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Structural Review of the Palisades Nuclear Power Plant Unit 1 Containment Structure Under Combined Loads for the Systematic Evaluation Program

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FOREWORD

The U.S. Nuclear Regulatory Commission (NRC) is conducting the Systematic Evaluation Program (SEP). The Program is a plant-by-plant reassessment of the safety of eleven operating nuclear reactors that received construction permits between 1956 and 1967. Many safety criteria have changed since these plants were licensed. The purpose of the SEP is to develop a current, documented basis for the safety of older facilities.

For the Palisades Unit 1, seismic analyses for a Safe Shutdown Earthquake (SSE) had been performed in a previous study for selected plant structures and components from generic groups of equipment. The results were reported in an earlier SEP report, NUREG/CR-1833. The SSE was considered to be the Extreme Environmental condition. In the study reported here, the containment structure was selected for further evaluation of the Abnormal/Extreme Environment.

This report is a collective effort by the following people:

Nelson, T. A., and Lo, T. [Lawrence Livermore National Laboratory (LLNL)], who provided project management support and reviewed the report.

Liaw, C. Y., and Debeling, A. G. (EG&G/San Ramon Operations), who conducted the structural reevaluation of the concrete containment structure.

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ABSTRACT

A structural reassessment of the containment structure of the Palisades Nuclear Power Plant Unit 1 was performed for the Nuclear Regulatory Commission as part of the Systematic Evaluation Program. Conclusions about the ability of the containment structure to withstand the Abnormal/Extreme Environment are presented.

The reassessment focused mainly on the overall structural integrity of the containment building for the Abnormal/Extreme Environment. In this case, the Abnormal Environmental condition is caused by the worst case of either a Loss-of-Coolant Accident or a main steam line break. The Extreme Environmental condition is the Safe Shutdown Earthquake.

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CHAPTER 1: INTRODUCTION

1.1 SCOPE OF WORK

Structural reassessment of nuclear power plants is one facet of the Systematic Evaluation Program (SEP), conducted by the Nuclear Regulatory Commission (NRC). This report is a structural review of the containment building of the Palisades Nuclear Power Plant Unit 1. We evaluated the overall structural integrity of the containment building for the Abnormal/Extreme Environmental condition as defined in the ASME Boiler and Pressure Vessel Code, Section III (ASME code). In this instance, the Abnormal load case is that induced by a Loss of Coolant Accident (LOCA), and the Extreme Environmental load case is induced by the Safe Shutdown Earthquake (SSE). It is important to point out that, in this report, LOCA includes both the primary and secondary loop break cases.

Two previous SEP reports served as the basis for this work: SEP Containment Analysis and Evaluation for the Palisades Power Plant,¹ which defined the LOCA Loading, and Seismic Review of the Palisades Nuclear Power Plant Unit 1 as Part of the SEP,² in which the plant was analyzed for SSE Load.

We based our analysis on the LOCA discussed in Ref. 1: pipe breaks in the primary and secondary systems. The seismic event we used is described in Site-Specific Ground Response Spectra for SEP Plants Located in the Eastern United States.^{5,11} Our reassessment combined the accident and seismic event with existing load conditions on the containment building. We then evaluated the containment building's and its steel liner's ability to withstand the Abnormal/Extreme environmental condition. We also evaluated the steel liner system for the Extreme environmental condition, which can be a more critical loading combination. Because the primary purpose of this analysis is to evaluate the overall structural integrity of the containment building, no local load effects are considered.

1.2 STRUCTURE DESCRIPTION

The reactor containment building of Palisades Plant Unit 1 houses the nuclear steam supply system. This building is a vertical, cylindrical, prestressed concrete structure (Fig. 1.1). The inside diameter is 116 ft; the inside height is 189 ft. The containment walls are 3.5 ft thick, the dome is 3 ft thick, and the base slab varies in thickness between 8 ft. and 13 ft. The dome has a radius of 89 ft. 2-1/4 in. The containment building was the first in the United States to be post-tensioned, in both directions, with fully prestressed walls and dome. Each of the 845 tendons is stressed to about 800,000 lb., and each contains ninety 1/4-in.-diameter, high-tensile steel wires.

The post-tensioning system consists of:

- 1) Three groups of 55 dome tendons oriented at 120^0 to each other for a total of 165 tendons anchored at the vertical face of the dome ring girder.
- 2) 180 vertical tendons anchored at the top surface of the ring girder and at the bottom of the base slab.
- 3) Six groups of 87 hoop tendons enclosing 120^0 of arc for a total of 522 tendons anchored at the six vertical buttresses.

The design strengths of the concrete are 5,000 psi at 28 days for the shell and 4,000 psi at 90 days for the base slab. The prestressed concrete dome has reinforcing steel bars on both outside and inside surfaces. The reinforcing bars on the outside surface are #9 (12 in. square mesh). The inside reinforcing bars are #6 (18 in. square mesh).

The prestressed concrete cylindrical wall is reinforced on the outside surface in both vertical and hoop directions. The bottom 13 ft. of the inside surface of the wall is also reinforced in both vertical and hoop directions.

Access to the structure for personnel and equipment is through a double-locked door and a 12 ft. 0 in. clear-diameter, double-gasketed single door. An emergency personnel escape is also provided by a double-locked door.

The massive reinforced concrete foundation of the containment building sits on compact glacial deposits and very dense, fine sand. The bedrock is at an elevation of about 440 ft. The grade elevation of the soil surface is 590 ft.

Fig. 1.1. Containment building.

The interior surface of the concrete shell is lined with a 1/4 in. thick ASTM A-442 carbon steel plate. The liner plate functions as a gas barrier to prevent uncontrolled release of fission products from the reactor building during operation and also during a large Loss of Coolant Accident (LOCA). The liner is not relied upon to help the concrete maintain its structural integrity. Figure 1.2 shows the arrangement of the liner system. Figure 1.3 shows some typical details of the system.

At the cylinder portion of the liner, ASTM A-36 stiffener angles are welded longitudinally to the liner at 15 in. intervals. The stiffener angles are, typically, L3x2x1/4. An intermittent fillet weld is used between the liner and the anchors. The typical weld dimensions are 3/16 in. x 4 in. at 12 in. spacing. Horizontal ASTM A-36 channels, angles, and flat bars are attached to the liner plate as well as to the longitudinal angles. The rolled structural shapes and flat bars stiffen the liner plate during the erection and placement of the concrete, and they anchor the liner to the hardened concrete.

