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SEISMIC AND STRUCTURAL ANALYSIS OF THE GENOA 3 STACK USING THE NRC SITE-SPECIFIC GROUND RESPONSE SPECTRA

PREPARED FOR

DAIRYLAND POWER COOPERATIVE

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1. SUMMARY

This report, prepared for Dairyland Power Cooperative (DPC), presents the results of the seismic/structural analysis of the GENOA 3 stack using the NRC site-specific ground response spectra[1] for the Safe Shutdown Earthquake Event (SSE).

Linear seismic analysis, using the site specific spectra and modal superposition, was used to determine the response of the GENOA 3 stack for the SSE Event. Soil structure interaction effects were included using the information provided by Dames & Moore.[2] The foundation springs reflect the updated information of the most recent boring program. The seismic response of the stack is compared to the load carrying capacities of the stack at corresponding elevations. From the results of the analysis, it has been concluded that under an SSE seismic event, the GENOA 3 stack will experience a failure 150 to 200 feet from its top. The surviving 300 to 350 feet of the stack will remain upright and attached to its foundation mat. Since the GENOA 3 stack is located approximately 400 feet from the LACBWR Reactor Containment Building and other safety related structures, the failed section of the stack should not impact on these structures.

2. BACKGROUND INFORMATION

In response to recent NRC questions Dairyland Power Cooperative (DPC) requested Nuclear Energy Services (NES) to analyze the GENOA 3 stack. This analysis was made using the most recent soils data from Dames & Moore, most recent design codes, current NRC Regulatory Guides and Standard Review Plans, and the recently established site specific ground spectra. Investigation of the following variables was made: soil properties, cracked, and uncracked section properties of the concrete. The results are presented within.

3. DESCRIPTION OF STACK

The GENOA 3 stack is a 500 foot high, tapered, reinforced concrete chimney with an independent steel liner. The outside diameter at the base is 38.198 feet with a wall thickness of 24 inches; the stack tapers to the top diameter of 17.42 feet with a wall thickness of 7 inches. The independent steel liner has an inside diameter of 15.25 feet for most of its height, bells out at its base and is supported on a concrete pedestal. Both the stack and its liner are founded on a 75 foot reinforced concrete octagonal mat. The foundation mat varies from 7 feet to 3'6" in depth and is directly supported by the soil (see Figure 3.1).[3]

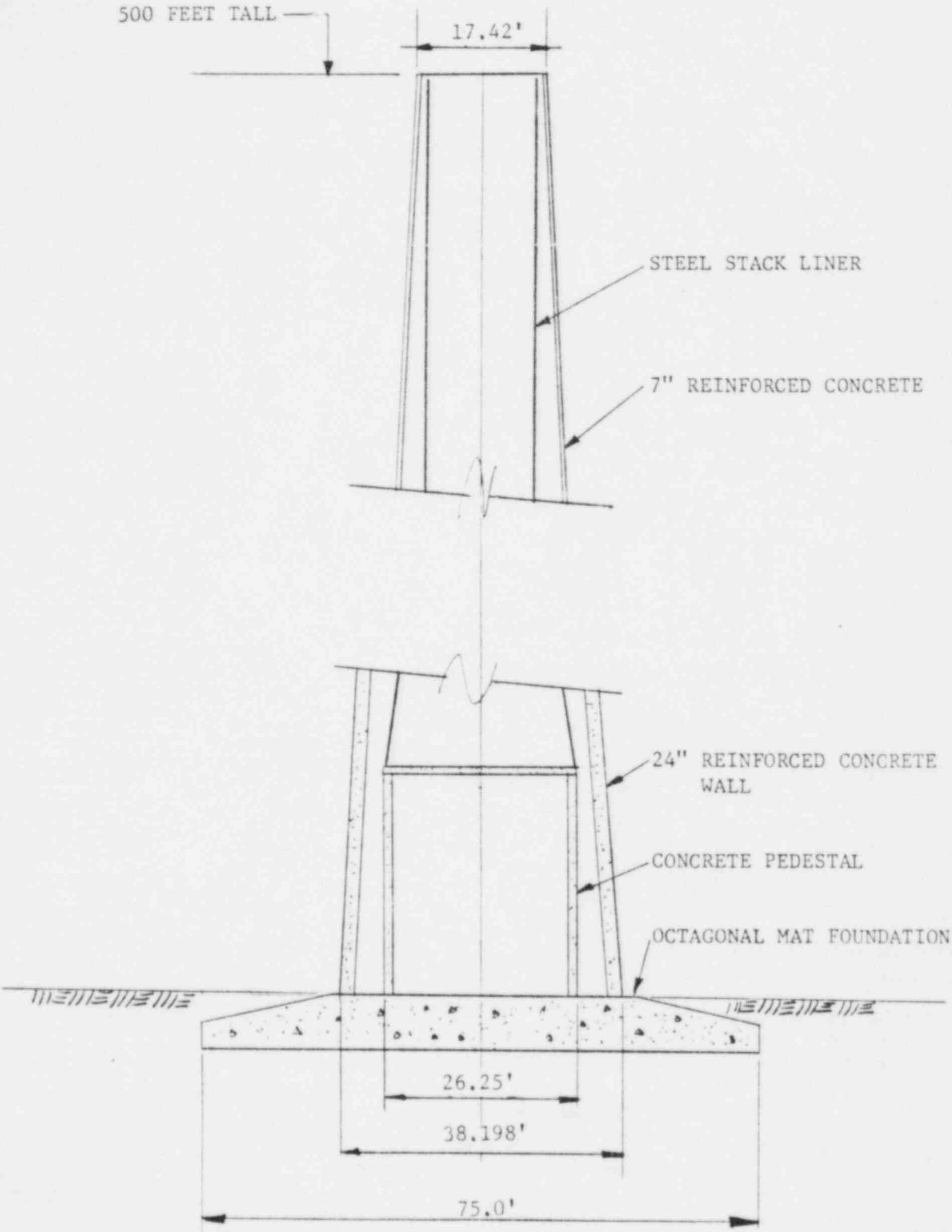

FIGURE 3.1
SCHEMATIC SKETCH OF GENOA 3 STACK

4. APPLICABLE CODES, STANDARDS AND SPECIFICATIONS

The following codes of practice and regulatory guides have been used in the analysis of the GENOA 3 Stack Analysis.

1. Specification For the Design and Construction of Reinforced Concrete Chimneys (ACI-307-79), American Concrete Institute, Detroit, Michigan, 1979.

2. Building Code Requirements for Reinforced Concrete (ACI 318-77), American Concrete Institute, Detroit, Michigan, 1977.

3. USNRC Regulatory Guide 1.61, "Damping Values for Seismic Design of Nuclear Power Plants", October, 1973.

4. USNRC Regulatory Guide 1.92, "Combination of Modes and Spatial Components in Seismic Response Analysis", Rev. 1, February, 1976.

5. USNRC Regulatory Guide 1.60, "Design Response Spectra for Seismic Design of Nuclear Power Plants", Rev. 1, December, 1973.

6. Uniform Building Code, 1979 Edition.

5. LOADS AND LOADING COMBINATIONS

The seismic lateral inertia loading on the coupled model of the stack and its foundations is in the form of the ground acceleration response spectra given in Reference 1. The free field ground response spectrum (Figure 5.1) for the Safe Shutdown Earthquake for 5 percent structural damping was modified to 7 percent and used in the seismic analysis. (See USNRC Reg. Guide 1.61).

In addition to the seismic inertia loading, the dead loads and their resulting moments have also been included in the analysis. The following load combination equation was used in evaluating the adequacy of the stacks to withstand a seismic event.

$$U = D + 1.0 E'$$

Where:

D = Dead loads and their resulting moments

E' = Loads and moments generated by the Safe Shutdown Earthquake

U = Section strength required to resist design loads and based on ultimate strength design methods described in ACI 318-77 Code.

The design loads from this load case were assumed to be resisted by the ultimate section capacities of the stack and its mat foundation.

