDARD HANGERS IN RHR COMPLEX

2. SAME AS FOR Rx-AUX-BLDG (CALC.# EE0013)

3. D.L. = 40 PSF. FOR ALL AREAS.

4. SAME AS Rx-AUX. BLDG. (CALC.# EE0013).

5. " " " " "

6. " " " " "

7. " " " " "

Form GQ 3.08.1 Rev. 2

SARGENT & LUNDY
ENGINEERS
CHICAGO

Calcs. For RHR EL. 617'-0" D(?) SEIS. RESP SPECTRA REVIEW		Calc. No.	
		Rev. 0	Date 4/21/81
Safety-Related	Non-Safety-Related	Page 2	of

Client D.E. Co.		Prepared by [illegible]	Date 4/21/81
Project FERMI-2		Reviewed by J. Nardi	Date 5/5/81
Proj. No. 6139-38	Equip. No.	Approved by	Date

The following Hangers represent 7% of existing ones at el. 617'-0", and they have been analysed for new seismic response spectra.

HANGER	A	B	C	TRIB. SPAN	DWG.	EXIST. SIZES			OVERSTRESS LEVEL
						H	V	D	
12H1	1'-9"	5'-0"	5'-0"	5'-6"	5E721N-55A 58A	0	0	-	O.K.
13H1	4'-8"	4'-6"	6'-0"	3'-6"	" - 55A 58A	0	0	-	O.K.
13H2	2'-9"	4'-6"	6'-0"	4'-0"	" - 55A 58A	0	0	—	O.K.
12H2	2'-4"	7'-6"	7'-6"	4'-0"	" - 55A 58A	0	0	-	O.K.
112H1	5'-0"	4'-6"	5'-6"	37'-0"	" - 55A 58A	0	0	-	O.K.
14H1	1'-4"	4'-6"	7'-6"	5'-5"	" - 55A 58A	0	0	-	87.8%
19H1	1'-5"	2'-0"	5'-0"	4'-3"	" - 55A 58A	0	0	—	O.K.

HAND CALC'S FOLLOW FOR 7HVG, 33H1, 54H1 & 1HV1 HGRS.

Form GO-3.08.1 Rev. 2

SARGENT & LUNDY
ENGINEERS
CHICAGO

Calcs. For EHR EL 653'-0" — ELEC. HANGER REVIEW PER NEW RESP. SPECTRA

x Safety–Related | Non–Safety–Related

Calc. No. [illegible]
Rev. 0 | Date [illegible]
Page 13 of

Client D. E. Co.
Project FERMI-2
Proj. No. 6139-38 Equip. No.

Prepared by J. Mayer | Date [illegible]
Reviewed by [illegible] | Date 4/23/81
Approved by | Date

Hanger for checking ... response spectra are listed below.

Hanger #	A	B	C	TRIB. SPAN	DWG. #	EXIST. STRS. H	EXIST. STRS. V	EXIST. STRS. D	LEVEL OF OVERSTRESS
2H1	4'-0"	6'-9	9'-0	5'-0"	5E721N-61A	0	1	0	15.4% [illegible]
2H11	3'-6	6'-0	7'-6	4'-0"	" -61B	1	1	1	O.K.
2H12	7'-0	6'-9	7'-6	4'-0"	" -61B	1	1	1	3%
13H5	1'-0	5'-11½	8'-3"	2'-0"	" -61A	0	0	–	O.K.
2H6	2'-0"	4'-9	7'-0	5'-0"	" -61A	0	1	0	O.K.
3H11	2'-4"	6'-0	9'-0	4'-9"	" -61B	1	4	1	O.K.
3H6	4'-7	6'-0	9'-0	5'-0"	" -61B	1	4	1	O.K.
4H1	1'-1	9'-0	9'-0	5'-0"	" -61C	0	0	0	O.K.
4H5	1'-4	6'-2	6'-2	5'-0"	" -60C	0	0	0	O.K.
14H3	4'-4	2'-3	4'-9	1'-6, 3'-0, 4'-9"	" -61B	0	0	–	O.K.
14H2	4'-4	3'-9	6'-3	2'-6, 3'-9, 5'-6"	" -61B	0	0	–	O.K.
1H1	1'-4	3'-9	3'-9	6'-2	" -61B	0	0	0	O.K.
1H2	1'-4	5'-0	5'-0"	5'-0"	" -61B	0	0	0	O.K.
14H1	6'-1	3'-6	6'-6	5'-5"	" -60B	1	4	–	O.K.