Construction of the liner system at the dome is similar to that at the cylinder. The exception are the angles, which are used both as stiffeners and as anchors. The angles are oriented in the hoop direction.

On the base slab, the liner plate is welded to embedded beams. An 18 in.-thick concrete slab is placed over the liner, and a leak-chase system is placed over the liner plate weld seams, which are composed of 1/4 in.-butt welds.

At the junctions where the cylinder intersects the dome and the base slab, horizontal channels or angles are attached as anchors near the locations of maximum change in meridional curvature. The details of typical liner construction at penetrations and polar-crane brackets are not described here because they were not evaluated.

1.3 LOADS AND LOAD COMBINATIONS

Two LOCA conditions were considered in this analysis: 1) the primary system pipe break, and 2) the secondary pipe break. To evaluate the containment building for the Abnormal/Extreme Environmental conditions, the following load conditions were analyzed:

Fig. 1.2. General arrangement of liner system.

Fig. 1.3. Typical liner anchor details in the cylindrical portion of the containment.

a. Deadweight loads (D)

Deadweight loads were generated by multiplying concrete weight density (150 lb/ft^3) by the structural volume. The dimensions used to determine the structural volume were based on the structural drawings supplied by Consumer Power Company (CPCO).

b. Prestress loads (F)

The response to prestress loads was extracted from information contained in Ref. 3, The Palisades Plant Preliminary Description and Safety Analysis Report (PDSAR), Amendment 1, Figure 2.12.2.3. Figure 3.3 of Sec. 3 illustrates the values used. No new analysis was performed for this load case. For the liner system, we assumed shrinkage of the concrete (prior to prestressing) of 100μ , where μ represents strain in micro-inch/inch.

c. Pressure loads (P)

According to Ref. 1, the peak post-accident containment pressure (Pa) for both the primary and secondary system pipe breaks is 68 psia (Figs. 1.4 and 1.5). This pressure is very close to the original design pressure of 55 psig (or 69.7 psia) given in the FSAR.⁴ Therefore, a relative pressure of 55 psi was applied to the structure for the pressure load case. For the liner system evaluation under the Extreme condition, we assumed a vacuum pressure (Pv) of -3 psig (11.7 psia) inside the containment, as given in the FSAR.⁴

d. Thermal Loads (T)

Figure 1.6 (Fig. 3.12T of Reference 1) gives a peak containment atmosphere temperature of 292°F for the primary system pipe break case. A temperature of about 410°F is given for the single-steam-generator blowdown of the secondary system pipe break case (Fig. 1.7). Accurate information about the temperature gradient in the concrete wall and dome is not available. It is estimated that the steel liner inside surface

Fig. 1.4. Containment pressure response for primary loop break.

Fig. 1.3. Containment pressure response for secondary loop break.

Fig. 1.6. Containment temperature response for primary loop break.

Fig. 1.7. Containment temperature response for secondary loop break.

will be heated to a temperature nearly equal to the high atmosphere-temperature, and only a small portion of the concrete wall will actually "see" a high temperature gradient. Because of the short duration of the high accident-temperature, it was decided that the following operating-condition temperatures be used for the concrete:⁸ summer--73° F at the outside containment wall and 123° F at the inside containment wall; winter--minus 1° F at the outside containment wall and 85° F at the inside containment wall. The stress-free temperature in the concrete was assumed to be 70° F.

The steel liner plate is only 1/4 in. thick and has a much higher thermal conductivity than the concrete wall; the liner plate was therefore assumed to have the same temperature as the containment atmosphere. The containment atmosphere temperature of the secondary system pipe break case (410° F) is much higher than the containment atmosphere temperature of the primary system pipe break case (292° F). The liner temperature was assumed, conservatively, to be 410° F for the thermal load calculation.

The following thermal load component usually does not need to be considered separately if a composite liner-concrete section is used in evaluating loads on the section: the additional equivalent pressure between the concrete wall surface and the liner due to the different thermal expansion in the liner and concrete. However, this load had to be included in the mathematical model of this analysis for evaluating the concrete structure because the concrete wall was modeled without the liner. The additional equivalent pressure due to differential concrete and liner expansion was estimated to be 23 psi on the wall surface under these average winter wall temperatures: 410° F in the liner and 43° F in the concrete. The stress responses to this additional equivalent pressure can be obtained by multiplying the response-to-pressure-load case by the factor 23/55.

e. Seismic loads

Reference 5 suggested a set of site-specific SSE horizontal ground response spectra for SEP plants, including the Palisades site. In a subsequent study (Ref. 11) the original spectra developed for the Palisades site were modified to account for the site amplification

Table 1.1. Horizontal site-specific spectral accelerations.

Pseudo acceleration (cm/sec ²) (5% damping)		
Period	Without Site Amplification ⁵	With Site Amplification ¹¹
0.03	102.50	205.0
0.04	122.29	244.58
0.05	130.19	260.38
0.08	152.05	304.10
0.10	179.69	359.38
0.20	214.77	429.54
0.30	224.41	448.82
0.40	218.32	430.09
1.00	174.57	174.57

effects. The vertical SSE response spectra are two-thirds of the horizontal response spectra.¹⁰

We made a seismic reanalysis of the Palisades containment building using the same structural model reported in Ref. 2, but with the site-specific spectra, including site amplification. The spectral values are shown in Table 1.1. The reanalysis was necessary because the seismic responses reported in Ref. 2 were based on 0.2g R.G. 1.60 spectra, rather than the site-specific spectra which were developed later. The structural damping of the containment shell structure was increased to 7% to account for the significant cracks expected to develop under LOCA conditions. Figures 1.8 and 1.9 present the results of the reanalysis for the site-specific spectrum (SSSP). Three soil cases were considered, following the approach of Ref. 2. The results due to the site-specific spectrum are significantly lower than both those of the 0.2g R.G. 1.60 spectrum and the licensee's original design seismic loads. The vertical response throughout the containment building is 0.24g from the present analysis. This response results primarily from a mode where the structure acts as a rigid mass on the vertical soil spring.

Because the primary purpose of this analysis is to evaluate the overall structural integrity of the containment building, no local load effects are considered. We assumed that the areas around the penetrations are

Fig. 1.3. Shear distribution in the containment building for the lower, median, and upper soil cases due to site-specific spectrum (SSSP), R.G. 1.50 (0.2g) spectrum, and the original seismic loads.

