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EVALUATION OF MIDLAND NUCLEAR POWER PLANT BORATED WATER STORAGE TANK FOR NON-UNIFORM SUPPORT LOADING RESULTING FROM RING WALL SETTLEMENT

prepared for

CONSUMERS POWER COMPANY
Jackson, Michigan

March, 1982

STRUCTURAL MECHANICS ASSOCIATES
A Calif. Corp.

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Note: New features, as well as changes, deletions, and addition to information within this report, are indicated by bars in the margins or by a bar under the page number if the entire page is affected.

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1. INTRODUCTION

1.1 STATEMENT OF PROBLEM

Soil settlement at the site of the Midlands Nuclear Power Plant has resulted in deformation of the ring walls that serve as a supporting base for the Units 1 and 2 Borated Water Storage Tanks (BWSTs). Survey measurements of the ring walls indicate that the top surfaces have distorted from their original position. Visual examination has also indicated that some cracking of the ring walls has occurred. The ring wall deformation has resulted in a non-uniform support condition for the BWSTs. Examination of the tanks indicated that some of the anchor bolts connecting the BWSTs to the ring walls were unloaded and some were loaded. During the initial installation, all bolts were lightly loaded.

Concerns have been raised that uneven support of the BWSTs may have resulted in yielding of the tank walls or that increased anchor bolt loading could have yielded the bolt chairs. Bechtel Corp. has designed a retrofit for the ring walls which would stiffen the walls and prevent further distortion. Shims would then be installed between the ring walls and tank bottoms to provide uniform support for the tanks. Prior to initiating the ring walls retrofits, the condition of the BWSTs must be assessed. Evaluation of the BWSTs in their current condition is the subject of the main body of this report.

Subsequent to the release of the main body of this report, an additional BWST study was undertaken. This additional study consisted of an evaluation of the tanks when subjected to projected end-of-life soil settlement conditions combined with Seismic Margin Earthquake loads. The description and results of this additional study have been integrated into this report as an Addenda.

1.2 DESCRIPTION OF BWSTs AND RING WALLS

There are two BWSTs in the Midland Nuclear Power Plant complex, one each for Units 1 and 2. The two tanks are indentical and are cylindrical flat bottom storage tanks with umbrella-shaped roofs. They are 52 feet in diameter and 32 feet in height. The roofs are welded to ring girders at the top of the tanks. The tank walls are 0.375 inches thick for a height of 8 feet from the flat bottom and are 0.25 inches thick for the remaining 24 feet. The bottom is 0.25 inches thick. All materials are type 304 L stainless steel.

The two tanks are located outdoors in the tank farm area, north of the Auxiliary Building. The Unit 1 tank, 1T-60, is located on the west side of the tank farm and the Unit 2 tank, 2T-60, is located on the east side of the tank farm. Tank details are shown on Graver drawings NL12046, Rev. 3, NL-12047, Rev. 2, and NL-12051, Rev. 2.

Ring walls for the two tanks are identical except in the valve pit area. Unit 1 has a larger valve pit than Unit 2. The ring walls are detailed on Bechtel Drawings C-127 (Q), Rev. 6, and C-128 (Q), Rev. 7.

Figures 1-1 and 1-2 show a plan view of the two ring walls with anchor bolt locations identified. Prior to erection of the tanks, elevations at the top of each ring wall were verified by Graver, the tank fabricator. Table 1-1, taken from Reference 1, tabulates the measured elevations relative to a bench mark. Design elevations according to the Bechtel construction drawings were 635.04 feet for 1T-60 and 635.12 feet for 2T-60. After erection of the tanks and filling with water, soil settlement has occurred resulting in distortion of the top surfaces of the ring walls. Table 1-2, taken from Reference 2, depicts elevations measured at the top of each ring wall on 16 June 1981. Note that Table 1-2 depicts actual elevations wherein Table 1-1 provides elevations relative to a bench mark. From Tables 1-1 and 1-2 it is evident that the deviation from a plane surface is greatest on 1T-60. Maximum deviation from a plane surface is about 1.2 inches for 1T-60 compared to about 0.36 inches for 2T-60. Figure 1-3 shows relative deflections of the ring wall top surface from the initial top surface contour for 1T-60. Note that the deflections are relative since the ring walls have settled to a lower elevation since initial contruction i.e., the surveyed contours have been adjusted in elevation so that at a common point, the elevations are identical. Relative deflections for 2T-60 are considerable less and are not the governing case.