SARGENT & LUNDY ENGINEERS CHICAGO

Calcs. For	RHR EL. 637'-0" - ELEC. HANGER REVIEW PER NEW RISP. CRITERIA
Safety-Related	Non-Safety-Related
Calc. No.	EE 301-?
Rev.	0
Date	4/17/81
Page	1.? of
Client	D.E.CO.
Project	FERMI-2
Proj. No.	6139-38
Equip. No.	
Prepared by	J. Nomi
Date	4/17/81
Reviewed by	T. Kountouritellos
Date	4/23/81
Approved by	
Date	

Hanger				TRIB. SPAN		H	V	D	
13H3	2'-6"	5'-11¾	8'-5"	2'-0"	SETRIN -60B	0	0	-	O.K.
3H4	2'-4	6'-0	9'-0"	5'-0"	-60B	1	4	1	O.K.

CONCLUSION:-

TOTAL NO. OF HGRS. ANALYZED = 16

" " " O.K. = 14

Form GQ-3.08.1 Rev. 2

SARGENT & LUNDY ENGINEERS CHICAGO	Calc. For RESPONSE SPECTRA FOR RHR BLDG. ELEC. CABLE TRAY HGRS	Calc. No. EC0014
	☐ Safety-Related ☐ Non-Safety-Related	Rev. 0 Date 5/7/81
		Page 15 of

Client D.E.CO.		Prepared by J. Nardi	Date 5/2/81
Project FERMI-2		Reviewed by T. Konstantellos	Date 5/5/81
Proj. No. 6139-38	Equip. No.	Approved by	Date