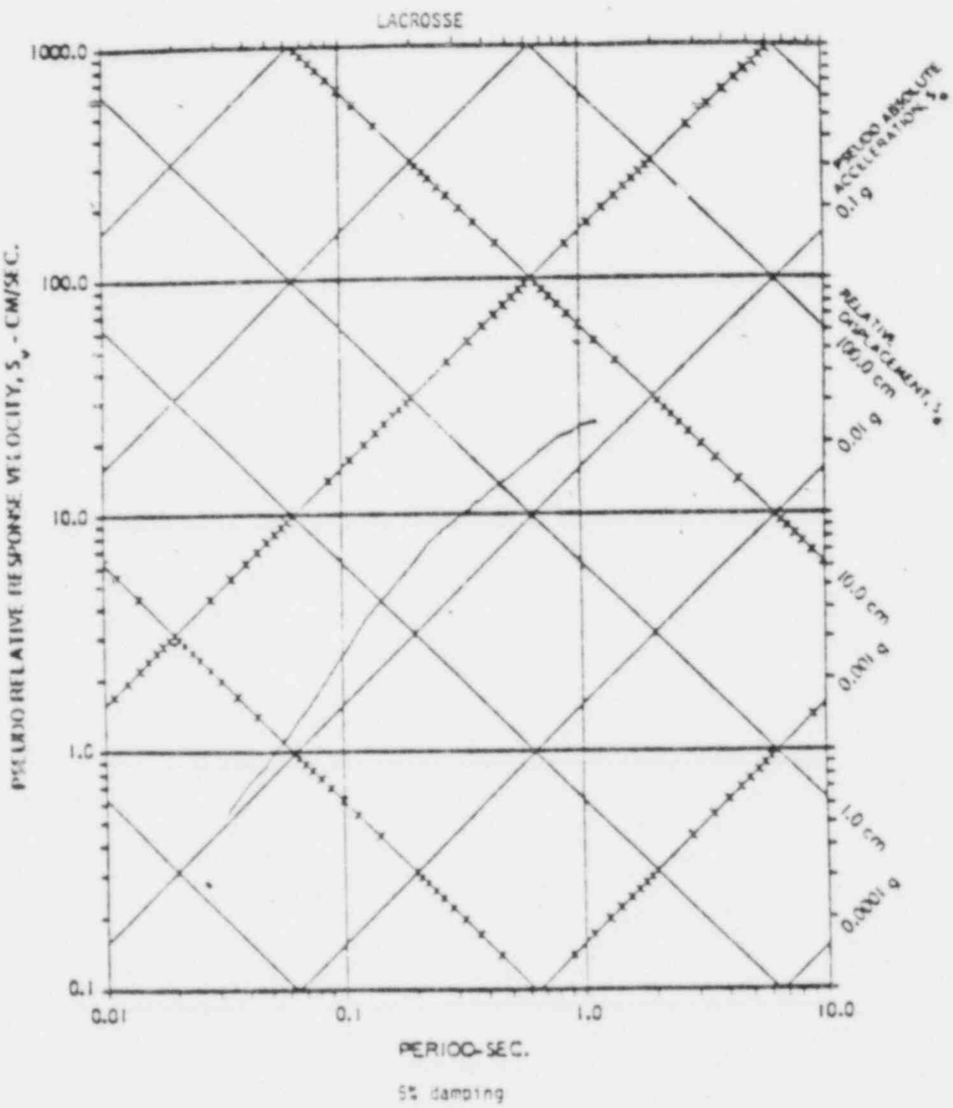

FIGURE 5.1
LACBWR SITE-SPECIFIC RESPONSE SPECTRA

6. ANALYTICAL PROCEDURES

6.1 SEISMIC ANALYSIS

6.1.1 Mathematical Model

In order to perform the seismic analysis, the stack is mathematically modeled as an assembly of elastic-structural elements interconnected at discrete nodal points. The three dimensional, multidegree of freedom model of the stack is attached to the ground by means of foundation springs, representing the deformations of the soil under the stack foundation. Lateral, as well as rocking springs, have been provided under the GENOA 3 stack mathematical model (Figure 6.1) to account for the shear and vertical deformation of the soil under the GENOA 3 stack foundation. To account for the variation in the soil properties and to evaluate the effect of changing the foundation spring constants on the seismic response of the stacks, the foundation springs were varied using information supplied by Dames and Moore. The frequencies found using this data is shown in Table 6.1. The effect of the variation can be seen in Figure 6.2.

The distributed mass of the stack is lumped at the system nodal points. Each mass represents the tributory weight of the stack walls above and below the nodal point. Masses are lumped so that the lumped mass, multidegree of freedom model represents the dynamic characteristics of the stack. In order to reduce the number of dynamic degrees of freedom, only translational degrees-of-freedom are considered at each mass point. (The masses associated with the rotational degrees-of-freedom are set to zero). The physical properties used in the model are given in Table 6.2.

6.1.2 Foundation Spring Stiffness

The stiffness of the lateral and rocking springs representing the shear and vertical deformation of the soil beneath the foundation mat are obtained using the equations shown in Figure 6.3. These equations are taken from Reference 4.

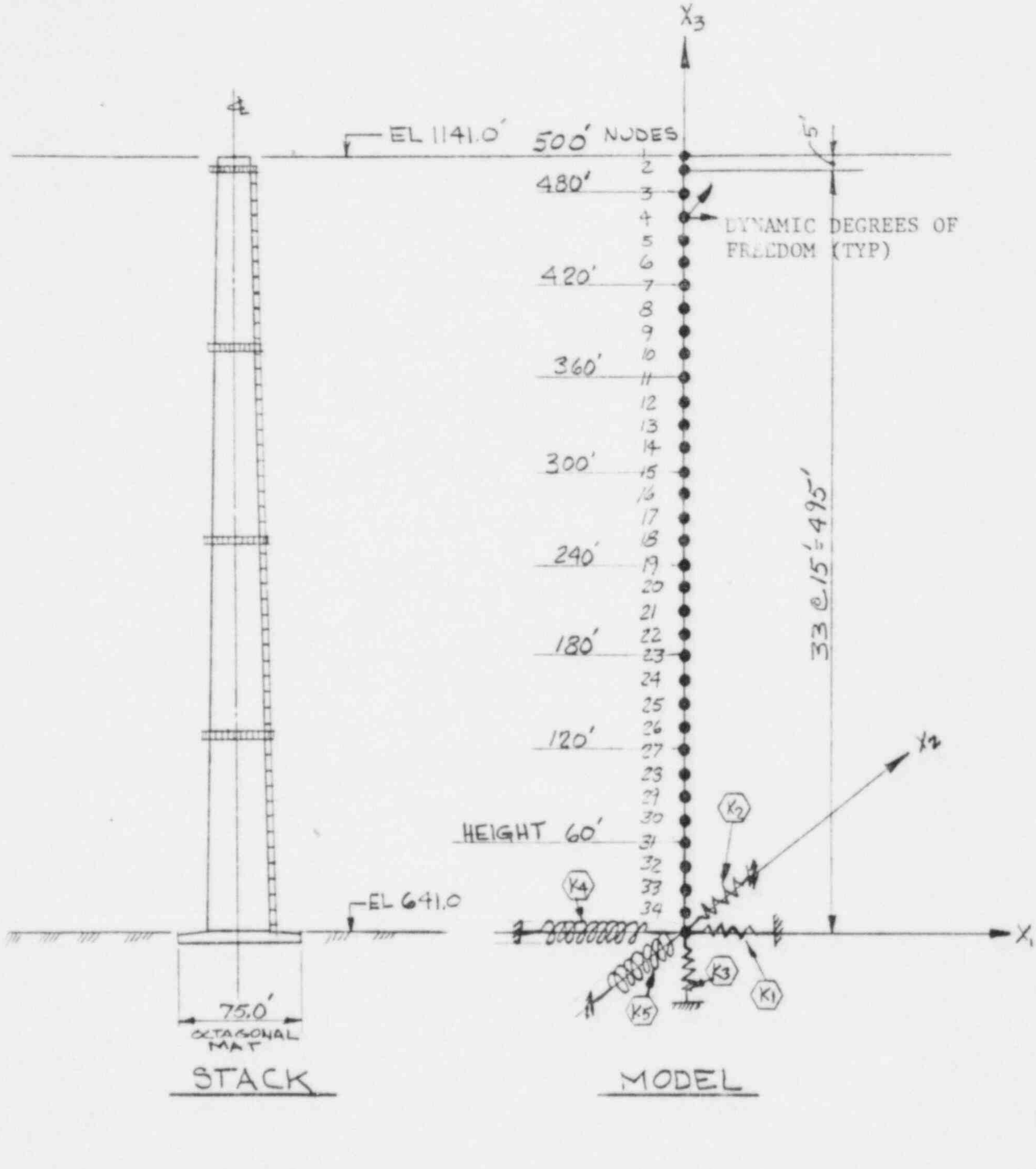

FIGURE 6.1
MATHEMATICAL MODEL OF GENOA 3 STACK

TABLE 6.1

NATURAL FREQUENCIES OF VIBRATION - GENOA 3 STACK

Mode No.	Modal Direction	Model 1 G = 1000 ksf Softer Foundation Spring	Model 2 G = 3000 ksf Stiffer Foundation Spring
1	X_2	0.363 CPS	0.388 CPS
2	X_1	0.363	0.388
3	X_2	1.349	1.487
4	X_1	1.349	1.487
5	X_2	2.976	3.235
6	X_1	2.976	3.235
7	X_3	3.497	4.710
8	X_2	4.67	5.561
9	X_1	4.67	5.561
10	X_2	6.124	8.138
11	X_1	6.124	8.138

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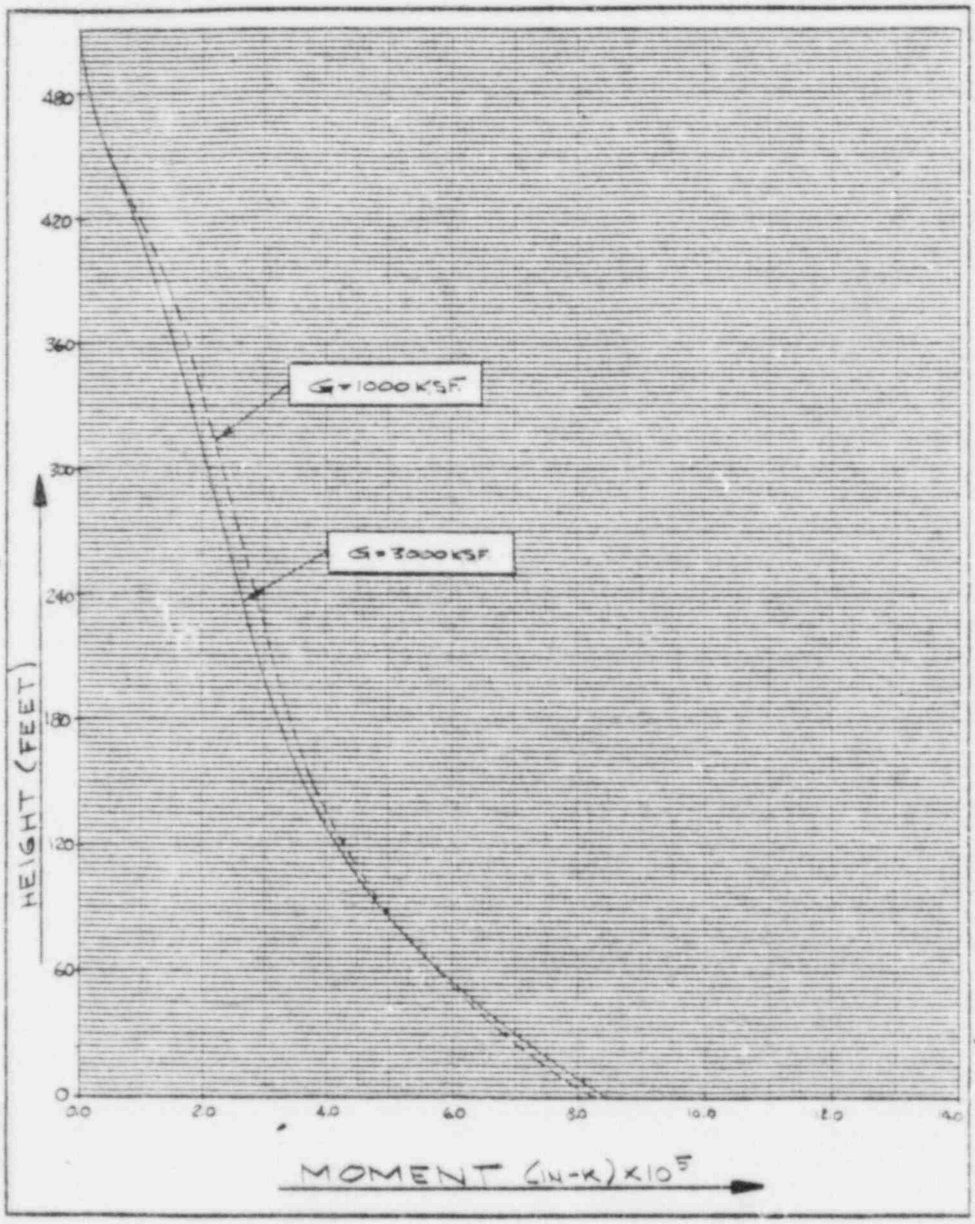