Fig. 1.9. Moment distribution in the containment building for the lower, median, and upper soil cases due to site-specific spectrum (SSSP), R.G. 1.60 (0.2g) spectrum, and the original seismic loads.

sufficiently strengthened so that they are stronger than the remainder of the structure. Therefore, we did no structural evaluation for these local areas and considered the structure to be axisymmetrical. The effects of live-loads, such as snow load, are considered small, and are therefore neglected. The total combined Abnormal/Extreme Environmental load for the containment building is the sum of all the load cases discussed above.

1.4 MATERIAL PROPERTIES

All material property values used in the analysis were extracted from section 5.1.3 of Ref. 4. Following is a list of these values for various loading conditions.

D = Dead load
F = Prestress
T = Thermal
P = Pressure
E = Earthquake

Materials	Loading conditions	
	D, F, T	P, E
Concrete walls		
E (Young's modulus) (psi)	2.7×10^6	5.5×10^6
ν (Poisson's ratio)	0.17	0.17
α (coefficient of thermal expansion) (in./in./°F)	5.0×10^{-6}	-
f'_c (compressive strength) (psi)	5000	5000
Steel Liner (ASTM A-442)		
E (Young's Modulus) (psi)	30×10^6	30×10^6
f_y (minimum yield stress) (psi) 34,000	34,000	-
ν (Poisson's ratio)	0.30	0.30
α (coefficient of thermal expansion) (in./in./°F)	6.5×10^{-6}	-
Liner anchor (ASTM A-36)		
E (Young's Modulus) (psi)	30×10^6	30×10^6
ν (Poisson's ratio)	0.30	0.30

1.5 PREVIOUS ANALYSES OF CONTAINMENT BUILDING

Many analyses have been performed for various load conditions. It is not our purpose to review all earlier work. We discuss only those analyses which dealt with load combinations similar to those considered in this report.

Amendment 1 of the Preliminary Description and Safety Analysis Report³ discussed a design-accident condition including dead load, prestress, thermal load, internal pressure, wind, and earthquake. The actual thermal gradients used in this analysis are not clear from the information available in Ref. 3, but they are presumably the same as those given in the FSAR (Ref. 4): i.e., 283°F inside and 10°F outside. The internal pressure load is 55 psig, as given in FSAR. The earthquake load is based on Housner's spectra with 0.2g peak ground acceleration.

The containment-building shell was modeled using an axisymmetrical solid-finite-element system. The axial force and the shear and moment distribution along the wall and dome were presented in Figure 2.12 of Reference 3. Amendment 4 of Reference 3 gave stress results for the liner plate and tendon and mat reinforcing bars, but this amendment did not give additional information on how the stresses were computed.

The FSAR of Palisades⁴ described the load combinations used in designing the containment liner. The FSAR also furnished the computed concrete and reinforcing steel stresses at several sections of the wall and dome. A separate supplement to the "Response to NRC Seismic Question" (Item 2.A of Ref. 7) presented a calculation for a cracked concrete section of a typical wall section. The calculation seems to indicate that the force and moment for the section were obtained from a finite-element model which included both the concrete and the liner. The stresses were calculated using a technique in which the stress distribution is first determined for the uncracked concrete section. The concrete is then assumed to crack, and the neutral axis therefore shifts until force equilibrium is achieved between the concrete in compression and the reinforcing steel in tension. This technique was applied in Reference 7 to the combined loads, which included thermal and other nonthermal loads.

CHAPTER 2: SUMMARY AND CONCLUSION

2.1 CONCRETE STRUCTURE

The concrete containment shell structure was analyzed for these combined load conditions: dead weight, prestress, accident pressure (55 psig), thermal loads (410° F in the liner and operating temperatures in the concrete), and seismic loads of 0.21g, site-specific spectra. The structure was first analyzed for the above load cases on the assumption that the concrete section was uncracked. The stresses in the concrete and reinforcing steel were then evaluated, based on cracked concrete sections. In considering the cracked concrete section, the self-relieving effects of thermal loads were included. The results indicate that the highest stress in reinforcing steel is 15 ksi, which occurs about 14 ft above the base. The compressive concrete stresses are below 1000 psi. The maximum shear stress in the concrete is less than 220 psi. All concrete and reinforcing steel stresses are within the allowable ranges given by ASME Boiler and Pressure Vessel Code Section IV, Division 2, Articles CC-3420 and CC-3520. The concrete structure is therefore considered adequate to withstand the Abnormal/Extreme loads of a primary or secondary system pipe break which is combined with a SSE event.

2.2 LINER PLATE SYSTEM

The liner system was evaluated for the Extreme Environmental and Abnormal/Extreme Environmental conditions. Both conditions include the SSE seismic loads. The Abnormal/Extreme conditions also includes the accident pressure and temperature from the blowdown of one steam generator.

The liner system near the cylinder-to-base junction was evaluated because it is the most critically loaded point. The liner strains and the anchor movement and forces were computed and then compared with the allowables specified in ASME Code Section III, Division 2, Subsection CC. From the evaluation, we concluded that the existing design of the liner plate system possesses sufficient capacity against failure in the event of an SSE or an SSE plus LOCA.

CHAPTER 3: ANALYSIS OF CONTAINMENT BUILDING

3.1 ASSUMPTIONS

The containment building was modeled by a finite-element system for all load cases, with the exception of the seismic analysis. Seismic responses were calculated from an analysis using the stick model from Ref. 2, which included soil structure interaction effects. The following assumptions were made in constructing the finite-element model.

1. Only the containment shell structure was modeled. The structure was assumed to be axisymmetrical. No internal structure was included in the model because the interaction between the containment shell and the internal structure is expected to have a minimal effect on the containment shell structure.

2. Because the model was not used for the seismic analysis, the foundation (including the building base) was assumed to be completely rigid. It was therefore not necessary to include the foundation in the model. This is a conservative assumption for concrete stresses near the cylinder and base junction, which are caused by loads other than seismic loads.

3. In computing the section loads, the concrete section was assumed to remain elastic (no cracking of the concrete). After the force and moment of the section were obtained from the elastic analysis, a cracked-section analysis was performed. The cracked-section analysis took into account the self-limiting nature of the thermal load.

4. In evaluating the section loads, this conservative assumption was made: the liner made no contribution to the structural stiffness.