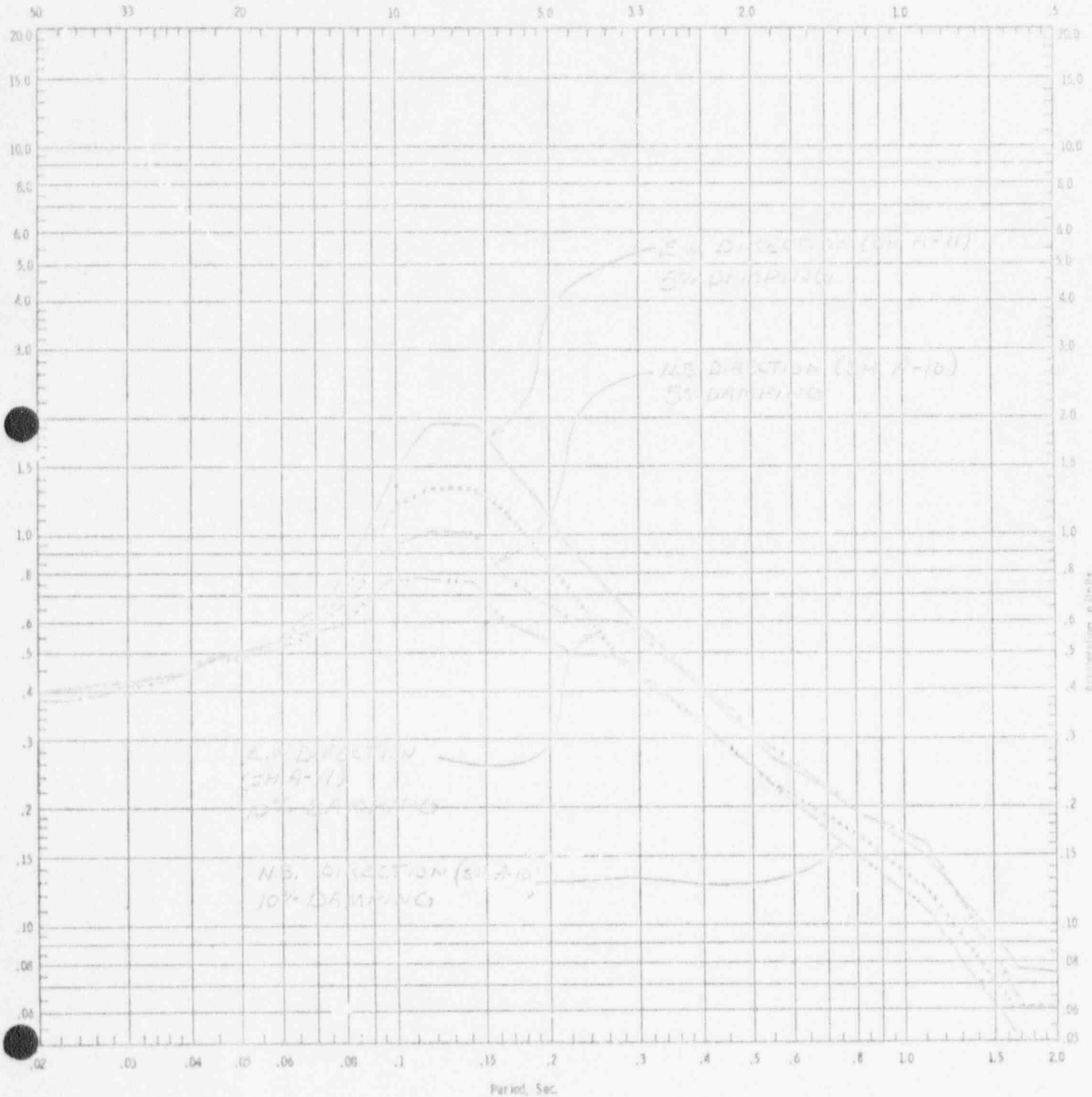
```
 4:       21              14
 5:HORIZ. SPECTRUM AT EL 617-0
 6:     .010     .400
 7:     .020     .400
 8:     .038     .430
 9:     .044     .490
10:     .074     .700
11:     .100    1.200
12:     .120    1.300
13:     .150    1.300
14:     .170    1.200
15:     .200     .850
16:     .270     .490
17:     .280     .480
18:     .300     .440
19:     .570     .220
20:     .800     .170
21:    1.100     .120
22:    1.300     .090
23:    1.500     .070
24:    1.700     .060
25:    2.000     .060
26:   20.000     .060
27:VERT.  SPECTRUM AT EL 617-0
28:     .002     .500
29:     .020     .500
30:     .028     .590
31:     .032     .900
32:     .039     .900
33:     .043    1.150
34:     .060    1.150
35:     .068     .900
36:     .073     .900
37:     .120     .430
38:     .380     .180
39:     .520     .100
40:    2.000     .100
41:   20.000     .100
```

```
 1:       20              21
 2:HORIZ. SEISMIC SPECTRA AT ELEV.637-0
 3:     .010     .530
 4:     .020     .530
 5:     .030     .540
 6:     .040     .600
 7:     .045     .610
 8:     .060     .710
 9:     .065     .790
10:     .070     .800
11:     .080     .900
12:     .085     .950
13:     .100    1.300
14:     .120    1.400
15:     .150    1.400
16:     .230     .700
17:     .270     .540
18:     .800     .220
19:    1.100     .120
20:    1.700     .060
21:    2.000     .060
22:   20.000     .060
23:VERT.  SEISMIC SPECTRA AT ELEV.637-0
24:     .001     .400
25:     .020     .400
26:     .022     .500
27:     .032     .760
28:     .036     .780
29:     .039    1.020
30:     .060    1.020
31:     .066     .900
32:     .080    1.400
33:     .120    1.400
34:     .132    1.200
35:     .150     .900
36:     .180     .425
37:     .220     .240
38:     .360     .210
39:     .600     .100
40:     .800     .100
41:    1.000     .100
42:    1.500     .100
43:    2.000     .100
44:   20.000     .100
```

SARGENT & LUNDY ENGINEERS CHICAGO	SPECTRA FOR	Calc. No.
	[X] Safety-Related / [ ] Non-Safety-Related	Rev. 0 Date
		Page of

Client D. E. Co.	Prepared by / Date
Project FERMI - 2	Reviewed by / Date
Proj. No. 6139-38 Equip. No.	Approved by / Date

Frequency, CPS RESPONSE SPECTRA GENERATED ON 4-14-81



EXCITATION HORIZ. SSE AT 10% DAMPING LOCATION: RHR BLDG.