FIGURE 6.2
EFFECT OF VARIATION OF SOIL PROPERTIES
ON STRUCTURAL RESPONSE

TABLE 6.2

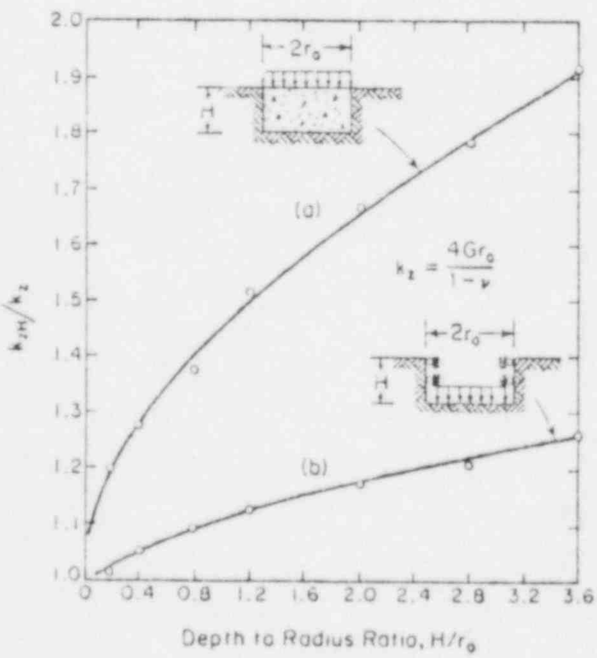
GENOA 3 STACK PROPERTIES

Node No.	Outside Diameter (in)	Concrete Wall Thickness (in)	Area Concrete (in^2)	Area Steel (in^2)	Steel Ratio	Dead Weight (kips)
1		7.0		11.60		14.825
2	211.0	7.0	4486.19	11.80	0.00259	60.810
3	217.0	7.0	4618.14	12.00	0.00260	132.965
4	223.0	7.0	4650.09	12.40	0.00261	207.180
5	229.0	7.0	4882.03	12.60	0.00258	283.457
6	235.0	7.0	5013.98	12.60	0.00251	361.795
7	241.0	7.0	5145.93	12.80	0.00249	442.195
8	247.0	7.0	5277.88	13.20	0.00250	524.657
9	253.0	7.0	5409.87	13.40	0.00248	609.180
10	259.0	7.0	5541.77	13.80	0.00249	695.765
11	265.0	7.0	5673.72	14.00	0.00247	786.410
12	271.0	7.0	5805.67	14.40	0.00248	877.119
13	277.0	7.0	5937.61	14.80	0.00249	970.017
14	284.5	7.0	6102.54	19.60	0.00321	1065.367
15	292.0	7.0	6267.48	24.00	0.00383	1163.287
16	299.5	7.0	6432.41	29.14	0.00453	1263.787
17	307.0	7.0	6597.34	34.72	0.00526	1366.867
18	314.5	7.0	6762.28	44.00	0.00651	1474.517
19	322.0	7.5	7410.23	54.56	0.00736	1590.377
20	329.5	8.0	8080.18	64.80	0.00800	1716.417
21	337.0	8.0	8268.67	74.26	0.00898	1847.797
22	344.5	8.5	8972.39	85.32	0.00950	1990.057
23	352.0	9.25	9960.22	96.00	0.00964	2145.717
24	359.375	10.0	10975.93	104.00	0.00948	2315.397
25	368.375	10.5	11805.12	108.00	0.00920	2499.897
26	377.375	11.0	12661.00	112.00	0.00884	2697.767
27	386.375	11.5	13543.60	116.00	0.00856	2911.427
28	395.375	12.0	14452.90	118.00	0.00816	3144.187
29	404.375	14.0	17169.58	124.00	0.00720	3412.617
30	413.375	16.0	19992.47	130.00	0.00650	3738.947
31	422.375	21.0	26480.09	134.00	0.00506	4179.777
32	431.375	24.0	30715.35	134.00	0.00436	4698.777
33	440.375	24.0	31393.94	134.00	0.00427	5206.177
34	449.375	24.0	32072.52	133.00	0.00415	5707.277
35	458.375	24.0	32751.10	133.00	0.00406	11580.127

Spring Constants for Rigid Circular Footing Resting on Elastic Half-Space

Motion	Spring Constant	Reference
Vertical	$k_z = \frac{4Gr_o}{1-\nu}$	Timoshenko and Goodier (1951)
Horizontal	$k_x = \frac{32(1-\nu)Gr_o}{7-8\nu}$	Bycroft (1956)
Rocking	$k_\psi = \frac{8Gr_o^3}{3(1-\nu)}$	Borowicka (1943)
Torsion	$k_\Theta = \frac{16}{3}Gr_o^3$	Reissner and Sagoci (1944)

$\left(Note: G = \frac{E}{2(1+\nu)}\right)$



Effect of depth of embedment on the spring constant for vertically loaded circular footings (*from Kaldjian, 1969*).

FIGURE 6.3
SOIL SPRING CONSTANTS

6.1.3 Eigenvalue Analysis

The eigenvalues (natural frequencies) and the eigenvectors (mode shapes) for each of the natural modes of vibration are calculated by solving the following frequency equation:

$$(K - \omega_n^2 M) \ \{\phi_n\} = \{0\} \tag{1}$$

Where:

ω_n = Natural angular frequency for the n^{th} mode

M = System mass matrix

ϕ_n = Mode shape vector for the n^{th} mode

0 = Null vector

The eigenvalue/eigenvector extraction is performed using the Householder QR Modal Extraction Methods.

6.1.4 Dynamic (Seismic) Load Analysis

Considering only translational degrees of freedom and assuming viscous (velocity proportional) form of damping, the equation of motion in matrix form can be expressed as follows:

$$M (\ddot{U}_t + \ddot{U}_{gt}) + C\dot{U}_t + KU_t = 0 \tag{2}$$

Where:

$\ddot{U}_t$ = Relative acceleration time history vector

$\ddot{U}_{gt}$ = Ground acceleration time history vector

C = Damping matrix

$\dot{U}_t$ = Velocity time history vector

U_t = Relative displacement time history vector

Rearranging equation (2):

$$M\ddot{U}_t + C\dot{U}_t + KU_t = M\ddot{U}_{gt} = P_{eff} \quad (3)$$

To uncouple equation (3), assume:

$$U = \phi Y_t$$

Where:

ϕ = Characteristic free vibration mode shapes matrix

Y_t = Generalized coordinate displacement time history vector

Pre- and post- multiplying equation (3) by the transpose of ϕ and ϕ respectively and using orthogonality conditions, the following uncoupled equations of motion are obtained:

$$\ddot{Y}_{nt} + 2\omega_n\lambda_n \dot{Y}_{nt} + \omega_n^2 Y_{nt} = M_n^{*-1} R_n \ddot{U}_{gt} \quad (4)$$

Where:

Y_{nt} = Generalized displacement coordinate time history for n^{th} mode.

λ_n = Damping ratio for the n^{th} mode expressed as percent of critical damping.

$M_n^* =$ Generalized mass for the n^{th} mode

$$= \phi_n^T M \phi_n = \sum M_i \phi_{in}^2$$

The mode shape ϕ_n is normalized such that $M_n^* = 1$

R_n = Participation factor for the n^{th} mode.

$$= \phi_n^T M I = \sum M_i \phi_{in}$$

I = Column vector whose elements are generally unity

The solution for the differential equation (4) is given by the Duhamel Integral:

$$Y_{nt} = \frac{R_n}{M_n^* \omega_n} \int_t^\tau \ddot{U}_{gt} e^{-\lambda_n \omega_n (t-\tau)} \sin \omega_n (t-\tau)\, d\tau$$

Using the response spectrum method of analysis, the maximum values of the generalized response for each mode is given by:

$$\ddot{Y}_{n\,max} = \frac{R_n S_{an}}{M_n^*} \qquad (5)$$

Where:

$\ddot{Y}_{n\,max}$ = Maximum generalized coordinate acceleration response for the n^{th} mode.

S_{an} = Spectral acceleration value for the n^{th} mode (from the applicable response spectrum curve)

From the maximum generalized coordinate response the maximum acceleration ($\ddot{U}_{n\,max}$) and maximum inertia forces ($F_{n\,max}$) at each mass point are given by:

$$\ddot{U}_{n\,max} = \ddot{Y}_{n\,max}\,\phi_{in}$$

$$F_{n\,max} = M_n\,\ddot{U}_{n\,max}$$

The inertia forces ($F_{n\,max}$) for each of the systems' natural modes are applied as external static forces, and system response (displacements, member internal forces and stresses) are calculated. Total system response is than obtained by combining the individual modal response values by the square root of the sum of the squares method; lower modes having large contribution to the response (all modes having natural frequency under 30 cycles per second) are considered and higher modes with negligible participation are neglected.

6.2 STRUCTURAL ANALYSIS

The Genoa 3 Stack was analyzed using the ultimate strength design method presented by Cannon[5]. The graphical solutions derived by Cannon are shown in Figure 6.4. The basic assumptions used in this method are given in Appendix A of this report. Tests performed at University of Michigan verify that Cannon's method predicts failure well.[7&8] The tests show that actual failure occurs at approximately 10 to 15% over the predicted. This is assumed to be due to the effect of strain hardening, which is not accounted for in the analysis method.

The octagonal mat foundation was evaluated using methods presented in Reference 6 and in accordance with ACI 318-77 Ultimate Strength Design Methods. The method used appears to be quite conservative. Appendix B contains the foundation analysis calculations.