5. During a LOCA, the temperature of the liner plate was assumed to be the same as the containment atmosphere-temperature. The concrete wall and dome remained at the operating temperature and had a linear gradient throughout their thickness. This is a reasonable assumption, because previous thermal transient analyses (such as those shown in the FSAR) indicate that only very small portions of concrete near the inside liner will experience the highest temperatures. The major portion of the concrete wall will remain at the operating temperature throughout the accident.

3.2 MATHEMATICAL MODEL

The load analysis of the containment building was performed using a finite-element mathematical model to depict the structure and the computer code SAP4. Figure 3.1 illustrates the model which utilized 2-D axisymmetric elements. Four layers of elements were used through the thickness. The following constraints were present: fixed footing-nodes in both the horizontal and vertical directions. The horizontal direction was constrained for the center-line node, at the top of the dome for all nonseismic loads.

As mentioned previously, for the purpose of determining section loads in this mathematical model, cracking was assumed not to occur. The analysis was performed for each of the load cases, using a linear elastic approach. We performed a verification analysis, using published shell stress equations. The verification analysis compared favorably with values predicted by SAP4.

3.3 METHOD OF ANALYSIS

The SAP4 finite element analysis generates only radial, meridian, hoop, and shear stresses for each element. It is necessary to determine the bending moment and axial force across the thickness of the shell in order to perform the cracked-section stress analysis. This was accomplished by calculating the appropriate meridian and hoop forces acting on each element, and then using this force distribution across the thickness to determine the hoop and meridian bending moments. To combine the loads we summed dead weight, prestress, pressure, thermal, and seismic loads. We included the additional pressure due to thermal expansion of the liner plate by increasing the pressure load response with the factor 23/55. This procedure was discussed in Section 1.3.

After the combined loads of a section were determined, an elastic bending-section analysis was performed to determine whether or not the section cracked. If the section did not crack, the concrete and reinforcement stresses were computed from the simple bending-section analysis. If cracking occurred within the section, the stresses were calculated using the following approach.

Fig. 3.1. Mathematical model of the containment building.

1. The total axial load on a section was divided into three groups.
  - a. P_a , This includes loads which always act at the same location of the section, regardless of whether or not the section is cracked. An example is dead load.
  - b. P_c , This includes loads which always act at the center of the uncracked portion of the cracked section. An example is pressure load.
  - c. P_t , This is the thermal load. It acts at the center of the cracked section and is proportional to the effective area (A_c) of the cracked section, i.e.:

$$P_t = (A_c/A_o) P_{to}$$

where A_o is the sectional area and P_{to} is the thermal axial load of the uncracked section.

2. The axial loads just discussed cause the total axial moment about the midsection. Following is a discussion of the total axial moment.
  - a. M_a , the moment due to P_a .
  - b. M_c , the moment due to P_c . Its value varies with the location of the center of the cracked section

$$M_c = P_c (t/2 - d_o)$$

where t is the thickness of the section and d_o is the distance from the compressive fiber to the center of gravity of the cracked section. These relationships are shown in the following diagram.

c. M_t , the moment due to thermal load. This includes two parts. One part is caused by the thermal gradient and is proportional to the cracked moment of inertia; the other part is caused by P_t . Therefore

$$M_t = (I_c/I_o) M_{t0} + (t/2 - d_o) P_t$$

where I_c is the moment of inertia of the cracked section, and I_o is the moment of inertia of the uncracked section.

3. The properties of the section give us the following relationships.

$$n = E_s/E_c$$

$$A_o = bt$$

$$I_o = bt^3/12$$

$$A_c = bkd + (n-1) A_s' + nA_s$$

$$d_o = [1/2b (kd)^2 + (n-1) A_s' d' + nA_s d] / A_c$$

$$I_c = b(kd)^3/12 + bkd (d_o - kd/2)^2 + (n-1) A_s' (d_o - d')^2 + nA_s (d - d_o)^2$$

By strain compatibility

$$F_c = f_c kdb/2$$

$$F_s = A_s n [(d - kd)/kd] f_c$$

$$F_s' = [(kd - d')/kd] f_c (n-1) A_s'$$

By static equilibrium

$$P_a + P_c + P_t = F_c + F'_s - F_s$$

$$M_a + M_c + M_t = F_c (t/2 - kd/3) + F'_s (t/2 - d) + F_s (d - t/2)$$

By substituting the above expressions into these two equilibrium equations and solving simultaneously for k , a seventh-order polynomial expression is obtained. The polynomial equation can be solved numerically for k . Subsequently, solutions for f_c and f_s can be obtained.

3.4 RESULTS OF ANALYSIS

Figures 3.2 through 3.6 illustrate the calculated forces and moments for each of the load cases, except for seismic loads. The prestress values were extracted from Ref. 3. The seismic loads are listed in Table 3.1.

The SAP4 results of finite element analysis show good agreement with closed-form solutions from shell analysis at locations where such solutions are applicable. For instance, SAP4 hoop forces due to pressure loading are 450 kip/ft in the cylinder and 340 kip/ft in the dome. The values of the closed-form shell solution are 459 kip/ft for the cylinder and 359 kip/ft for the dome. The meridian forces for the cylinder are 232 kip/ft from SAP4 and 230 kip/ft from the shell solution. The meridian moment at the base due to pressure is 490 kip/ft from SAP4 and 470 kip/ft from the shell solution. The thermal moment at the cylinder in the meridian direction is 192 kip/ft from SAP4 and 205 kip/ft from the shell solution.

To evaluate the concrete and reinforcement stresses, 16 cross sections were taken along the dome and cylinder. At each of the sections the cracked-section analyses described in Section 3.3 were performed for both meridian and hoop directions. Table 3.2 gives the concrete and reinforcement stresses for all sections. The results for the winter thermal case are given in Table 3.2. The results for the summer thermal case are a little lower

Fig. 3.2. Dead load.

Fig. 3.3. Prestress load.

Fig. 3.4. Pressure load.

Fig. 3.5. Winter thermal load.

Fig. 3.6. Summer thermal load.

than those for winter. According to ASME Boiler and Pressure Vessel Code, Section III, the allowable concrete stress is 0.85 f'c or 4250 psi, and the allowable stress for reinforcing steel is 36 ksi. The concrete flexural stresses are all less than 1000 psi. The maximum steel stress is about 15 ksi. This stress is located about 14 ft above the base. The shear stresses were evaluated according to ASME code articles CC-3420 and CC-3520. Among the 16 sections, the more critical are those near the base and the ring girder. In comparison with the code allowable, the lowest factor of safety near the ring girder is 1.3, and the lowest factor of safety near the base is 1.7.

