SEISMIC AND STRUCTURAL ANALYSIS

OF THE LACBWR STACK

USING THE NRC SITE-SPECIFIC GROUND RESPONSE SPECTRA

PREPARED FOR

DAIRYLAND POWER COOPERATIVE

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REVISION LOG

NUCLEAR ENERGY SERVICES, INC.

DOCUMENT NO. 81A0046

REV. NO.	DATE	PAGE NO.	DESCRIPTION	APPROVAL

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1. SUMMARY

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

Linear seismic analysis, using the site specific spectra and modal superposition, was performed to determine the response of the LACBWR stack for the SSE Event. Soil structure interaction effects were included using the information provided by Dames & Moore. The foundation springs reflect the updated information of the most recent boring program. The seismic response of the stack was 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 LACBWR stack will experience a failure 50 to 100 feet from its top. The surviving 250 to 300 feet of the stack will remain upright and attached to its foundation mat.

2. BACKGROUND INFORMATION

In response to recent NRC questions Dairyland Power Cooperative (DPC) requested Nuclear Energy Services (NES) to analyze the LACBWR stack. This analysis was performed using the most recent soils data from Dames & Moore,[2] most recent design codes, current NRC Regulatory Guides and Standard Review Plans, and the recently established site specific ground spectra.[1] The analysis included investigation of the following variables: soil properties, cracked, and uncracked section properties of the concrete. The results are presented within. The LACBWR stack is not a safety related structure. However, since the LACBWR stack is located adjacent to the LACBWR Reactor Containment Building and other safety related structures, the failed section of the stack may impact on these structures.

3. DESCRIPTION OF THE LACBWR STACK

As shown in Figure 3.1, the LACBWR stack is a 350 foot high, tapered, reinforced concrete structure with an outside diameter of 7.19 feet at the top and 24.719 feet at the base. The wall thickness varies from 15 inches at the bottom to 6 inches at the top. The 4 foot thick foundation mat of the LACBWR stack rests on a pile cluster composed of 78 piles. Each pile is 60 feet long with a minimum capacity of 50 tons. The drawings of Reference 3 show the diameter, thickness and the arrangement of the reinforcing steel at various heights of the stack.

4. APPLICABLE CODES, STANDARDS AND SPECIFICATIONS

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

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.