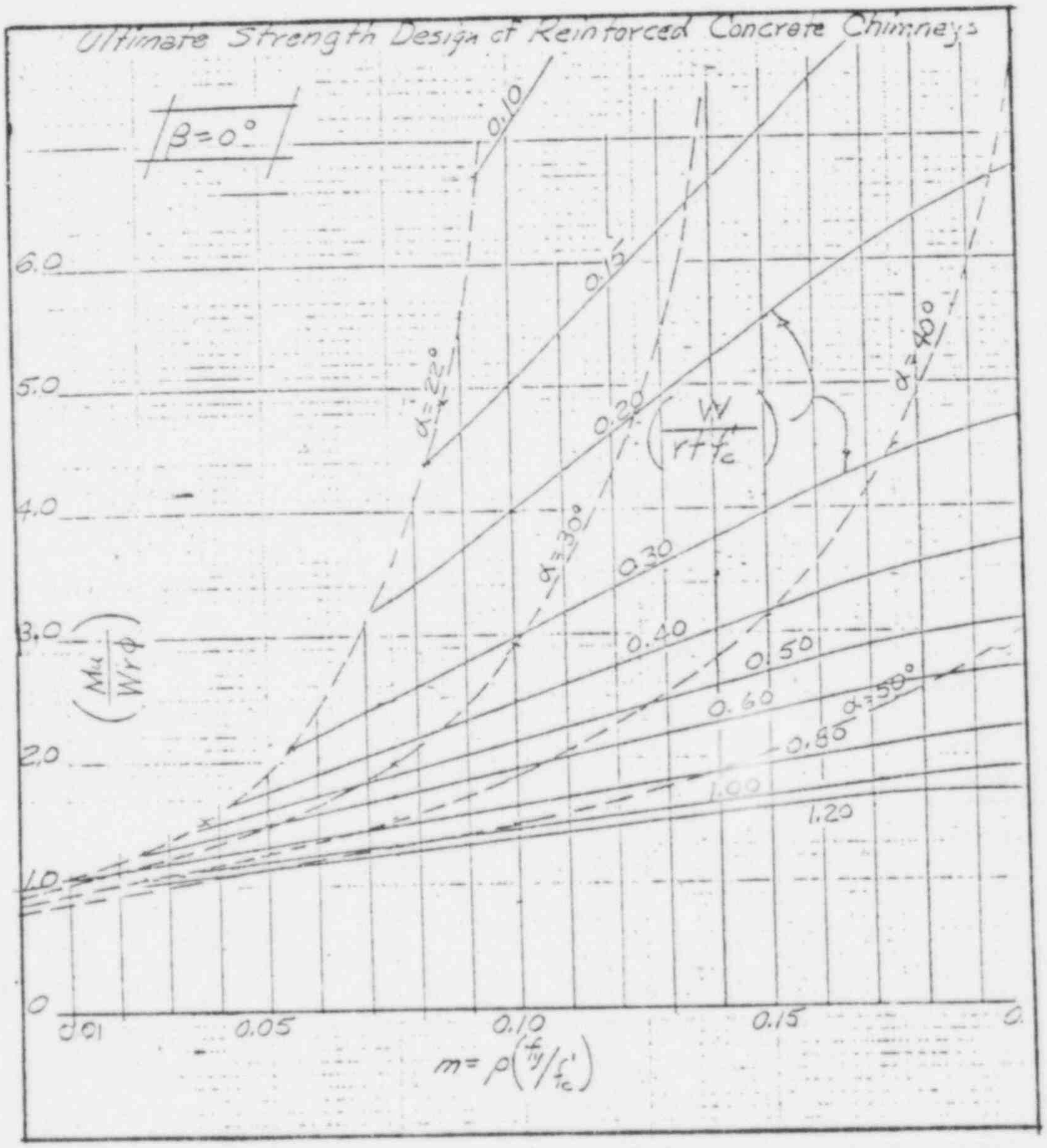


FIGURE 6.4
CANNON'S SOLUTIONS

7. ACCEPTANCE CRITERIA

The ultimate moment and shear load-carrying capacities of the stack cross-sections have been calculated using the acceptable ultimate stress values as given in the ACI 318-77 Design Code and References.

The specific acceptable stress values used in this analysis are given below:

Maximum concrete compressive stress	$= 0.85 f'_c$	(ACI 318-77)
Maximum concrete shear stress	$= 4\phi\sqrt{f'_c}$	(ACI 318-77)
Maximum stress in reinforcing steel	$= f_y$	(Reference 5)

Where:

f'_c = compressive strength of concrete at 28 days
= 4,000 psi for Genoa 3 Stack
3,000 psi for Genoa 3 stack foundation mat

ϕ = 0.75 for Genoa 3 stack (Reference 5)

f_y = Yield stress value for reinforcing steel
= 40.0 ksi for Genoa 3 stack
60.0 ksi for Genoa 3 stack foundation mat

8. RESULTS OF ANALYSIS AND CONCLUSIONS

The results of the seismic analysis of the Genoa 3 stack performed with Stardyne computer code are contained in Reference 9.

Appendix A contains the assumptions used in the analysis, and the detail structural evaluation of Genoa 3 stack. Appendix B contains the detail calculations for the structural evaluation of concrete mat foundation.

The natural frequencies of vibration of the Genoa 3 stack are given in Table 6.1. From Table 6.1 it can be seen that the stack is a low frequency system (fundamental frequency of 0.363 and 0.388 Hz) and the variation in the fundamental frequencies is small (0.363 Hz to 0.388 Hz) as compared to the variation in the foundation soil constants (G = 1000 ksf to 3000 ksf). The results of the seismic and structural analysis are summarized in Table 8.1 and 8.2 and shown in Figure 8.1. Table 8.1 summarizes the moments due to the SSE seismic event and compares them to the allowable ultimate moment capacities of the stack. From Table 8.1 it can be seen that the moments due to SSE event at Nodes 15 through 4 (height: 300 ft. to 465 ft.) exceed the allowable moment capacities (ultimate moment capacities).

The maximum ratio of SSE seismic moment to the ultimate moment capacity is 1.6. This 60% overstress during the SSE event is considerably greater than the 10 to 15% variation between the test results 7&8, and the calculated ultimate moment capacity. Figure 6.2 shows the continuous variation of the seismic moment through the height of the stack and the insensitivity of the seismic moment response to the foundation soil properties.

Table 8.2 compares the ultimate shear capacity of the stack to the SSE shear values. It can be seen that the ultimate shear capacity of the stack is considerably greater than the SSE seismic shear.

The octagonal mat foundation has been evaluated for the foundation pressure distribution resulting from the seismic moment and dead weight loadings. Results of the analysis shows that the foundation will be slightly overstressed. However, the method of analysis are quite conservative. A detailed finite element model is now being developed for further evaluation of the mat foundation.

It can be concluded from the above that under an SSE seismic event, the 500-foot GENOA 3 stack will experience a failure 150 to 200 feet from its top. The surviving 300 to 350 feet of the stack will remain upright and attached to its foundation mat. Since the GENOA 3 stack is located approximately 400 feet from the LACBWR Reactor Containment Building and other safety related structures, the failed section of the stack should not impact on these structures.

The seismic and dead weight loadings and the soil bearing pressure distributions have been supplied to Dames & Moore for their evaluation. Dames & Moore will confirm the soil's capability to withstand these loads.

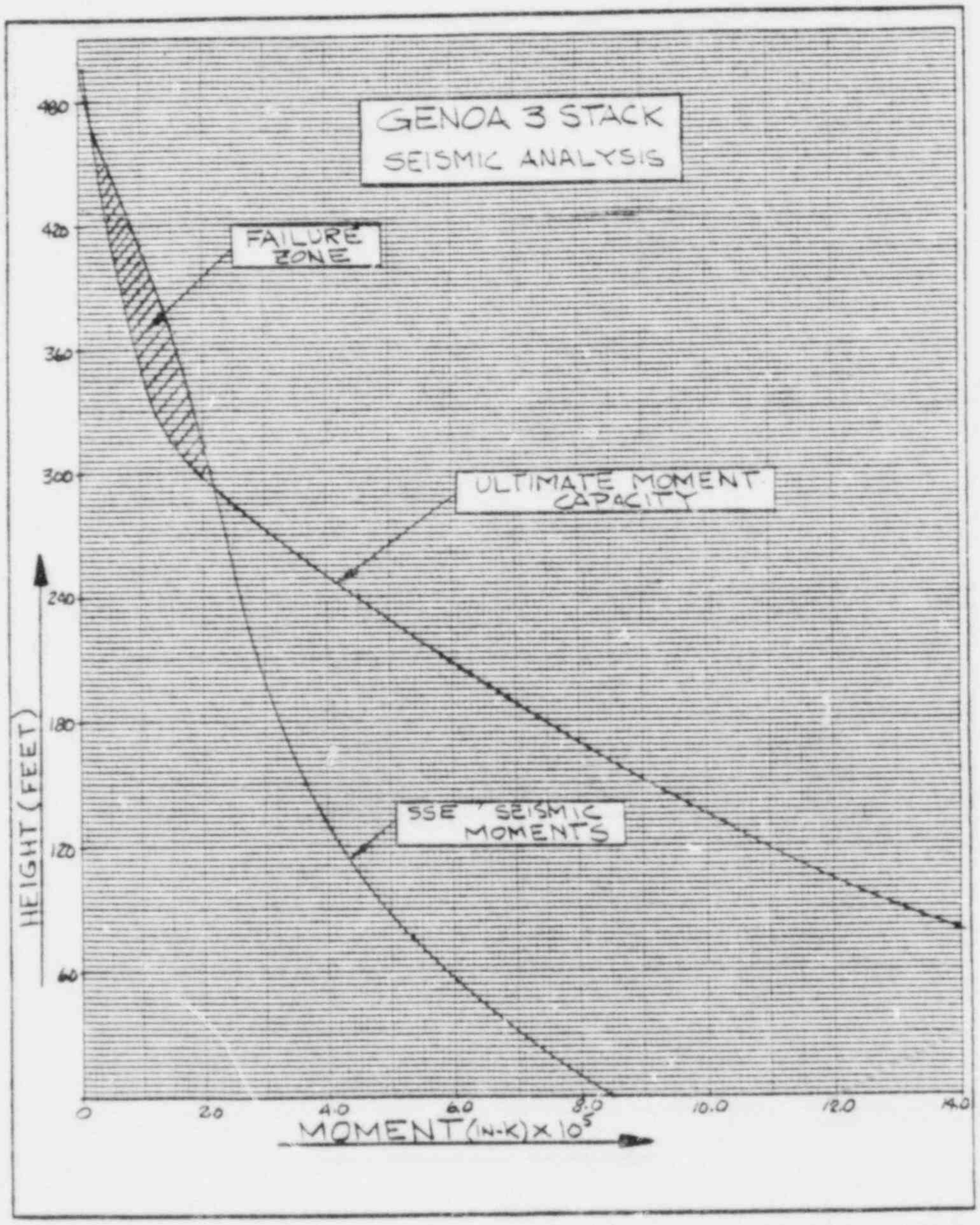


FIGURE 8.1
SUMMARY OF SEISMIC/STRUCTURAL EVALUATION (MOMENT)

TABLE 8.1

SUMMARY OF SEISMIC/STRUCTURAL EVALUATION (MOMENT)