Table 3.1. Section forces due to horizontal seismic load.

Section	True Elevation ft	Global Moment 10^6 kip-ft	Global Shear 10^3 kip	Force	
				Meridional kip/ft	Shear kip/ft
E	765	0	8.4	0	22.4
F	760	0.04	8.4	3.6	22.4
G	754	0.08	8.4	7.1	22.4
H	747	0.12	8.4	10.7	22.4
I	739	0.17	8.4	15.2	22.4
J	720	0.30	8.4	26.7	22.4
K	678	0.45	13.2	40.1	35.2
L	619	1.17	14.5	104.3	38.6
M	604	1.50	15.2	133.7	40.5
N	600	1.60	15.2	142.7	40.5
O	596	1.70	15.2	151.6	40.5
P	591	1.82	15.2	162.3	40.5

Table 3.2. Stresses of cracked section in concrete and steel
(winter thermal case).

Section	<u>Meridian stress, ksi</u>		<u>Hoop stress, ksi</u>		<u>Shear stress, ksi</u>
	f_c	f_s	f_c	f_s	v_c
A	0.455	3.611	0.627	2.712	0.008
B	0.545	4.284	1.964	0.866	0.017
C	0.536	2.255	N/C ^a	N/C	0.010
D	0.565	0.271	N/C	N/C	0.130
E	N/C	N/C	N/C	N/C	0.217
F	N/C	N/C	N/C	N/C	0.136
G	N/C	N/C	N/C	N/C	0.139
H	0.614	1.805	N/C	N/C	0.127
I	0.672	6.977	N/C	N/C	0.095
J	0.420	3.899	0.810	4.483	0.054
K	0.415	3.265	0.884	3.675	0.070
L	0.495	8.348	0.805	4.414	0.085
M	0.572	14.933	1.207	0.142	0.121
N	0.602	12.039	N/C	N/C	0.098
O	0.427	8.881	1.849	4.989	0.068
P	0.205	4.893	0.469	7.086	0.053

^aNot cracked.

f_c = normal stress in concrete

f_s = normal stress in steel

v_c = shear stress in concrete

CHAPTER 4: ANALYSIS OF LINER PLATE SYSTEM

4.1 METHOD OF ANALYSIS

Most of the loads imposed on the liner plate result from the shortening of the concrete shell relative to the liner plate. The relative strain causes compressive membrane-loads on the plate. The anchors will not be loaded if all the liner plates are perfectly fabricated and are erected so that they are either perfectly flat or have outward curvature. When one panel has an inward curvature, caused by a fabrication or construction imperfection, it will deform inwardly because it has lower in-plane stiffness than the other panels. A panel with inward curvature is illustrated in Figure A.1. The anchor system is then subjected primarily to a shear load, which is largest at the two anchors adjacent to the bent plate and diminishes rapidly away from them. The anchors will also be subjected to radial force, longitudinal force, etc.; these are minor when compared with the shear load.

For the liner system, there are several possible modes of failure.

Examples are:

- a. Excessive strain in the liner.
- b. Shearing failure of anchors in the hoop direction.
- c. Radial pullout of an anchor adjacent to a bent plate with an inward curvature.
- d. Longitudinal buckling of the liner plate.

We considered the possibility of pullout of the anchor. Reference 9 has demonstrated that the concrete and anchorage have a capacity of about 1500 lb/in. against pullout. This capacity arises from the shearing and bonding of the anchor and concrete, which has been shown to be much greater than the pullout force that can be developed adjacent to a bent plate. Therefore, a pullout failure of the anchor is not a concern. This leads to the further conclusion that longitudinal buckling of the liner plate is also highly unlikely, unless an anchor pullout does take place. Evaluation of the liner system can thus be concentrated on the liner strains and the shearing movement of the anchor adjacent to a bent plate.

The following analysis was made for both Extreme Environmental and Abnormal/Extreme Environmental conditions. The Extreme Environmental condition was considered because, as will be seen later, under mechanical loads it produces a more severe anchor load, in comparison with code allowables, than does the Abnormal/Extreme Environmental condition.

4.2 ANALYSIS MODEL

Based on the load combination, the liner system was most critically loaded near the junction of the cylinder and the base slab (Section N, relative el. 9.6 ft). Therefore, the analysis considered a 1-inch-wide strip of the liner system that runs in the hoop direction. One of the panels was given an initial inward curvature corresponding to a radial deflection of $\Delta = 1/8$ in. at the center of the panel. The remaining liner was treated as flat plate. The 2 in. x 3 in. x 1/4 in. angles anchor the plate to the concrete at 15 in. intervals along the hoop direction. The resulting model is shown in Fig. A.2(a) of the Appendix. This model can be further reduced to the spring system illustrated in Fig. A.2(b).

The spring system consists of three types of springs: K_{BP} , K_C , and K_{FP} . The spring K_{BP} represents the in-plane stiffness of the bent plate panel; it is nonlinear in nature and its property was adopted from Ref. 9. These parameters were based on an in-plane compression test on a bent plate having similar material properties. The K_{BP} curve is shown in Fig. A.3. The linear portion of the curve has a slope of 130 kip/in./in.

The stiffness of the anchorage against shear movement is represented by K_C . This is also nonlinear in nature, as shown in Fig. A.4. It was adopted from the tests described in Ref. 9. These tests were performed on 3 x 2 x 1/4 angles embedded in concrete, which had a Young's modulus of 5400 ksi. The linear portion of the K_C curve had a slope of 270 kip/in./in.

The in-plane stiffness of the flat plate is represented by K_{FP} . This is equal to 500 kip/in./in., as computed in the Appendix.

4.3 ANALYSIS AND RESULTS

The procedure that follows is outlined in the Appendix. Strains in concrete on the inside face of the concrete shell were first computed from stresses due to mechanical loads. The total compressive strain in the liner plate was determined by combining the strains on the concrete due to mechanical loads, concrete shrinkage, and the differential strain between the liner and the concrete resulted from thermal loads. This liner strain was converted to the unbalanced membrane force N_h , which was combined with the unbalanced membrane force N_h' (due to pressure acting on the bent plate) and then applied to the anchors adjacent to the assumed bent plate to determine the shear force and movement of the anchor.