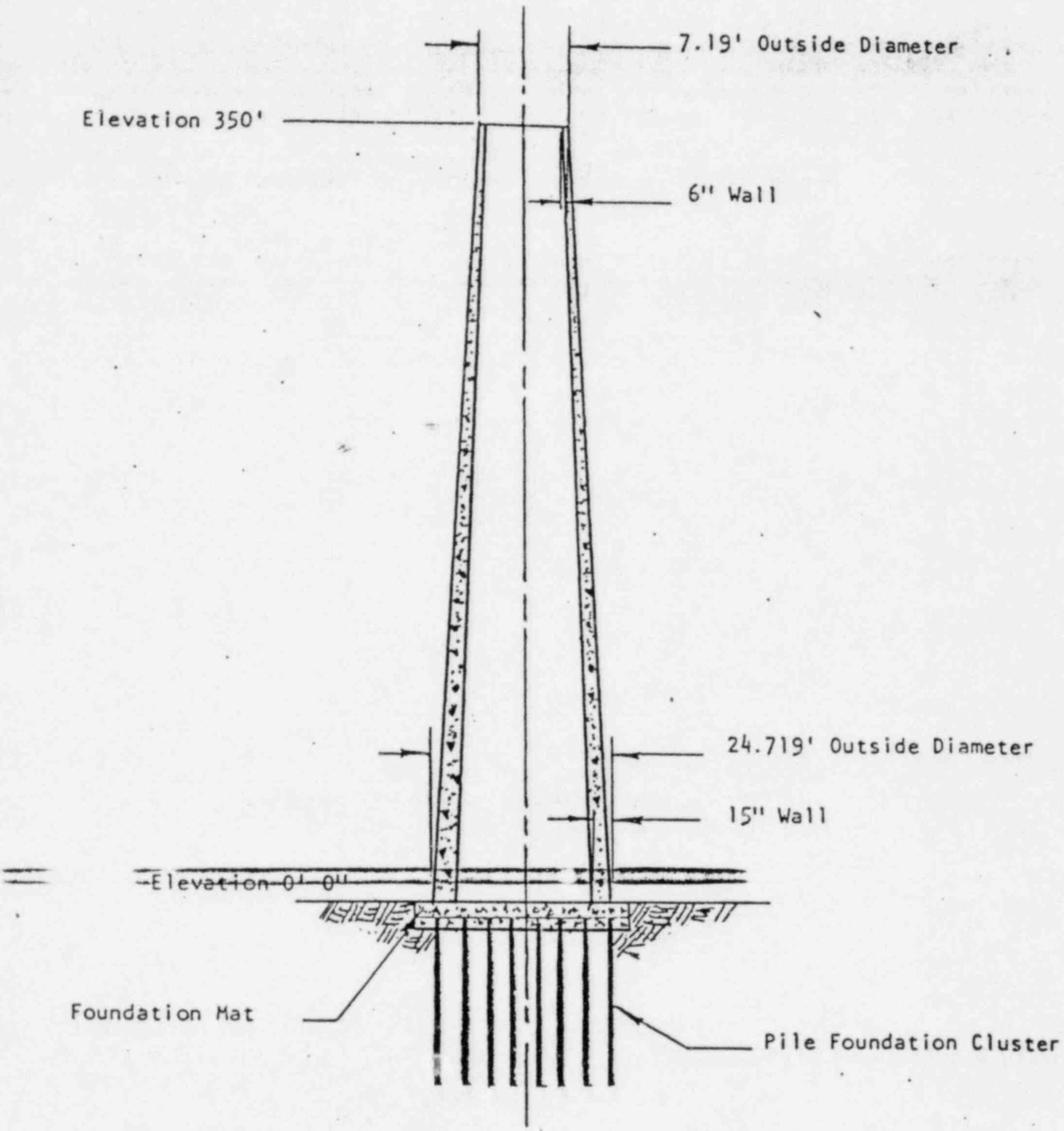


FIGURE 3.1
SCHEMATIC SKETCH OF LACBWR STACK

5. LOADS AND LOADING COMBINATIONS

The seismic lateral inertia loading on the coupled model of the stack and its foundation 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.