Node	Distance from Top (ft)	Ultimate Moment Capacity (K-in) x 10^5	Moment due to SSE Event (K-in) x 10^5
2	5	0.061	0.0069
3	20	0.136	0.095
4	35	0.220	< 0.252 NG
5	50	0.307	< 0.447 NG
6	65	0.387	< 0.656 NG
7	80	0.485	< 0.862 NG
8	95	0.590	< 1.055 NG
9	110	0.703	< 1.234 NG
10	125	0.822	< 1.401 NG
11	140	0.9511	< 1.559 NG
12	155	1.0854	< 1.713 NG
13	170	1.2270	< 1.861 NG
14	185	1.5500	< 2.001 NG
15	200	1.9270	< 2.130 NG
16	215	2.4260	2.250
17	230	3.1520	3.368
18	245	3.7405	2.482
19	260	4.5020	2.602
20	275	5.1730	2.731
21	290	5.810	2.873
22	305	6.640	3.032
23	320	7.450	3.210
24	335	8.340	3.410
25	350	9.059	3.635
26	365	9.640	3.822
27	380	10.440	4.189
28	395	11.070	4.530
29	410	11.490	4.918
30	425	13.094	5.358
31	440	12.270	5.853
32	455	12.560	6.411
33	470	13.660	7.039
34	485	15.020	7.732
35	500	17.742	8.491

* Results from Computer Program S532 E82

TABLE 8.2

SUMMARY OF SEISMIC/STRUCTURAL EVALUATION(SHEAR)

Node	Distance from Top (ft)	Ultimate Capacity (kips)	SSE Shear * (kips)
2	5	964.69	11.65
3	20	993.06	49.92
4	35	999.93	86.97
5	50	1049.81	108.05
6	65	1078.18	116.71
7	80	1106.55	118.95
8	95	1134.93	120.94
9	110	1163.31	125.99
10	125	1191.67	133.01
11	140	1220.05	138.94
12	155	1248.42	141.28
13	170	1276.79	140.39
14	185	1312.26	139.00
15	200	1347.73	140.39
16	215	1383.19	147.74
17	230	1418.66	158.99
18	245	1454.13	171.66
19	260	1593.46	183.38
20	275	1737.52	193.38
21	290	1778.05	202.54
22	305	1929.38	212.23
23	320	2141.79	224.90
24	335	2360.21	242.03
25	350	2538.51	263.30
26	365	2722.56	287.09
27	380	2912.35	311.24
28	395	3107.88	334.62
29	410	3692.06	357.29
30	425	4299.08	382.19
31	440	5694.14	414.03
32	455	6604.87	463.24
33	470	6750.79	508.74
34	485	6896.71	543.41
35	500	7042.63	583.08

* Results from Computer Program S532 E82

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Rumman, W.S.: Basic Structural Design of Concrete Chimneys, Journal of the Power Division, pp 309 - 317 (June, 1970)

Rumman, W.S. and Mauch, L.C.: Earthquake Forces Acting on Tall Concrete Chimneys, International Association for Bridge and Structural Engineering (September, 1968)

Rumman, W.S. and Ru-Tsung Sun: Ultimate Strength of Reinforced Concrete Chimneys, Journal of ACI Proceedings, pp 179 - 184 (April, 1977)

Rumman, W.S.: Vibrations of Steel-Lined Concrete Chimneys, Journal of the Structural Division, ASCE, pp 35 - 63 (October, 1963)

Rumman, W.S.: Earthquake Forces in Reinforced Concrete Chimneys, Journal of the Structural Division, ASCE, pp 55 - 70, (December, 1964)

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APPENDIX A
STACK ANALYSIS

Assumptions:

1. The assumptions used for ultimate strength design and compatibility of strains are the same as those given in ACI Building Code (318-77).

2. Maximum steel stress at ultimate capacity is assumed as "fy".

3. The ultimate moment occurs when the strain in the concrete reaches 0.003 inch per inch.

4. A uniform compressive stress block is assumed with ($a = 0.85\ K_1$).

5. Compressive reinforcement is not considered.

6. Reinforcement is uniform throughout the section.

REF.

APPENDIX A

GENOA 3 STACK

	REF.

GENOA 3 STACK

USING REFERENCE "VIBRATIONS OF SOILS AND FOUNDATIONS"
BY F.E. RICHART, JR
J.R. HALL, JR
R.D. WOODS
PRENTICE-HALL, INC ENGLEWOOD CLIFFS, N.J.
Pg 350

VERTICAL SPRING CONSTANT FOR CIRCULAR FOOTINGS

$$K_z = \frac{4\,G R_o}{(1-\nu)}$$

DAMES & MOORE: $\nu = 0.3$, G = 1000 KSF, G = 3000 KSF

R_o = EFFECTIVE RADIUS

$$K_z = \frac{4 \times 1000 \times 38.5}{(1-.3)}$$

$$K_z = 220000\ \text{K/FT}\ \frac{1\text{FT}}{12\text{IN}}$$

AREA OF OCTAGON

$A = 0.828\, d^2$

$A = 0.828(75)^2 = 4657.5\ \text{FT}^2$

AREA OF CIRCLE $= \frac{\pi}{4} D^2$

$D = \sqrt{\frac{4A}{\pi}} = 77.0$

$R_o = 38.5$ FT

G=1000 → K_z = 18,333 K/IN

G=3000 → K_z = 54,999 K/IN

G=1600 → 29333. K/IN

TAKING THE DEPTH OF EMBEDMENT INTO ACCOUNT.

$$\frac{H}{R_o} = \frac{7}{38.5} = 0.182$$

FROM GRAPH CURVE A

$K_{zH}/K_z = 1.2$ NOT LARGE ENOUGH SINCE THE FACTOR IS 3 FOR A RANGE OF SHEAR MODULUS VALUES. NEGLECT

$\frac{K}{FT^2}\ \frac{FT^2}{144}$

	REF.
GENOA 3 STACK	

HORIZONTAL SPRING CONSTANT FOR CIRCULAR FOOTINGS

$$K_x = \frac{32(1-\nu)GR_0}{7-8\nu}$$

$\nu = 0.3$
$R_0 = 38.5$ FT
$G = 1000$ KSF
$G = 3000$ KSF

$$K_x = \frac{32(1-0.3)\ 1000\,K/FT^2\ 38.5\,FT}{7-8(0.3)}$$

$$K_x = 1.875 \times 10^5\ K/FT\ \frac{1\,FT}{12\,IN}$$

G=1000 $K = 1.5623 \times 10^4$ K/IN   G=1600 KSF $K = 2.50 \times 10^4$ K/IN

G=3000 $K = 4.6870 \times 10^4$ K/IN

REF.

GENOA 3 STACK

TORSIONAL SPRING FOR CIRCULAR FOOTING

$K_\theta = \frac{16}{3} G R_o^3$

$R_o = 38.5$ FT
$G = 1000$ KSF
$G = 3000$ KSF > $G = 1600$

$K_\theta = \frac{16}{3}\ 1000\ K/FT^2\ 38.5^3\ FT^3$

$K_\theta = 3.044 \times 10^8 \frac{K\text{-}FT}{RAD} \frac{12\ IN}{1\ FT}$

G=1000 KSF $K_\theta = 3.6528 \times 10^9 \frac{K\text{-}IN}{RAD}$ G=1600 $K_\theta = 5.8445 \times 10^9 \frac{K\text{-}IN}{RAD}$

G=3000 KSF $K_\theta = 1.0958 \times 10^{10} \frac{K\text{-}IN}{RAD}$

ROCKING SPRING FOR CIRCULAR FOOTING

$K_\psi = \frac{8 G R_o^3}{3(1-\nu)}$

$\nu = 0.3$
$R_o = 38.5$ FT
$G = 1000$ KSF
$G = 3000$ KSF

$K_\psi = \frac{8 \times 1000\ KSF \times 38.5^3\ FT^3}{3(1-0.3)}$

$K_\psi = 2.174 \times 10^8 \frac{K\text{-}FT}{RAD} \frac{12\ IN}{FT}$

G=1000 $K_\psi = 2.6088 \times 10^9 \frac{K\text{-}IN}{RAD}$ G=1600 KSF $K_\psi = 4.1741 \times 10^9 \frac{K\text{-}IN}{RAD}$

G=3000 $K_\psi = 7.8263 \times 10^9 \frac{K\text{-}IN}{RAD}$

GENOA 3 STACK — ULTIMATE STRENGTH METHOD | REF.

ULTIMATE STRENGTH DESIGN $F_y = 40$ksi $\phi = .75$

PROCEDURE:

1. CALCULATE $\frac{W}{r t f'_c}$

2. READ $\frac{M_u}{W r \phi}$ FROM GRAPH B = 0

3. FIND $\rho (f_y / f_c)$

REFERENCE: CANNON, R.W AND BOOP, W.C: ULTIMATE STRENGTH DESIGN CHARTS FOR DESIGN OF REINFORCED CONCRETE CHIMNEYS, CIVIL ENGINEERING DESIGN RESEARCH REPORT, p 14, TENNESSEE VALLEY AUTHORITY, KNOXVILLE, KY (1971)

AT NODE 5

$W = 283.457^k$ ✓

$r = 111.0$ IN

$t = 7.0$

$f'_c = 4.0$ ksi

$$\frac{W}{r t f_c} = 0.0912$$

$$m = 0.00258 \left(\frac{40}{4}\right) = 0.0258$$

$$\frac{M_u}{W R \phi} = 1.3$$

$M_u = 1.3 \times W r \phi$ $\phi = .75$

$$M_u = 1.3 \times 0.75 \times 111.0 \times 283.457 = 30677.1^{\text{IN-K}}$$

0.3×10^{5} IN-K

GENOA 3 STACK — ULTIMATE STRENGTH METHOD | REF.

AT NODE 2

$f_y = 40$ ksi $\phi = 0.75$

$W = 60.81^k$

$r = 102$ in

$t = 7.0$ in

$f'_c = 4$ ksi

$$\frac{W}{r t f'_c} = 0.021$$

$m = 0.026$

$$\frac{M_u}{W r \phi} = 1.30$$

$M_u = 6047.6^{k\text{-}in}$

$= 0.061 \times 10^5$ k-in

AT NODE 3

$W = 132.97^k$

$r = 105.0$ in

$t = 7.0$ in

$f'_c = 4.0$ ksi

$m = 0.026$

$$\frac{M_u}{W r \phi} = 1.30$$

$M_u = 13612.8^{k\text{-}in}$

$= 0.136 \times 10^5$ k-in

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GENOA 3 STACK ULTIMATE STRENGTH METHOD	REF.

AT NODE 4

$f_y = 40ksi$ $\phi = 0.75$

$W = 207.18^k$

$r = 108.0 in$

$t = 7.0 in$

$f'_c = 4.0 ksi$

$\frac{W}{r t f'_c} = 0.069$

$m = 0.026$

$\frac{M_u}{W R \phi} = 1.3$

$M_u = 21816.0^{k-in}$

$= 0.22 \times 10^5 \text{ k-in}$

GENOA 3 STACK ULTIMATE STRENGTH METHOD

REF.