STEP 1: Compute Liner Plate Membrane Strains

Concrete strains due to mechanical loads were first computed at Section N (El. 9.6 ft) of the concrete cylinder near the base, where thickening of the concrete section begins. For the location of Section N, refer to the axisymmetric finite-element stress-analysis model of the concrete shell shown in Fig. 3.1. As stated previously, the concrete strain due to initial shrinkage prior to prestressing of the containment wall was assumed to be -100μ . Otherwise, concrete strains due to dead load (D), vertical and horizontal seismic loads (E_v and E_h), pressure load (P_v and P_a), and prestressing load (F) were converted from the forces/moments generated by the finite-element stress-analysis of the concrete shell. Note that the stress results for the prestressing load are adopted from the PDSAR of Palisades Unit 1.

Table 4.1 lists the meridional force, f_z ; meridional moment, M_z ; hoop force, f_h ; and hoop moment, M_h , due to D, E, P, and F. A positive force signifies tension. A positive moment is one which causes a tensile bending-stress on the inside face of the concrete shell. The combined forces and moments are also shown for the Extreme and Abnormal/Extreme conditions. The load combination for the Extreme condition was $D + 0.4E_v + E_h + F + P_v$. For the Abnormal/Extreme condition, the load combination was $D + 0.4E_v + E_h + F + P_a$. To simulate the equivalent effect of the

square root of sum of squares (SSRS) combination between the vertical and horizontal seismic loads, the factor 0.4 was applied to the vertical seismic load.¹⁰ Since $E_v = 0.24g$, upon application of the factor 0.4 the value of 0.4 E_v becomes 0.096g.

Table 4.1. Force and moment at Section H of concrete shell.

	D+0.4E _v ≈ 1.096D	E _H	P _a	P _v	F	<u>Extreme</u> D+E+P _v +F	<u>Abnormal/ Extreme</u> D+E+P _a +F
f _z (kip/ft)	-125.3	-142.7	216.9	-11.8	-293.0	-573.3	-344.6
M _z (kip-ft/ft)	- 14.3	--	131.8	- 7.2	- 50.0	- 71.5	67.5
f _h (kip/ft)	- 5.3	--	123.0	- 6.7	-715.0	-727.0	-597.6
M _h (kip-ft/ft)	2.3	--	24.1	- 1.3	- 20.0	- 18.5	6.9

Normal stresses at an element on the inside face of the concrete shell were then computed:

$$\begin{aligned}
 s_z &= f_z / (42 \times 12) + 6M_z / 42^2 \\
 &= f_z / 504 + M_z / 294 \\
 s_h &= f_h / 504 + M_h / 294
 \end{aligned}$$

The concrete strains were related to the stresses in the following manner:

$$\begin{aligned}
 e_z &= (s_z - \nu_c s_h) / E_c = (s_z - 0.17s_h) \times 10^3 \mu / 5.5 \\
 e_h &= (s_h - 0.17s_z) \times 10^3 \mu / 5.5
 \end{aligned}$$

When the concrete strains computed above were combined with the assumed strain of -100μ , caused by the initial concrete shrinkage, we obtained the total strain for mechanical loads. This total included the relative membrane strain induced in the liner plate by the mechanical loads.

Thermal loads cause additional compressive strain in the liner because they produce larger expansion in the liner than in the concrete. According to the calculation procedures described in the Appendix, the thermal-induced relative strain in the liner was conservatively computed as follows:

Extreme Condition: for an inside temperature of 35°F and

(Winter) . outside temperature of -1°F ,

$$e_z = e_h = -6.5 \mu (85+1)/2 = -280 \mu.$$

Abnormal/Extreme: for a peak accident temperature of 410°F ,

(Winter) $e_z = e_h = -5.5 \mu [410-(35+1)/2] = -2392 \mu.$

Table 4.2 lists the computed relative liner strains due to the mechanical loads, the thermal loads, and the combined effect of both.

STEP 2: Compute the unbalanced force, N .

The unbalanced membrane force is applied to the anchor point at the edge of the bent plate. This force is composed of two parts: N_h and N_h' . N_h is due to the liner strains shown in Table 4.2. N_h' is due to the pressure, P_v or P_a , acting on the bent plate. For a 1/4 in. thick plate, Eq. (A-5) from the Appendix gave

$$N_h = 0.25 \times 30000 (e_h + 0.3e_z)/(1-03^2) = 8242 (e_h + 0.3e_z) \quad (\text{A-5})$$

From Eq. (A-6) of the Appendix,

$$N_h' = P \times 15^2/(2\pi^2 \times 1000 \times 1/8) = 0.0912P \quad (\text{A-6})$$

and

$$N = N_h + N_h'$$

Table 4.3 summarizes the values of N_h , N_h' , and N for both the Extreme and Abnormal/Extreme conditions. Note that N was not computed for the mechanical loads under the Abnormal/Extreme condition because the mechanical loads under the Extreme condition were more critical. This is shown by Table 4.2.

Table 4.2. Relative liner plate membrane strains.

Condition	Loads	s_z	s_h	e_z	e_h
		(kip/in. ²)	(kip/in. ²)		
Extreme Condition	D + E + F + Pv	- 1.38	- 1.50	- 205 μ	- 230 μ
	So	N/A	N/A	- 100 μ	- 100 μ
	<u>Thermal</u>	N/A	N/A	<u>- 280 μ</u>	<u>- 280 μ</u>
	Total I			- 585 μ	- 610 μ
	Total II (mechanical loads)			- 305 μ	- 330 μ
Abnormal/ Extreme Condition	D + E + F + Pa	- 0.45	- 1.17	- 46 μ	- 199 μ
	So	N/A	N/A	- 100 μ	- 100 μ
	<u>Thermal</u>	N/A	N/A	<u>-2392 μ</u>	<u>-2392 μ</u>
	Total I			-2538 μ	-2691 μ
	Total II (mechanical loads)			- 146 μ	- 299 μ

So = shrinkage

Table 4.3. Unbalanced force, N, and equivalent force, $\tilde{N}$.

	<u>Extreme Environment</u>		<u>Abnormal/Extreme Environment</u>
	w/thermal	mech. load	w/thermal
	kip/in.	kip/in.	kip/in.
N_h	-6.48	-3.48	-28.4
N'_h	<u>-0.28</u>	<u>-0.28</u>	<u>5.1</u>
N	-6.76	-3.72	-23.3
$\tilde{N}$	-9.26	-5.15	-31.9

STEP 3: Compute anchor movement and force.