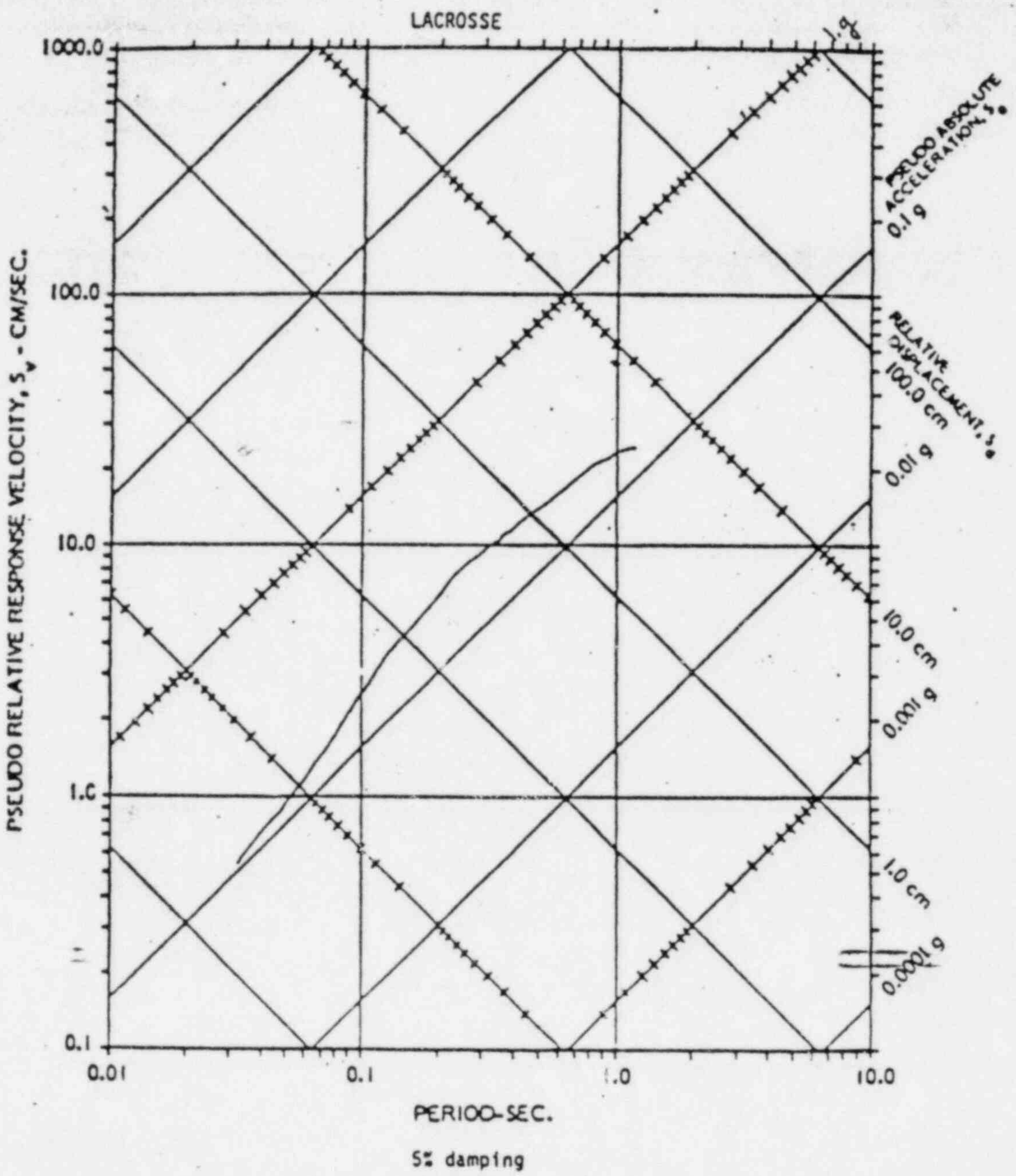


FIGURE 5.1

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 was 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 LACBWR stack mathematical model (Figure 6.1) to account for the shear and vertical deformation of the soil under the LACBWR stack foundation. To account for the variation in the soil properties and to evaluate the effect of 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 was lumped at the system nodal points. Each mass represents the tributory weight of the stack walls above and below the nodal point. Masses were 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 were 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.