$F_y = 40$ ksi $\phi = 0.75$

At Node 6

$W = 361.8^k$

$r = 114.0$ in

$t = 7.0$ in

$f'_c = 4.0$ ksi

$\frac{W}{rtf'_c} = 0.113$

$m = 0.025$

$\frac{M_u}{Wr\phi} = 1.25$

$M_u = 38667.4^{k\text{-}in}$

0.39×10^5 k-in

At Node 7

$W = 442.195^k$

$r = 117.0$ in

$t = 7.0$ in

$f'_c = 4.0$ ksi

$\frac{W}{rtf'_c} = 0.135$

$m = 0.025$

$\frac{M_u}{Wr\phi} = 1.25$

$M_u = 0.485 \times 10^5$ k-in

Node 8

$W = 524.66^k$

$r = 120.0$ in

$t = 7.0$ in

$f'_c = 4.0$ ksi

$\frac{W}{rtf'_c} = 0.156$

$m = 0.025$

$\frac{M_u}{Wr\phi} = 1.25$

$M_u = 0.5902 \times 10^5$ k-in

Node 9

$W = 609.18^k$

$r = 123$ in

$t = 7.0$ in

$f'_c = 4.0$ ksi

$\frac{W}{rtf'_c} = 0.177$

$m = 0.025$

$\frac{M_u}{Wr\phi} = 1.25$

$M_u = 0.7025 \times 10^5$

GENOA 3 STACK ULTIMATE STRENGTH METHOD | REF.

$f_y = 40ksi$ $\phi = 0.75$

AT NODE 10

$W = 695.765$

$r = 126$

$t = 7$

$F'_c = 4\ ksi$

$\frac{W}{r t F'_c} = 0.154$

$m = \rho\left(\frac{F_y}{F'_c}\right) = 0.025$

$\frac{M_u}{W R \phi} = 1.25$

$M_u = 82187.2$ IN-K

AT NODE 12

$W = 877.12^k$

$r = 132.0\ in$

$t = 7.0\ in$

$f'_c = 4\ ksi$

$\frac{W}{r t f'_c} = 0.237$

$m = 0.025$

$\frac{M_u}{W R \phi} = 1.25$

$M_u = 1.0854 \times 10^5$ K-IN

NODE 11

$W = 786.41^k$

$r = 129\ in$

$t = 7.0\ in$

$f'_c = 4.0\ ksi$

$\frac{W}{r t f'_c} = 0.218$

$m = 0.025$

$\frac{M_u}{W r \phi} = 1.25$

$M_u = 0.9511 \times 10^5$

NODE 13

$W = 970.017^k$

$r = 135\ in$

$t = 7.0\ in$

$f'_c = 4.0\ ksi$

$\frac{W}{r t f'_c} = 0.257$

$m = 0.025$

$\frac{M_u}{W r \phi} = 1.25$

$M_u = 1.2277 \times 10^5$ K-IN

Genoa 3 Stack — Ultimate Strength Method

$F_y = 40\,ksi$ $\phi = .75$

At Node 15

$W = 1163.287^k$

$r = 192.5$ in

$t = 7.0$ in

$f_c' = 4.0\,ksi$

$\frac{W}{r t f_c'} = 0.292$

$m = \ldots(f_y/f_c') = 0.0383$

$\frac{M_u}{W r \phi} = 1.55$

$M_u = 192705.8^{in-k}$

Node 14

$W = 1065.37^k$

$r = 138.75$ in

$t = 7.0$ in

$f_c' = 4.0\,ksi$

$\frac{W}{r t f_c'} = 0.274$

$m = 0.032$

$\frac{M_u}{W R \phi} = 1.4$

Node 16

$W = 1263.787^k$

$r = 146.25$ in

$t = 7.0$ in

$f_c' = 4.0\,ksi$

$\frac{W}{r t f_c'} = 0.309$

$m = 0.045$

$\frac{M_u}{W r \phi} = 1.75$

$M_u = 2.426 \times 10^5$ k-in

Node 17

$W = 1366.87^k$

$r = 150.0$ in

$t = 7.0$ in

$f_c' = 4.0\,ksi$

$\frac{W}{r t f_c'} = 0.325$

$m = 0.053$

$\frac{M_u}{W r \phi} = 2.05$

$M_u = 3.152 \times 10^5$ k-in

GENOA 3 STACK ULTIMATE STRENGTH METHOD	REF.

$f_y = 40$ ksi $\phi = 0.75$

At Node 18

$W = 1474.5^k$

$r = 153.75$ in

$t = 7.0$ in

$f'_c = 4.0$ ksi

$m = 0.065$

$\frac{W}{r t f'_c} = 0.343$

$\frac{M_u}{W r \phi} = 2.2$

$M_u = 3.7405 \times 10^5$ k-in

Node 20

$W = 1716.417^k$

$r = 160.75$ in

$t = 8.0$ in

$f'_c = 4.0$ ksi

$m = 0.80$

$\frac{W}{r t f'_c} = 0.334$

$\frac{M_u}{W r \phi} = 2.5$

$M_u = 5.173 \times 10^5$ k-in

Node 19

$W = 1590.377^k$

$r = 157.25$ in

$t = 7.5$ in

$f'_c = 4.0$ ksi

$m = 0.074$

$\frac{W}{r t f'_c} = 0.337$

$\frac{M_u}{W r \phi} = 2.4$

$M_u = 4.502 \times 10^5$ k-in

Node 25

$W = 2500^k$

$r = 178.94$ in

$t = 10.5$ in

$f'_c = 4.0$ ksi

$m = 0.092$

$\frac{W}{r t f'_c} = 0.333$

$\frac{M_u}{W r \phi} = 2.70$

$M_u = 9.059 \times 10^5$ k-in

GENOA 3 STACK ULTIMATE STRENGTH METHOD	REF.

$f_y = 40$ ksi $\phi = 0.75$

NODE 21

$W = 1847.80^k$ ✓
$r = 164.5''$ ✓
$t = 8.0''$ ✓

$\frac{W}{r t f'_c} = 0.351$ ✓

$m = 0.0898$ ✓

$\frac{M_u}{W r \phi} = 2.55$ ✓

$M_u = 5.81 \times 10^5$ in-k

NODE 22

$W = 1990.06^k$ ✓
$r = 168.0''$ ✓
$t = 8.5''$ ✓

$\frac{W}{r t f'_c} = 0.348$ ✓

$m = 0.095$ ✓

$\frac{M_u}{W r \phi} = 2.65$ ✓

$M_u = 6.64 \times 10^5$ in-k

NODE 23

$W = 2145.72^k$ ✓
$r = 171.375''$ ✓
$t = 9.25''$ ✓

$\frac{W}{r t f'_c} = 0.338$

$m = 0.0964$ ✓

$\frac{M_u}{W r \phi} = 2.7$ ✓

$M_u = 7.45 \times 10^5$ in-k ✓

NODE 24

$W = 2315.40^k$ ✓
$r = 174.6875''$ ✓
$t = 10.0''$ ✓

$\frac{W}{r t f'_c} = 0.331$ ✓

$m = .0948$ ✓

$\frac{M_u}{W r \phi} = 2.75$

$M_u = 8.34 \times 10^5$ in-k ✓

GENOA 3 STACK ULTIMATE STRENGTH METHOD	REF.

$f_y = 40$ ksi $\phi = 0.75$

NODE 26

$W = 2697.77^k$ ✓
$r = 183.1875''$ ✓
$t = 11.0$ ✓

$\frac{W}{r t f'_c} = 0.335$ ✓

$m = 0.0884$ ✓

$\frac{M_u}{W r \phi} = 2.6$ ✓

$M_u = 9.64 \times 10^5$ in-k ✓

NODE 27

$W = 2911.43^k$ ✓
$r = 187.4375''$ ✓
$t = 11.5''$ ✓

$\frac{W}{r t f'_c} = 0.338$ ✓

$m = 0.0856$ ✓

$\frac{M_u}{W r \phi} = 2.55$ ✓

$M_u = 10.44 \times 10^5$ in-k ✓

NODE 28

$W = 3144.19^k$ ✓
$r = 191.6875''$ ✓
$t = 12.0''$ ✓

$\frac{W}{r t f'_c} = 0.342$

$m = 0.0816$ ✓

$\frac{M_u}{W r \phi} = 2.45$ ✓

$M_u = 11.07 \times 10^5$ in-k ✓

NODE 29

$W = 3412.62^k$ ✓
$r = 195.1875''$ ✓
$t = 14.0''$ ✓

$\frac{W}{r t f'_c} = 0.312$

$m = 0.072$ ✓

$\frac{M_u}{W r \phi} = 2.3$ ✓

$M_u = 11.49 \times 10^5$ in-k ✓

GENOA 3 STACK ULTIMATE STRENGTH METHOD

REF.

$f_y = 40$ ksi $\phi = 0.75$

NODE 31

$W = 4179.78^k$ ✓

$r = 200.6875''$ ✓

$t = 21.0''$ ✓

$\frac{W}{r t f'_c} = 0.248$ ✓

$m = 0.0506$ ✓

$\frac{M_u}{W r \phi} = 1.95$ ✓

$M_u = 12.27 \times 10^5$ in-k ✓

NODE 32

$W = 4698.78^k$ ✓

$r = 203.6875''$ ✓

$t = 24.0''$ ✓

$\frac{W}{r t f'_c} = 0.240$ ✓

$m = 0.0436$ ✓

$\frac{M_u}{W r \phi} = 1.75$ ✓

$M_u = 12.56 \times 10^5$ in-k ✓

NODE 33

$W = 5206.18^k$ ✓

$r = 208.1875''$ ✓

$t = 24.0''$ ✓

$\frac{W}{r t f'_c} = 0.260$ ✓

$m = 0.04271$ ✓

$\frac{M_u}{W r \phi} = 1.68$ ✓

$M_u = 13.66 \times 10^5$ in-k ✓

NODE 34

$W = 5707.28^k$ ✓

$r = 212.6875''$ ✓

$t = 24.0''$ ✓

$\frac{W}{r t f'_c} = 0.280$ ✓

$m = 0.0415$ ✓

$\frac{M_u}{W r \phi} = 1.65$ ✓

$M_u = 15.02 \times 10^5$ in-k ✓

GENOA 3 STACK ULTIMATE STRENGTH METHOD	REF.

$F_y = 40ksi$ $\phi = 0.75$

NODE 30	NODE 35
$W = 3738.9^k$	$W = 6601.367^k$
$r = 198.7$	$r = 217.1875$ in
$t = 16.0$ in	$t = 24.0$ in
$f'_c = 4.0$ ksi	$f'_c = 4.0$ ksi
$m = 0.065$	$m = 0.041$
$\frac{W}{rtf'_c} = 0.294$	$\frac{W}{rtf'_c} = 0.317$
$\frac{M_u}{WR\phi} = 2.35$	$\frac{M_u}{WR\phi} = 1.65$
$M_u = 13.094 \times 10^5$ MIN	$M_u = 17,742 \times 10^5$

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	REF.