SPECTRA NO A-10 & A-11 ELEVATION: 617'-0"

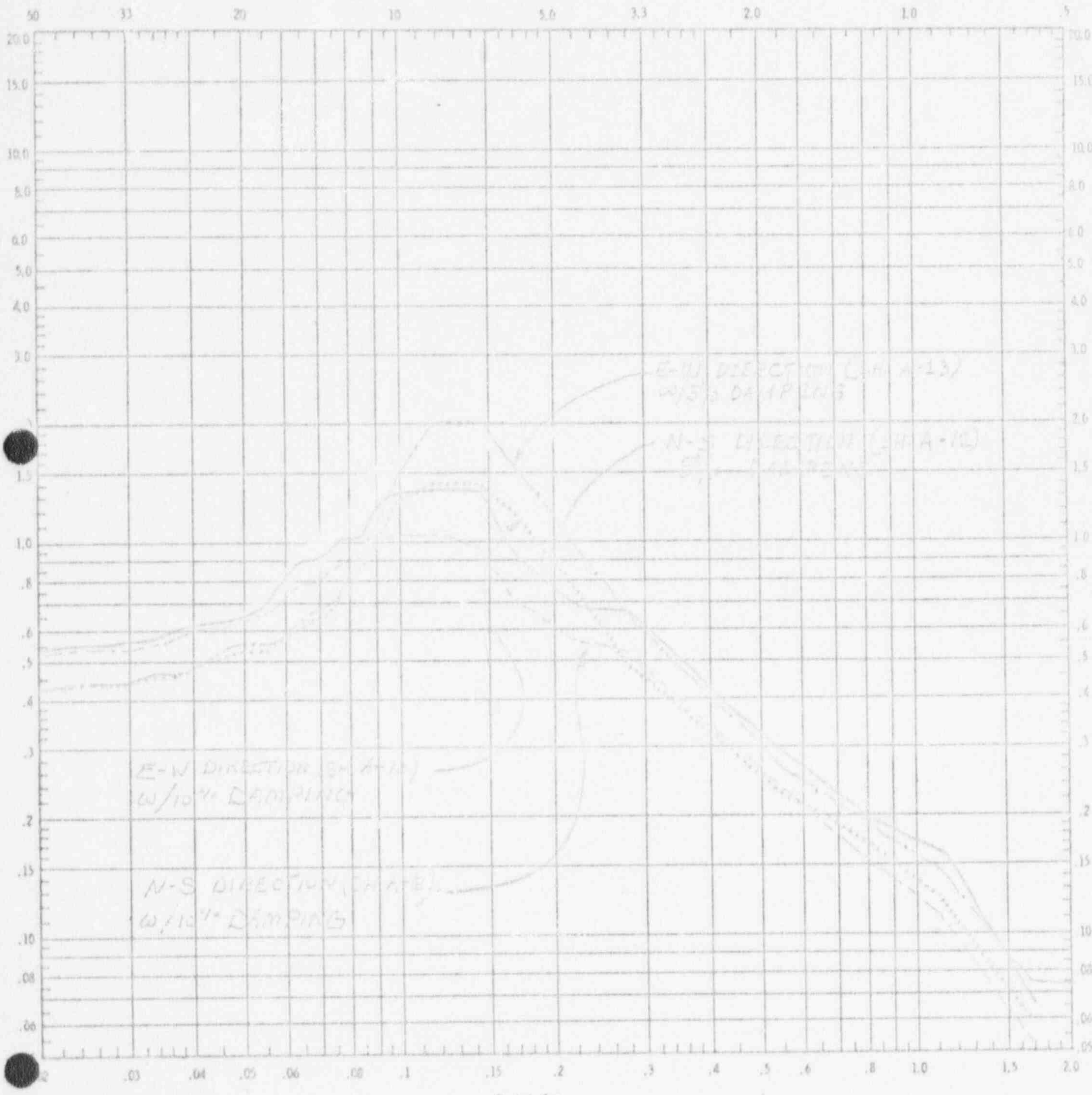
BPD

SARGENT & LUNDY
ENGINEERS
CHICAGO

SPECTRA FOR RHR EL 637' SLAB	Calc. No. [illegible]	
HORIZ. SPECTRA ENVELOPE	Rev. 0	Date [illegible]
✓ Safety-Related	Non-Safety-Related	Page [illegible] of

Client D.E.Co.	Prepared by [illegible]	Date [illegible]
Project FERMI-2	Reviewed by [illegible]	Date [illegible]
Proj. No. 5139-38 Equip. No.	Approved by	Date

Response Spectra Generated on 4/14/81



EXCITATION HORIZ. SSE @ 10% DAMPING

LOCATION: RHR @ 637'-0"