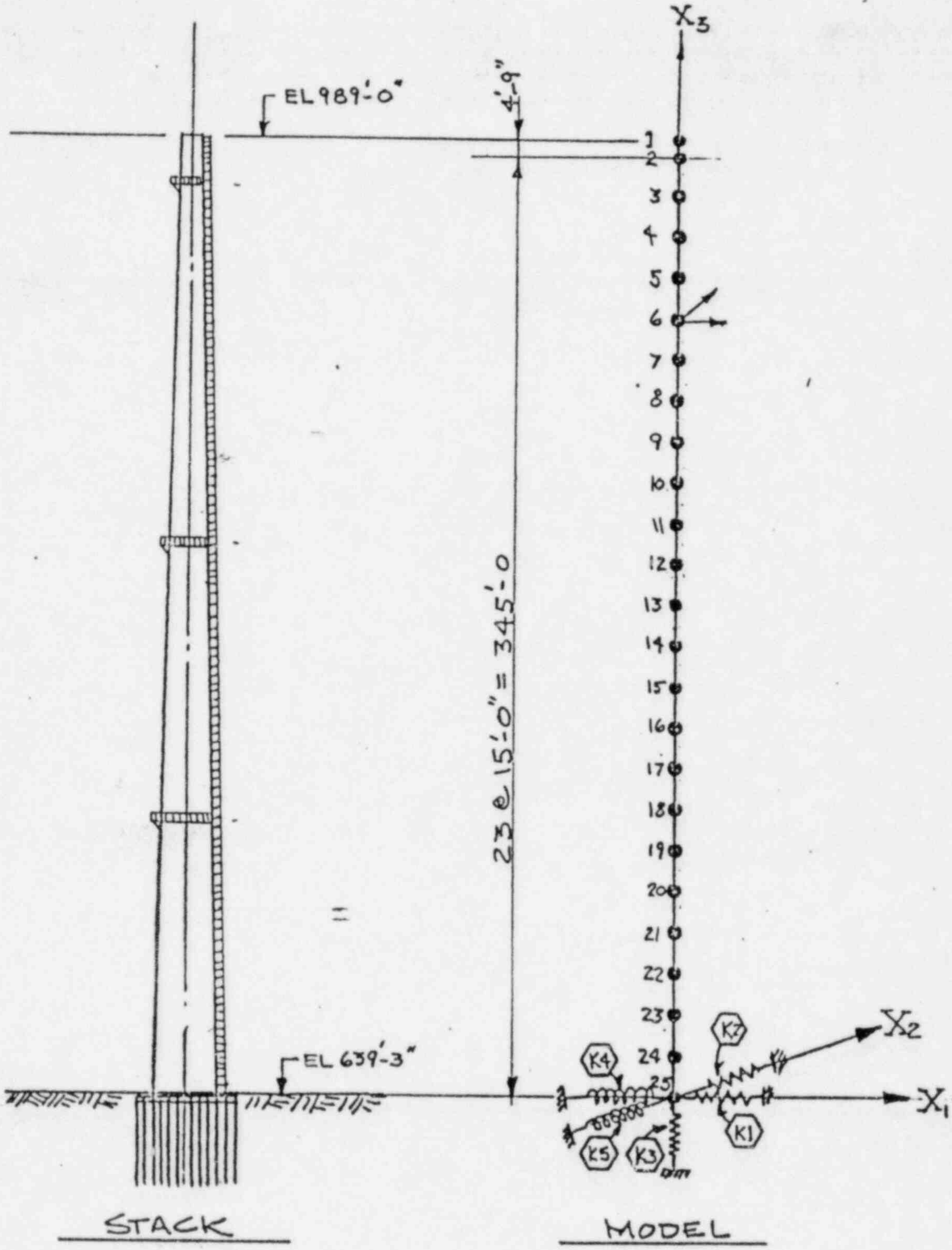


FIGURE 6.1
MATHEMATICAL MODEL OF LACBWR STACK

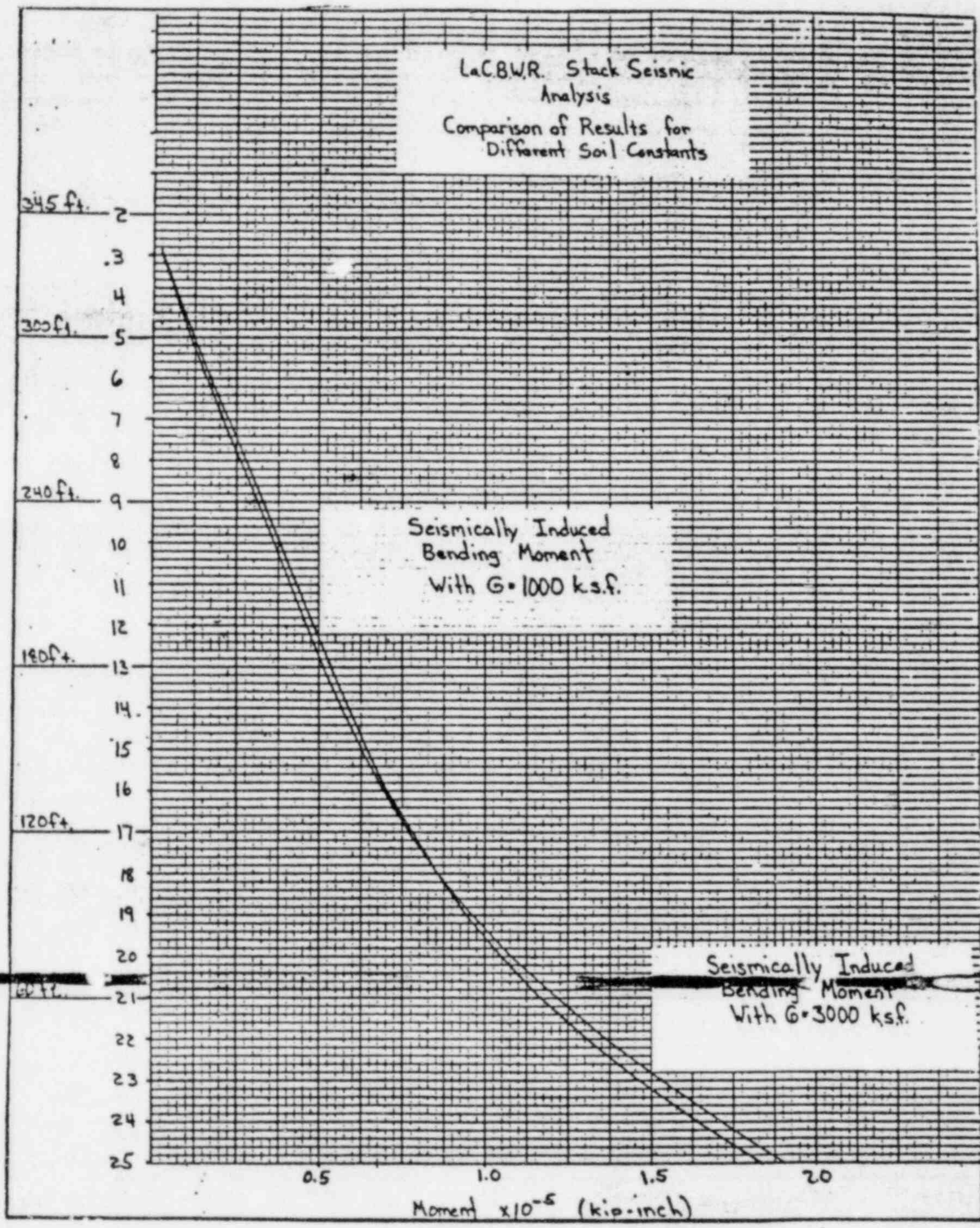


FIGURE 6.2

EFFECTS OF VARIATION OF SOIL PROPERTIES
ON STRUCTURAL RESPONSE

TABLE 6.1

NATURAL FREQUENCIES OF VIBRATION - LACBWR STACK

Mode No.	Modal Direction	Frequencies For Model 1 G = 1000 ksf Softer Foundation Spring	Frequencies For Model 2 G = 3000 ksf Stiffer Foundation Spring
1	X_2	0.476 CPS	0.508 CPS
2	X_1	0.476	0.508
3	X_2	1.477	1.609
4	X_1	1.477	1.609
5	X_2	3.513	3.777
6	X_1	3.513	3.777
7	X_3	5.848	8.632*
8	X_2	6.226	6.799*
9	X_1	6.226	6.799*
10	X_2	8.368	10.444*
11	X_1	8.368	10.444*

*Mode numbers may not correspond to those of Model 1.

TABLE 6.2

LACBWR 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						
2	86.3	6.0	1512.67	4.8	0.00317	7.82
3	91.0	6.0	1602.2	4.8	0.00300	32.15
4	95.8	6.0	1691.7	5.0	0.00296	57.88
5	100.6	6.0	1783.2	5.6	0.00314	85.01
6	105.6	6.0	1877.4	6.0	0.00320	113.61
7	110.6	6.0	1971.7	6.4	0.00325	143.69
8	115.6	6.0	2065.9	8.02	0.00388	175.24
9	120.6	6.0	2160.2	17.6	0.00815	208.26
10	130.6	~~6.25~~	2441.6	22.88	0.00937	244.19
11	140.6	6.5	2738	27.6	0.01008	284.63
12	150.6	6.875	3103.7	31.2	0.01005	330.06
13	160.6	7.25	3493	34.76	0.00995	381.78
14	170.6	8.125	4135	39.5	0.00955	441.17
15	180.6	9.0	4852	42.66	0.00879	511.13
16	190.6	9.0	5135	45.82	0.00892	589.16