The analysis given in Appendix A demonstrated that the liner system model can be replaced by the 3-spring system shown in Fig. A.5. For this system, according to Eq. (A-14) the equivalent force, $\tilde{N}$, is

$$\tilde{N} = (1 + D)N \quad (A-14)$$

where

$$D = a_1 + a_1 a_2 + \dots \quad (A-12)$$

Using

$$K_C \text{ (linear)} = 270 \text{ kip/in./in.}$$

$$K_{BP} \text{ (linear)} = 130 \text{ kip/in./in.}$$

$$K_{FP} = 500 \text{ kip/in./in.}$$

Eq. (A-12) also gave

$$a_1 = 0.280, \quad K'_2 = 222 \text{ kip/in./in.}$$

$$a_2 = 0.248, \quad K'_3 = 243 \text{ kip/in./in.}$$

$$a_3 = 0.240, \quad K'_4 = 254 \text{ kip/in./in.}$$

$$a_4 = 0.238,$$

Thus,

$$D = 0.280 + 0.069 + 0.017 + 0.004 + \dots = 0.370$$

and

$$\tilde{N} = 1.37N$$

The value of $\tilde{N}$ is also listed in Table 4.3.

To compute the anchor movement, first try a linear solution.

$$\delta \text{ (linear)} = \frac{\tilde{N}}{K_{FP} + K_C(\text{linear}) + K_{BP}(\text{linear})}$$

$$= \tilde{N}/900$$

When the linear solution for the anchor movement exceeds the elastic limit of K_C or K_{BP} , a nonlinear solution becomes necessary. This can be done by trial and error until equilibrium is reached. The results are shown in Table 4.4. For the Extreme Environmental condition the anchor shear force, V , is also computed. V is caused by the mechanical loads.

Table 4.4. Computed results vs. ASME code allowables.

		Extreme Environment		Abnormal/Extreme Environment
		w/thermal	mech. load	w/thermal
Liner	$e_{\max.}$	-610 μ	N/A	-2691 μ
Plate	$e_{\text{allow.}}$	-2000 μ		-5000 μ
Liner	δ	0.0103 in.	N/A	0.0516 in.
Anchor	$\delta_{\text{allow.}}$	0.0350 in. (= $\delta_U/4$)		0.0700 in. (= $\delta_U/2$)
	V	N/A	1.54 kip/in.	N/A
	$V_{\text{allow.}}$		2.22 kip/in. (= $V_U/3$)	

STEP 4: Evaluate the liner plate and anchor.

For the liner plate, the calculated membrane strain was compared with the allowables specified in Division 2 of ASME Boiler & Pressure Vessel Code, Section III, Subsection CC, Article CC-3720. The liner anchor was evaluated against the allowable shear force (under mechanical loads only) and the

displacement specified in Article CC-3730, the subsection of the ASME code given above. Both the allowable anchor force and displacement are specified as a fraction of test-determined ultimate capacity.

The test results for the case of no gap between the liner plate and concrete are tabulated in Figs. 5 through 19 of the FSAR. The minimum ultimate load is shown to be $V_u = 6.67$ kip/in. The ultimate displacement is $\delta_u = 0.14$ in.

The analysis results are compared with the applicable allowables in Table 4.4. All computed results are within the code allowables, and it may be stipulated that the liner system possesses a sufficient margin of capacity under both the Extreme and Abnormal/Extreme Environmental conditions.

APPENDIX

PROCEDURES FOR CALCULATING LINER MEMBRANE STRAIN AND ANCHOR MOVEMENT

A.1 INTRODUCTION

In the Palisades Plant Unit 1 containment building, the liner plate is typically 1/4-inch thick and liner anchors in the cylindrical wall are typically L3x2x1/4 steel angles installed 15 in. apart in the hoop direction (Fig. A.1). For the purpose of analysis, all liner panels except one are assumed to be flat plates. The exception liner panel is assumed to have initial inward curvature. The maximum initial inward deflection at the center of the panel is assumed to be 1/8 in. (Ref. 4). The physical model thus described is illustrated in Fig. A.2(a), which represents a 1-inch-wide strip of the liner system. Analysis results based on this one-way physical model will be conservative because the benefit of the bi-axial stiffening of the plate is not taken into account. The corresponding analysis model may be represented by the spring system illustrated in Fig. A.2(b). The spring properties and the analysis method are based on Ref. 9, with some minor modifications to the analysis procedure. The stiffness properties of the anchor and the concrete spring, K_C , and of the bent plate spring, K_{BP} , were established from test data.⁹ These stiffness properties are applicable to the present study because the materials and configurations of the bent plate and liner anchor test models are similar to those used in the construction of the Palisades containment liner system.

Fig. A.1. Circumferential section of the cylinder liner with one bent panel.

Fig. A.2. Analysis model of the liner system at the cylinder base.

A.2 ANALYSIS MODEL

As shown in Fig. A.2(b), the analysis model is composed of three types of springs. K_C represents the shear resistance of the liner anchor in the concrete, K_{bp} represents the in-plane stiffness of the bent plate, and K_{fp} represents the in-plane stiffness of a flat plate panel. The assumed initial inward curvature of the bent plate results in an in-plane unbalanced force, W , at anchor point No. 1: this results from the differential strains between the liner and concrete and to the pressure acting on the bent plate. This force generates tangential movements of all the anchors toward the bent plate panel. The anchor movement will maximize at anchor point No. 1 and diminish rapidly as the distance from anchor point No. 1 increases.

The spring stiffness properties are described below:

- (a) K_{FP} , The in-plane stiffness of a flat plate is

$$K_{FP} = AE_s/L \quad (A-1)$$

A = section area of the 1-inch wide liner plate strip (0.25 in. x 1 in.)

E_s = Young's modulus of liner (30,000 ksi)

L = hoop direction spacing of anchors (15 in. typical)

Hence,

$$K_{FP} = 500 \text{ kip/in./in.} \quad (A-2)$$

- (b) K_C , The tangential shear resistance capacity of the L3x2x1/4 angle embedded in concrete was established from tests.⁹ The idealized K_C , corresponding to a concrete having $E_c = 5.4 \times 10^3$ ksi, is reproduced in Fig. A.3.

- (c) K_{bp} , The in-plane stiffness of the bent plate was also adopted from Ref. 9. Figure A.4 illustrates the idealized K_{bp} corresponding to liner material having a minimum yield stress of 32 ksi.

Fig. A.3. Load vs. displacement curve, K_{3p} , for the bent plate.