APPENDIX B

GENOA 3 STACK FOUNDATION MAT

ANALYSIS

NES 105 (2/74)

GENOA 3 STACK FOUNDATION MAT ANALYSIS

REF.

FOUNDATION PROPERTIES

$F'c = 3$ KSC

$f_y = 60$ KSC

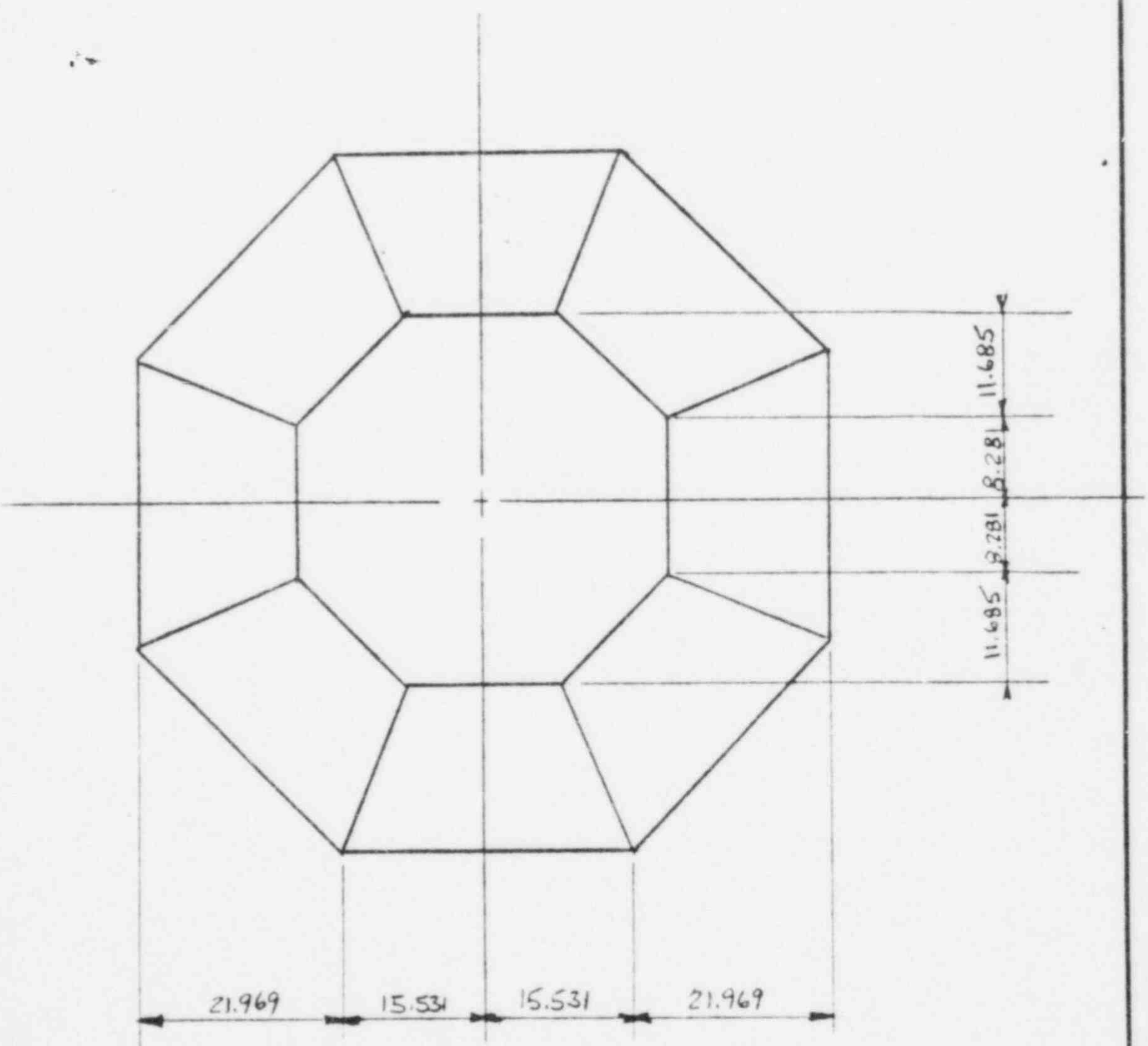
REFERENCE

THE M.W. KELLOGG COMPANY
JOB 7692
DWG 6152-ED

TOTAL DEAD WEIGHT = 11580^k

FOUNDATION AREA = $A = 0.828 d^2$ $d = 75$ FT

$A = 4657.5$ FT^2



PLAN VIEW

GENOA 3 STACK FOUNDATION MAT ANALYSIS

REF.

MOMENT FROM COMPUTER PROGRAM = 5.9798×10^5 K-IN
CALCULATING SRSS MOMENT = $\sqrt{2} \times 5.98 \times 10^5 = 8.457 \times 10^5$ K-IN

TAKING THE SHEAR AT THE BASE = 412^K * CHANGED TO 583^K WICH WILL HAVE A SMALL EFFECT ON THE RESULTS < 2% THE CORRECT VALUE WILL BE USED IN THE FINITE ELEMENT MODEL.

$M_o = 8.457 \times 10^5 + 412 \times 12 \times 7.0 = 8.803 \times 10^5$ K-IN

FACTOR OF SAFETY AGAINST OVERTURNING

$$F.S. = \frac{M_{STABILIZING}}{M_{OVERTURNING}} = \frac{37.5 \times 12 \times 11580}{8.803 \times 10^5} = 5.90$$

$$F.S. = 5.9 \geq 1.5 \text{ OK}$$

STACK IS STABLE

$$\text{CALCULATE } e = \frac{M_o}{W} = \frac{8.803 \times 10^5 \text{ K-IN}}{11580^K} = 76.02 \text{ IN}$$

$$\frac{d}{6} = \frac{75 \times 12}{6} = 150$$

TOTAL FOUNDATION UNDER COMPRESSION

DEAD LOAD COMPRESSIVE STRESS

$$\sigma_o = \frac{P}{A} = \frac{11580}{4657.5} = 2.49 \text{ KSF}$$

SECTION MODULUS FOR OCTAGONAL MAT $Z = 0.1016\, d^3$

$Z = 0.1016\,(75)^3$

$Z = 42862.5 \text{ FT}^3$

REFERENCE "FOUNDATION DESIGN HANDBOOK" By GULF PUBLISHING Co 1968

	REF.
GENOA 3 STACK FOUNDATION MAT ANALYSIS	

SOIL STRESS DUE TO EARTHQUAKE = $\frac{8.803 \times 10^5 \text{ K-IN}}{42862.5 \text{ FT}^3} \frac{1 \text{FT}}{12}$

= 1.71 KSF

MAXIMUM TOE SOIL STRESS = 2.49 + 1.71 = 4.20 KSF

MINIMUM TOE SOIL STRESS = 2.49 - 1.71 = 0.78 KSF

OUTSIDE DIAMETER OF STACK = 38'-2 3/8" = 38.20 FT

AREA = 1146 FT²

EQUIVALENT SQUARE = $\sqrt{1146}$ = 33.85 FT

1/2 SIDE = 16.925 FT

CHECK SHEAR DUE TO BENDING AT CRITICAL SECTION AT A DISTANCE d FROM EQUIVALENT SQUARE

d = 6.75 FT

SOIL STRESS @ d FROM SQUARE

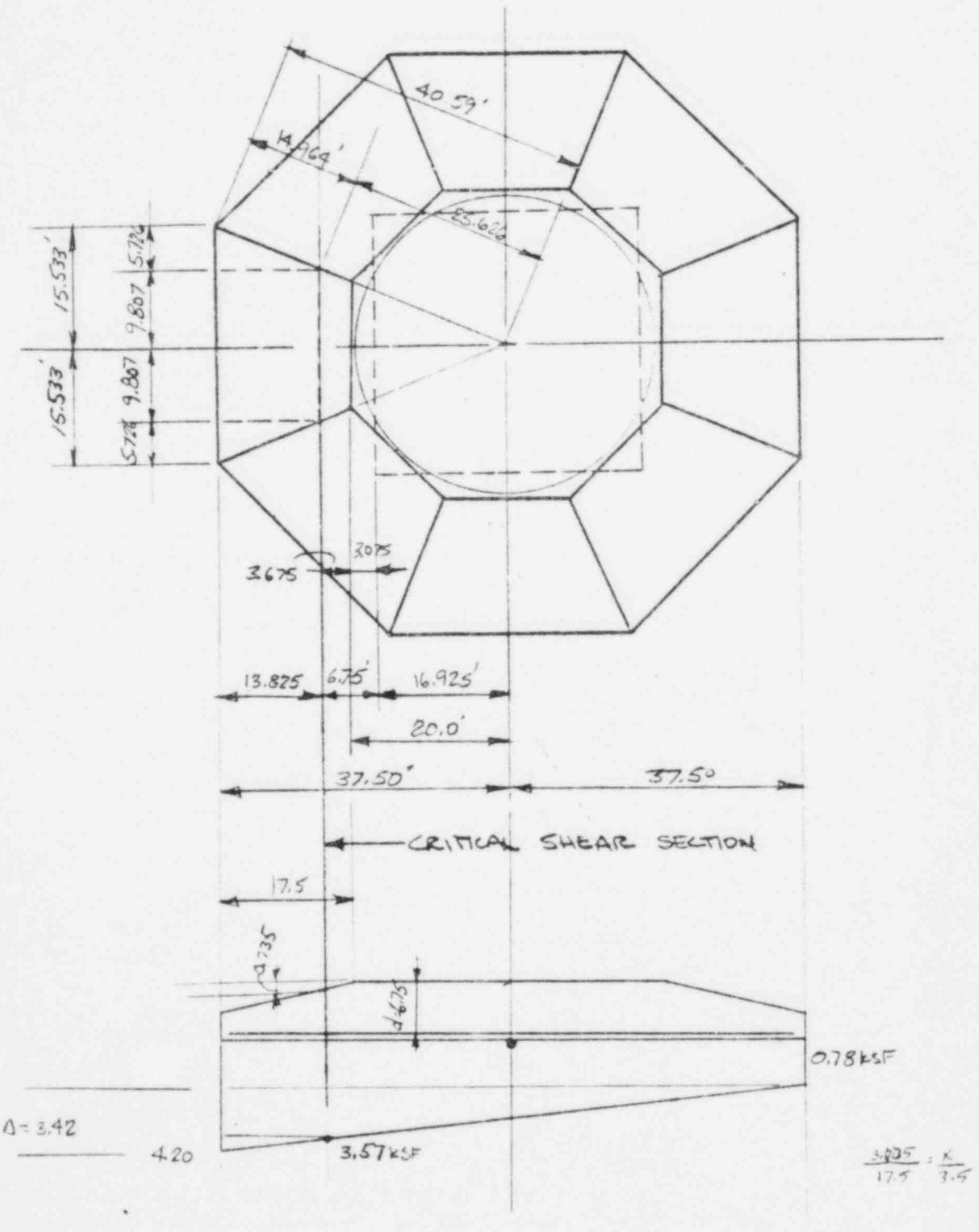
$$\frac{61.175}{75} = \frac{x}{3.42}$$

$x = 2.79$

STRESS AT THAT POINT = 2.79 + 0.78 = 3.57 KSF

GENOA 3 STACK FOUNDATION MAT ANALYSIS

REF.