17	200.6	9.0	5417	48.19	0.00890	671.6
18	212.6	9.0	5757	50.56	0.00879	758.9
19	224.6	9.0	6096	52.93	0.00868	851.5
20	236.6	9.125	6522	53.72	0.00824	950.39
21	248.6	9.25	6956	54.5	0.00784	1056.
22	260.6	9.375	7400	53.4	0.00722	1167.8
23	272.6	59.50	7853	51.6	0.00657	1287.3
24	284.6	14.0	11902	48.0	0.00403	1448.9
25	296.6	15.0	13270	48.0	0.00362	1642.29

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 were obtained using the equations shown in Figure 6.3. These equations were taken from Reference 4.

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:

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

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 was performed using the the Lanczos Modal Extraction Method.

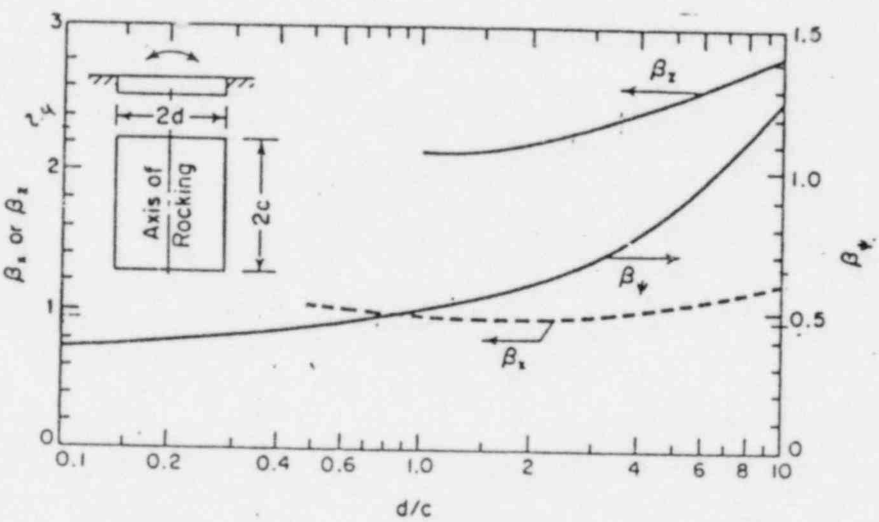
6.1.4 Dynamic (Seismic) Load Analysis

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:

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

Motion	Spring Constant	Reference
Vertical	$k_z = \frac{G}{1-\nu}\beta_z\sqrt{4cd}$	Barkan (1962)
Horizontal	$k_x = 4(1+\nu)G\beta_x\sqrt{cd}$	Barkan (1962)
Rocking	$k_\psi = \frac{G}{1-\nu}\beta_\psi 8cd^2$	Gorbunov-Possadov (1961)

(*Note:* values for β_z, β_x, and β_ψ are given in Fig. 10-16 for various values of d/c)



Coefficients β_x, β_z, and β_ψ for rectangular footings (*after Whitman and Richart, 1967*).