Fig. A.4. Load-displacement curve, K_C , for the liner anchor.

A.3 LINER STRAINS AND UNBALANCED FORCE

The force, N , to be applied at anchor point No. 1 is composed of two parts. The first is N_h . This is due to the differential strain between the liner and concrete that arises from the applicable mechanical and thermal loads. The second part is N_h' , which is due to pressure directly acting on the bent plate. The methods used to compute N_h and N_h' are discussed below.

(a) N_h Maximum concrete stresses or strains due to dead load and seismic load on the containment wall occurs near the cylinder-to-base junction. First, therefore, compute the meridional and hoop concrete strains near the base junction for the following loads: dead load, seismic load, effective prestress load, and pressure load. Strain due to initial concrete shrinkage was assumed to be -100μ ($\mu = 10^{-6}$ in./in.).

For the mechanical loads, the concrete strains also represented the differential strains between liner and concrete that were imposed upon the liner by way of the anchors. For thermal loads, the differential strains imposed on the liner were conservatively calculated as follows:

$$\begin{aligned}\text{Extreme Environment: } e_h = e_z &= 6.5 \mu [T_i - (T_i + T_o) / 2] \\ &= 3.5 \mu (T_i - T_o) / 2\end{aligned}\quad (\text{A-3})$$

$$\text{Abnormal/Extreme: } e_h = e_z = 6.5 \mu [T_a - (T_i + T_o) / 2] \quad (\text{A-4})$$

e_h = differential liner-concrete strain in hoop direction.

e_z = differential liner-concrete strain in meridional direction.

T_i = ambient temperature inside containment.

T_o = temperature outside containment.

T_a = peak temperature on liner surface for abnormal condition.

The above expressions are conservative. They are based on the following simplifications:

- Thermal expansion coefficients for both liner and concrete were 6.5×10^{-6} in./in./ $^{\circ}\text{F}$.
- The concrete did not crack and the concrete wall was restrained from rotation.

- Under abnormal conditions, the liner was assumed to be instantaneously heated to T_a , while the temperature gradient in the concrete wall still remained $(T_i - T_o)$.

The membrane force N_h can then be determined:

$$N_h = \frac{tE_s(e_h + ve_z)}{1-\nu^2} \quad (A-5)$$

where t is the thickness, E_s is Young's modulus, and ν is Poisson's ratio of the liner plate.

(b) N_h The membrane reaction-force at both edges of the bent plate, when subjected to a normal pressure acting directly on the plate, may be approximately computed as follows.⁹

$$N_h' = PL^2/2\pi^2\Delta_m \quad (A-6)$$

in which

P = pressure

L = plate span (15 in.)

Δ_m = initial inward deflection at center of the bent plate (1/8 in.)

A.4 ANCHOR MOVEMENT

To derive the anchor movement of the first anchor when the analysis model is subjected to the unbalanced membrane force of $N = N_h + N_h'$, K_{BP} and all K_C were first assumed to be linear. Letting K_1 represent the effect of all flat plates and anchors other than anchor No. 1, the analysis model became that shown at the top of Fig. A.2(c). From this model,

$$\begin{aligned} \delta_1 &= \frac{N}{K_{BP} + K_C + K_1'} \\ &= \frac{N}{K_{BP} + K_C + K_{FP}} \left(\frac{K_{BP} + K_C + K_{FP}}{K_{BP} + K_C + K_1'} \right) \\ &= \frac{N}{K_{BP} + K_C + K_{FP}} (1 + D) \end{aligned} \quad (A-7)$$

where

$$D = \frac{K_{FP} - K_1'}{K_{BP} + K_C + K_{FP}} .$$

Because K_1' is related to K_C , K_{FP} and a spring, K_2' , as shown in Fig. A.2(c), i.e.,

$$K_1' = \frac{1}{\frac{1}{K_{FP}} + \frac{1}{K_C + K_2'}} = \frac{K_{FP}(K_C + K_2')}{K_{FP} + K_C + K_2'} \quad (A-8)$$

we have

$$D = \frac{K_{FP}^2}{K_{FP}K_C + (K_{BP} + K_C)(K_{FP} + K_C) + K_2'(K_{FP} + K_C + K_{BP})} \quad (A-9)$$

Similarly, according to Fig. A.2(c), K_2' is related to K_{FP} , K_C and a certain K_3 as in Eq. (A-8), with K_2 and K_3 replacing K_1 and K_2 , respectively. It can then be shown that

$$D = \frac{K_{FP}^2}{(K_C + 2K_{FP})(K_C + K_{BP} + K_{FP}) - K_{FP}^2} \times \left[1 + \frac{K_{FP}^2}{K_{FP}K_C + (K_{BP} + K_C)(K_{FP} + K_C) + K_3'(K_{FP} + K_{BP} + K_C)} \right] \quad (A-10)$$

where

$$K_3' = \frac{K_{FP}(K_C + K_{BP})}{K_{FP} + K_C + K_{BP}} . \quad (A-11)$$

Based on Eq. (A-10) and Eq. (A-9), a recursive relationship can be established

$$D = a_1(1 + a_2(1 + a_3(1 + \dots))) = \sum_{i=1}^{\infty} a_1 a_2 \dots a_i$$

$$a_i = \frac{K_{FP}^2}{(K_C + 2K_{FP})(K_{FP} + K_C + K_i') - K_{FP}^2}$$

$$K_1' = K_{BP}$$

(A-12)

$$K_i' = \frac{K_{FP}(K_C + K_{i-1}')}{K_{FP} + K_C + K_{i-1}'} \quad (i = 2, 3, \dots)$$

Equation (A.7) now becomes

$$\delta_1 = \frac{\tilde{N}}{K_{FP} + K_C + K_{BP}} \quad (A-13)$$

where

$$\tilde{N} = N(1 + D). \quad (A-14)$$

The problem is thus reduced to analyzing the equivalent 3-spring system (shown in Fig. A.5) when subjected to the equivalent force $\tilde{N}$. The actual nonlinearity in K_C and K_{BP} can now be taken into account, depending on the magnitude of $\tilde{N}$.

It is advisable to first try a linear solution for δ_1 . If the resultant value of δ_1 exceeds the elastic limit of K_{BP} or K_C or both, a nonlinear solution becomes necessary. This can be accomplished by trial and error until a force equilibrium is reached in the solution.

Fig. A.5. The 3-spring equivalent analysis model.

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