SPECTRA NO A-12 & A-13

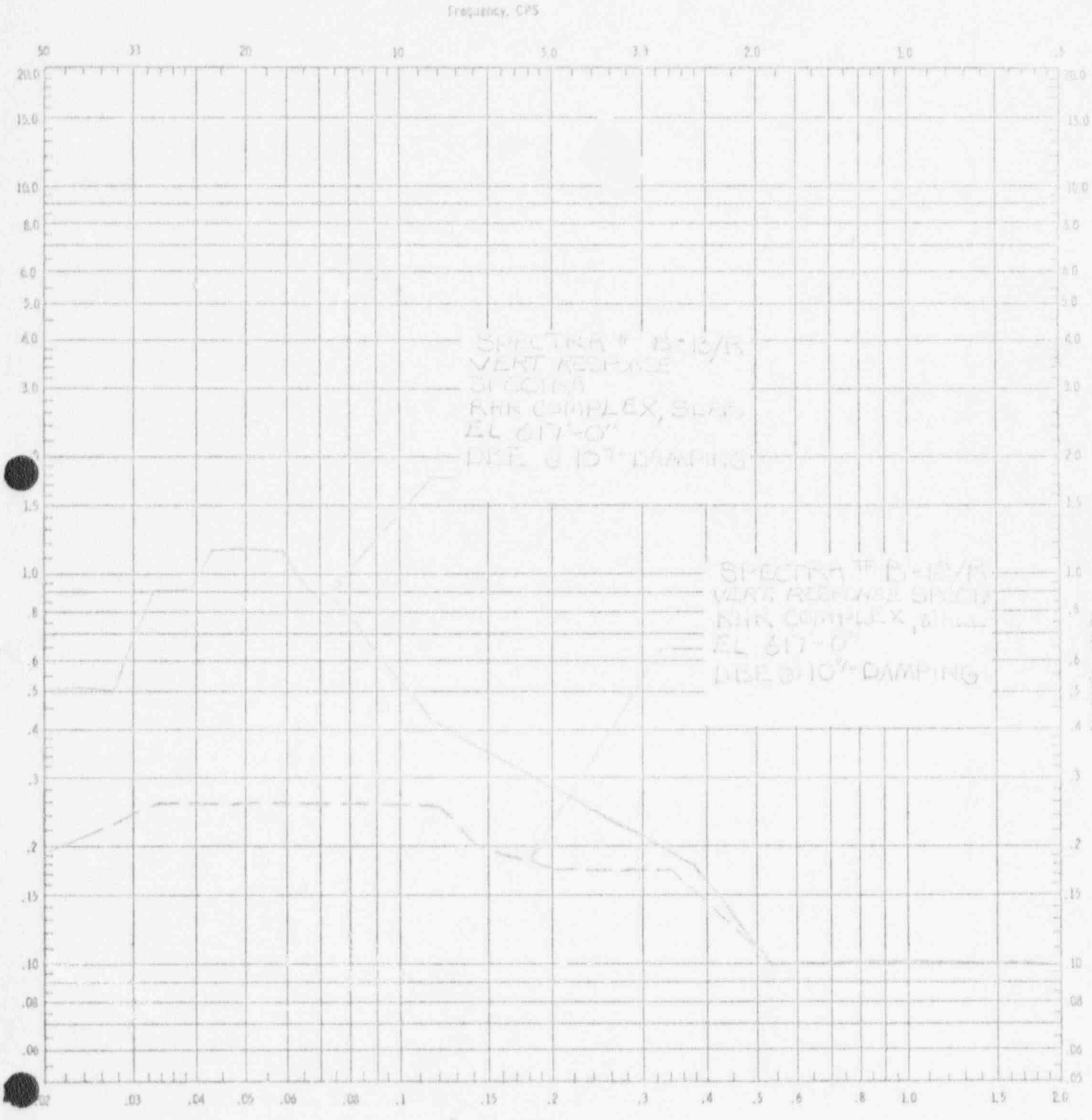
ELEVATION:

BPD

SARGENT & LUNDY
ENGINEERS
CHICAGO

SPECTRA FOR ELECTRICAL CABLE TRAY HANGERS IN RHR COMPLEX		Calc. No.
X Safety-Related	Non-Safety-Related	Rev. 0 Date
		Page 18 of

Client DETROIT EDISON	Prepared by	Date
Project ENRICO FERMI	Reviewed by	Date
Proj. No. 4139-38 Equip. No.	Approved by	Date