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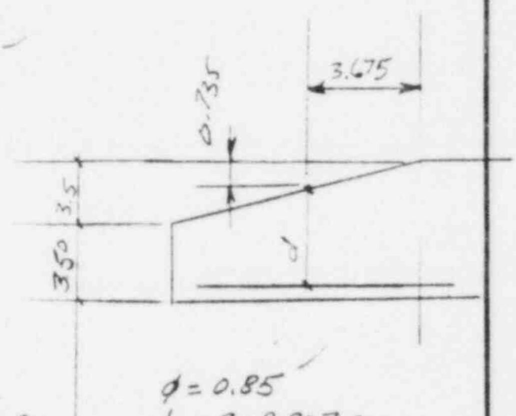
GENOA 3 STACK FOUNDATION MAT ANALYSIS

REF.

AREA	STRESS	F
9.807 × 2 × 13.825	3.57	968.05 k
2[5.726 × ½ × 13.825]	3.57	282.61 k
9.807 × 2 × 13.825]	$\frac{4.2-3.57}{2}$	85.42 k
2[$\frac{5.726 \times 13.825}{3}$]	4.2 - 3.57	33.24
	SHEAR	1369.32 k



CHECK CRITICAL SHEAR

$$N_{u2} = \frac{V_u}{\phi b d} = \frac{1369.32}{0.85 \times 2 \times 9.807 \times 6.015}$$

$\phi = 0.85$

$b = 2 \times 9.807 =$

$\frac{3.5}{17.5} = \frac{x}{3.675} = 0.735$

$d = 6.015$ FT $= 72.18$ IN

$N_u = 13.65$ K/FT² $\frac{FT^2}{144 IN^2}$

$N_u = 0.095$ KSI

$N_u = 95$ PSI — Vs CODE ALLOWABLE $2\sqrt{f'_c}$: 109.5 PSI

OK MEETS CODE

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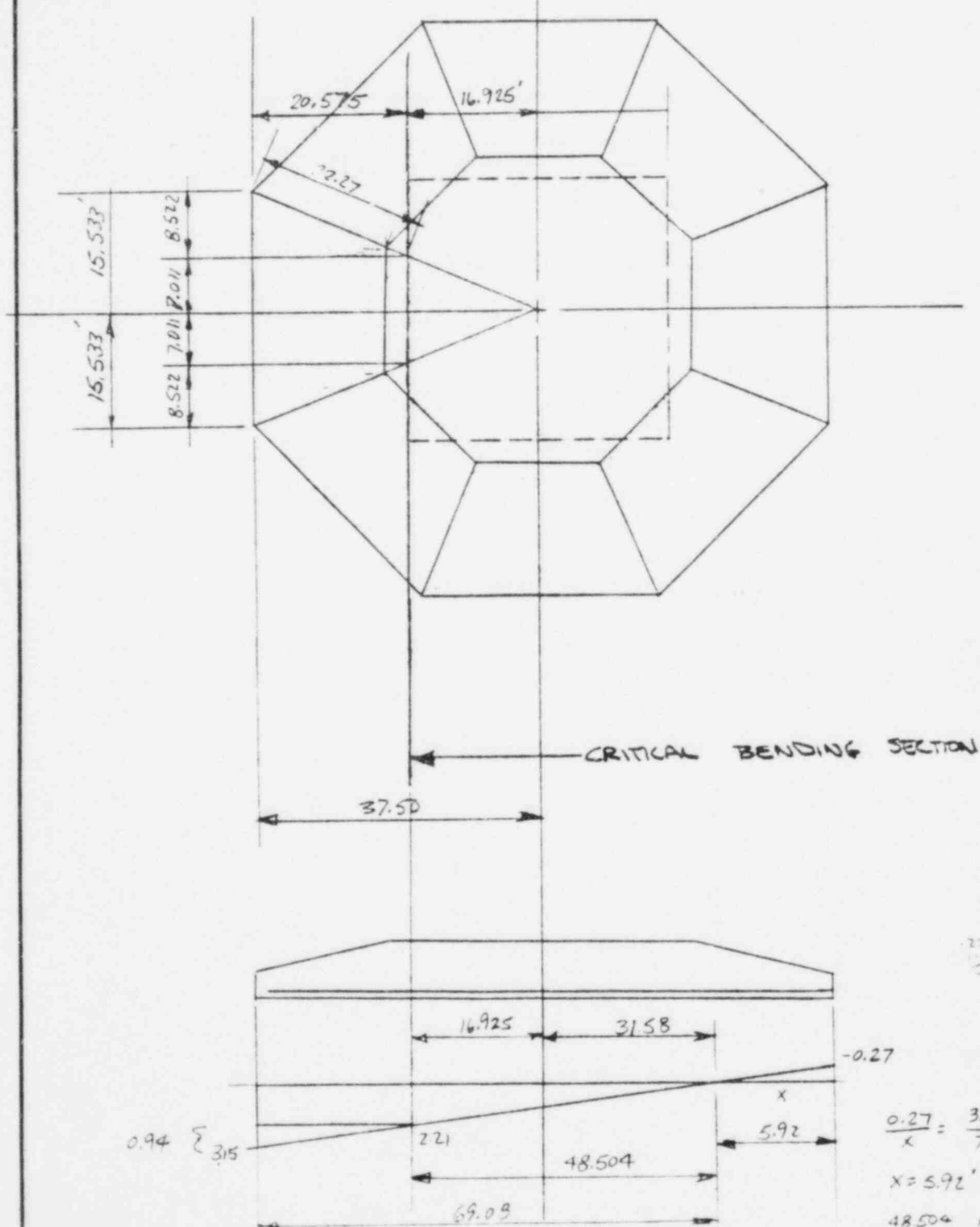
GENOA 3 STACK FOUNDATION MAT ANALYSIS

REF.



$\frac{0.27}{x} = \frac{3.42}{75}$

$x = 5.92'$

$\frac{48.504}{69.09} = \frac{x}{3.15}$

$x = 2.21$

NES 105 (2/74)

GENOA 3 STACK FOUNDATION MAT ANALYSIS

REF.

FOR BENDING

SINCE THE WEIGHT OF THE SOIL AND FOUNDATION MAT DO NOT ADD TO THE BENDING THAT WEIGHT WILL BE DEDUCTED.

$A = 4657.5 FT^2$

$V = 7 \times 4657.5$

$WEIGHT = 7 \times 4657.5 \times 0.150 = 4890.375^k$

STRESS = 1.05 K.S.F.

MAXIMUM TOE STRESS = 4.20 - 1.05 = 3.15 KSF
MINIMUM TOE STRESS = 0.78 - 1.05 = -0.27 KSF

AREA	STRESS	F	d	M
2×7.011 × 20.575 = 288.50	2.21	637.59	$\frac{20.575}{2}$	6559.16
2 × 8.522 × 20.575 × ½ = 175.34	2.21	387.50	$\frac{2}{3}$ 20.575	5315.23
7.011 × 2 × 20.575 = 288.50	$\frac{0.94}{2}$	135.60	⅔ 20.575	1859.91
2 [$\frac{8.522 \times 20.575}{3}$] = 116.89	0.94	109.88	⅔ 20.575	1507.18
				15241.48

MOMENT PER FOOT WIDTH = $\frac{14487.89 \times 12}{2 \times 7.011} = 12398.7^{k\text{-}in}$

CHECK ULTIMATE CAPACITY

$\phi = 0.9$
$f_y = 60$ ksi
$d = 6.75 \times 12$

ASSUME $a = 4.0$

$$A_s = \frac{M_u}{\phi f_y (d - a/2)} = \frac{13043.6}{0.9 \times 60 (6.75 \times 12 - 2)}$$

$A_s = 3.06 \text{ in}^2$

$$a = \frac{A_s f_y}{0.85 f'_c b} = \frac{3.06 \times 60}{0.85 \times 3 \times 12} = 6.0 \text{ in}$$

$A_s = 3.10 \text{ in}^2/\text{ft} > 2.88 \text{ in}^2/\text{ft}$

AREA STEEL SUPPLIED
#11 BARS @ 6.5 INCHES
$A_s = \frac{12}{6.5} \times 1.56 = 2.88 \text{ in}^2/\text{ft}$

NUCLEAR ENERGY SERVICES

BY R.R. DATE 11-18-80 PROJ. 5701 TASK 051
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LACBWR

GENOA 3 STACK FOUNDATION MAT	REF.

DISCUSSION OF RESULT

THE AREA OF STEEL SUPPLIED IN THE FOUNDATION MAT IS SLIGHTLY LESS THAN THAT REQUIRED BY THE ABOVE ANALYSIS ($2.88\ IN^2/FT$ VS $3.10\ IN^2/FT$). THIS ANALYSIS, HOWEVER, IS CONSERVATIVE FOR THE FOLLOWING REASONS:

1. THE CRITICAL SECTION USED IN THIS ANALYSIS NEGLECTS OTHER PORTIONS OF THE MAT WHICH MUST FAIL IN ORDER FOR TOTAL FAILURE TO OCCUR.

2. NO ALLOWANCE FOR STRESS REDISTRIBUTION OUTSIDE THE ABOVE CRITICAL SECTION IS RECOGONIZED

3. THE OCTAGONAL SHAPE FOUNDATION IS DIFFICULT TO MODEL BY SIMPLISTIC METHODS, THIS RESULTS IN CONSERVATIVE ASSUMPTIONS BEING MADE TO EASE CALCULATIONS

A FINITE ELEMENT ANALYSIS WILL BE PERFORMED ON THE FOUNDATION MAT TO MORE ACCURATELY REPRESENT THE OCTAGONAL FOUNDATION MAT. THE RESULTS OF THIS ANALYSIS WILL SHOW THAT THE FOUNDATION WILL NOT FAIL DURING THE SSE EVENT.