FIGURE 6.3

SOIL SPRING CONSTANTS

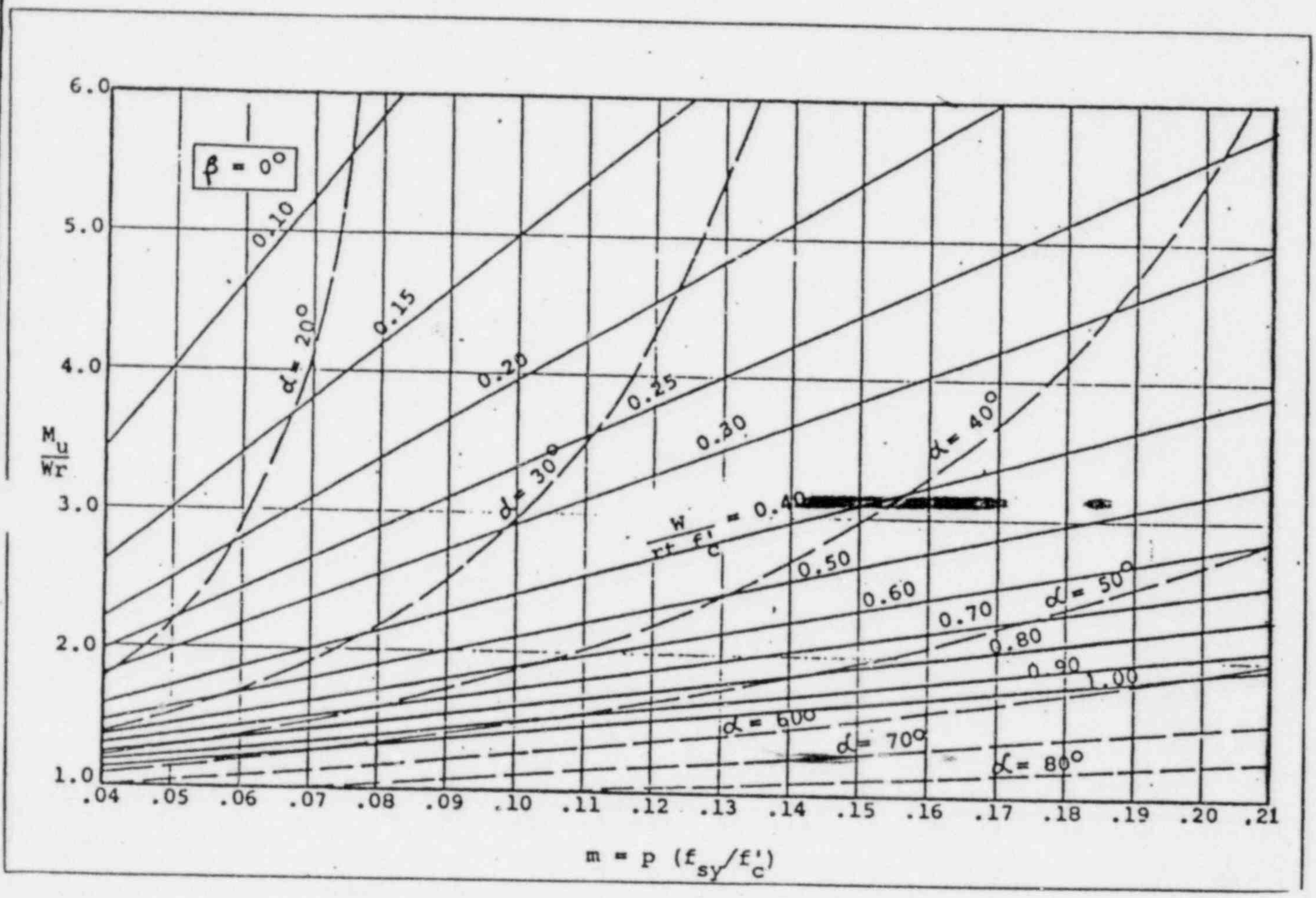


FIGURE 6.4
ULTIMATE STRENGTH CHART FOR REINFORCED CONCRETE CHIMNEY CROSS SECTION

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

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):

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

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:

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

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.

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

I = Column vector whose elements are generally unity

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

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

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 were applied as external static forces, and system response (displacements, member internal forces and stresses) were calculated. Total system response was 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) were included and higher modes with negligible participation were neglected.