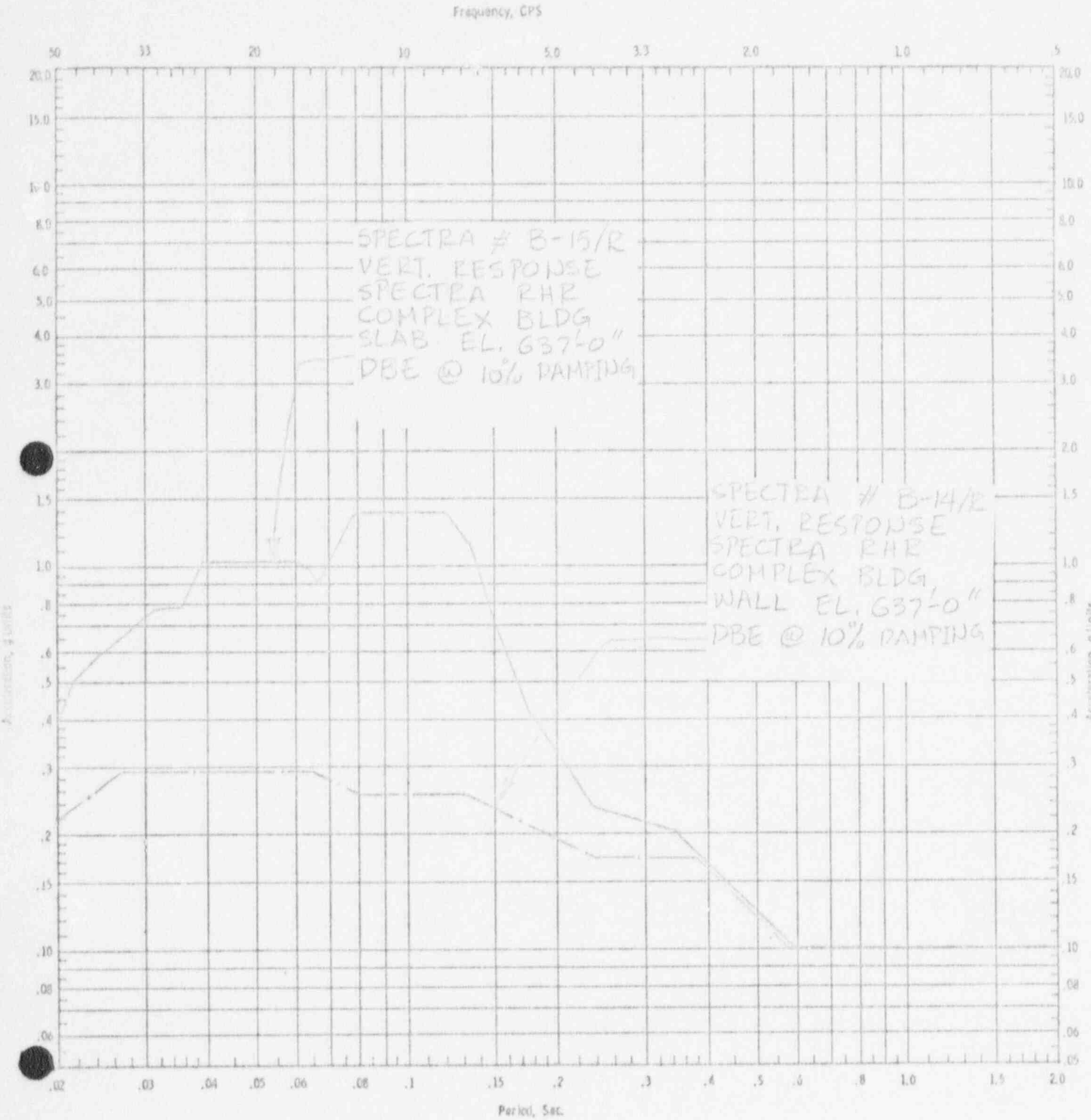


EXCITATION DBE VERTICAL @ 10% DAMPING LOCATION: RHR COMPLEX WALL & SLAB

SPECTRA NO B-12/R, B-13/R ELEVATION: 617'-0"

SARGENT & LUNDY ENGINEERS CHICAGO	SPECTRA FOR RHR COMPLEX VERT. RESPONSE SPECTRA @ EL. 637'-0"	Calc. No. EE0014
	Safety-Related / Non-Safety-Related	Rev. 2 Date 5/5/81
		Page 1.9 of

Client D.E.CO.	Prepared by Tr[illegible]	Date 5/5/81
Project FERMI-2	Reviewed by J. Nanda	Date 5/5/81
Proj. No. 6139-38 Equip. No.	Approved by	Date



EXCITATION DBE VERT. @ 10% DAMPING LOCATION: RHR COMPLEX WALL & SLAB

SPECTRA NO. B-14/R, B-15/R ELEVATION: 637'-0"