6.2 STRUCTURAL ANALYSIS

The ultimate moment capacity of the stack has been determined by graphical solutions by Cannon[5] and Ru-Tsung Sun[9], and manual calculations where graphical solutions were not applicable. For the following two reasons, the graphical solutions were supplemented by manual calculations.

1) Cannon's ductility limit is based on 60 ksi steel which has a minimum strain of 8 percent and the LACBWR stack is constructed of 40 ksi steel with a minimum ductile strain of 12 percent at failure.

2) The reinforcement ratio factor for the LACBWR stack is so low that the required solutions are out of the range of the graphs for the upper nodes of the stack.

The graphs of References 5 and 9 provide very similar results and compare closely with solutions calculated manually at NES. The graphical solution of References 5 and 9 is illustrated in Figure 6.4.

The manual calculations are based on the ultimate strength design methods presented in REF 5 and 9 and as given in ACI 318-77, "Building Code Requirements for Reinforced Concrete," American Concrete Institute. The neutral axis for the stack cross-section is first established by equating compressive force to the tensile force plus the weight of the stack above the section of interest. The ultimate capacity is the summation of the moments about the neutral axis due to the compressive and tensile forces.

7. ACCEPTANCE CRITERIA

The ultimate moment and shear load-carrying capacities for 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 = 0.85fi (ACI 318-77)
Maximum stress in reinforcing steel = fy (Reference 5)
Maximum peak concrete shear stress = 4Ø f'_c (ACI 318-77)

f'_c = 3500 psi
fy = 40000 psi
For the LACBWR Stack and Foundation Mat

Recent tests at the University of Michigan[9] indicate that a value for Ø of 0.9 is reasonable. A 10 percent reduction should be taken to account for the P-Delta effect, since the deflections begin to get large prior to stack failure.

8. RESULTS OF ANALYSIS AND CONCLUSIONS

The results of the seismic analysis of the LACBWR stack performed with the Stardyne computer code are contained in Reference 8.

The natural frequencies of vibration of the LACBWR stack are given in Table 6.1. From Table 6.1 it can be seen that the stack is a low frequency system and the variation in the fundamental frequencies is small (0.476 Hz to 0.508 Hz) between the two extremes of variation in the foundation soil constants (G=1000 ksf to 3000 ksf) considered. 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 5 through 8 (height: 250 ft. to 300 ft.) exceed the allowable moment capacities (ultimate moment capacities).

The maximum ratio of SSE seismic moment to the ultimate moment capacity is 1.3. This 30% overstress during the SSE event is considerably greater than the 10 to 15% variation between the test results (Ref 6, 7 and 9) 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 mat foundation was evaluated to ensure the required structural integrity would exist during and after the SSE event. Results of the analysis shows that the foundation will not be overstressed. The piles were investigated and found to meet their requirements.

It can be concluded from the above that under an SSE seismic event, the 350-foot LACBWR stack will experience a failure 50 to 100 feet from its top. The surviving 250 to 300 feet of the stack will remain upright and attached to its foundation mat.

The LACBWR stack does not meet the current chimney code ACI-309-79 with respect to the following items:

1. The current codes require 2 layers of vertical steel, LACBWR only has 1 layer.

2. The minimum wall thickness required is 8 inches, LACBWR's wall thickness goes to 6 inches.

The LACBWR stack was built to the ACI 505-54 specification. It is felt that the above deficiencies will not affect the ultimate capacity to any important degree and can be neglected.

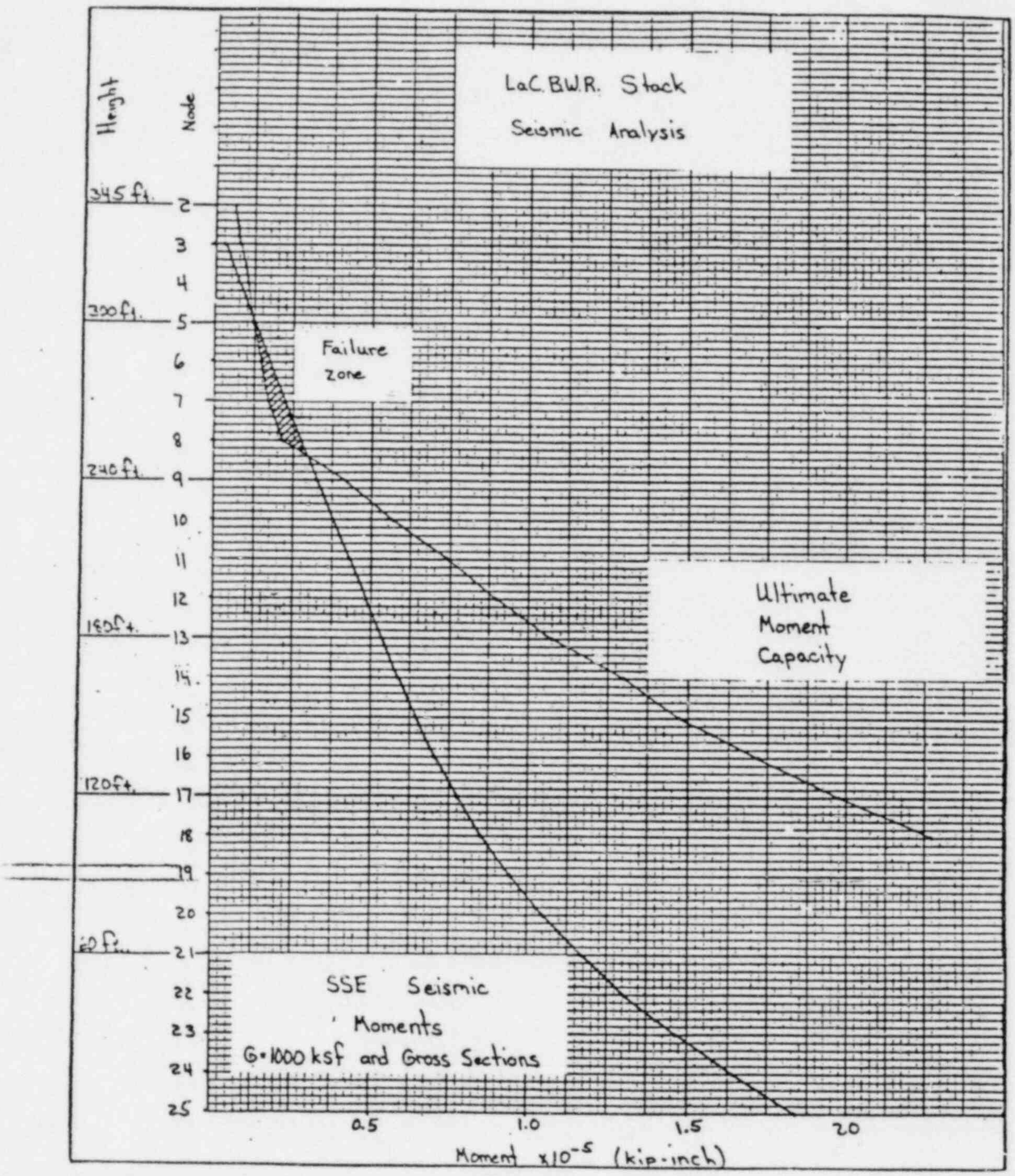


FIGURE 8.1

SUMMARY OF SEISMIC/STRUCTURAL EVALUATION

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	0.067	0.0020
3	20	0.079	.0297
4	35	0.106	.0745
5	50	0.120	< .125 NG
6	65	0.145	< .175 NG
7	80	0.171	< .224 NG
8	95	0.239	< .274 NG
9	110	0.416	.327
10	125	0.566	.380
11	140	0.734	.433
12	155	0.894	.485
13	170	1.074	.535
14	185	1.307	.588
15	200	1.474	.644
16	215	1.720	.706
17	230	1.981	.775
18	245	2.272	.854
19	260	2.602	.944
20	275	2.802	1.048
21	290	3.102	1.169
22	305	3.268	1.307
23	320	3.457	1.465
24	335	3.363	1.640
25	350	3.989	1.833

* Results from Computer Program S5300QE.
Values from this computer program are SRSS of moments due to each of the horizontal components.

TABLE 8.2

SUMMARY OF SEISMIC/STRUCTURAL EVALUATION (SHEAR)

Node	Distance from Top (ft)	Ultimate Capacity (kips)	SSE Shear * (kips)
2	5	304.2	3.406
3	20	322.2	15.35
4	35	340.2	25.08
5	50	358.6	29.04
6	65	377.6	31.09
7	80	396.6	34.10
8	95	415.6	37.72
9	110	434.6	40.35
10	125	491.2	41.62
11	140	550.8	42.59
12	155	624.4	44.76
13	170	702.6	48.70
14	185	834.3	53.99
15	200	976.0	59.82
16	215	1033.0	65.80
17	230	1089.6	71.92
18	245	1158.0	78.64
19	260	1226.2	86.37
20	275	1312.0	95.10
21	290	1399.2	104.43
22	305	1488.6	113.71
23	320	1579.6	122.43
24	335	2394.	131.62
25	350	2669.2	142.30

* Results from Computer Program S5300QE
Values from this run are SRSS.

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APPENDIX A
STACK ANALYSIS ASSUMPTIONS

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.