During initial erection of the tanks, the anchor bolts were tightened snugly. Torque was not specified. Settlement of the ring walls has resulted in distortion of the ring wall top surfaces which support the BWSTs. As a result, several of the anchor bolts were observed to have gaps between the bolt chairs and nuts while several appeared to be loaded. In order to determine actual bolt loads, strain gages were applied by another contractor to the loaded anchor bolts and the nuts were backed off to zero load. Tables 1-3 and 1-4 from Reference 3 indicate acutal bolt loads prior to backing off the nuts. As would be deduced from the measured elevations of the ring walls supporting both tanks, the bolt loads are much higher in Tank 1T-60. Three of the bolt loads in Tank IT-60 exceed the faulted condition design load of 20.43 kips from Reference 7. All other loads are within the original design load. The tank walls are very stiff with respect to remaining in a horizontal plane along the bottom surface and at points where the ring walls have settled out of the horizontal plane, the anchor bolts are loaded, trying to close up the gap between the tank walls and ring walls. At points where the tanks are resting on the ring walls, bolt loads are zero.

There is a 1/2 inch thick asphalt impregnated fiberboard (Celotex) between the tanks bottoms and the ring walls. The material is compressible and tends to distribute the tank wall loading to the ring wall in a more uniform manner than if there were no compressible material at the interface.

The tanks are currently full of water and are resting on the ring walls with the anchor bolts all unloaded. The current unloaded anchor bolt condition is much less critical to the tanks than the prior condition with some of the bolts loaded, trying to force the tanks walls to the contours of the ring wall surfaces.

Visual observations of the BWSTs with the bolt loads still applied did not reveal any obvious damage. Distortion from welding was apparent in the bolt chairs and there was no apparent difference between chairs that were loaded vs. chairs that were not.

1.3 PURPOSE OF STUDY

The purpose of this study is to evaluate the present or worst condition for each of the tanks in order to determine if any yielding, buckling or permanent damage has occurred due to soil settlement and ring wall distortion. The objective is to verify the BWSTs structual integrity prior to retrofiting the ring walls.

1.4 SCOPE OF WORK

The scope of work consists of performing a finite element analysis of the worst tank condition to determine stress conditions in the tank walls and bolt chairs caused by soil settlement and consequent ring wall distortion. Calculated stresses are to be compared to code based acceptance criteria or material yield strength to determine if permanent deformation has occurred and to cylinder buckling criteria to determine if elastic or plastic buckling has occurred. In the event that yielding or buckling is predicted to occur, an assessment is to be made as to the possible detrimental effects that may result due to the anticipated level of inelastic strain.

1.5 GENERAL APPROACH

A three-dimensional finite element model was constructed to represent the BWSTs cylindrical walls. The model was constructed of flat plate elements possessing both bending and membrane stiffness. Vertical boundary elements are utilized at the bottom surface to represent the nonlinear behavior of the asphalt impregnated fiberboard between the tank and ring wall. The boundary elements have gap capability so that non-uniform support conditions, including gaps, can be properly represented. Actual load-deflection relationships were determined by conducting material compression tests on samples of asphalt impregnated fiberboard used in the tank installation. Additional horizontal boundary elements at the tank wall lower edge represent the stiffness and restraint offered by the flat tank bottom. The upper edge of the cylindrical tank is stiffened by a ring girder with properties chosen to represent the stiffness of the tank roof and the restraint that it offers to the tank at the roof/cylinder intersection.

Loading conditions on the tank cylindrical shell, which are reacted by the ring wall, include dead weight of the tank roof, weight of the cylindrical shell and weight of an effective annulus of water plus measured anchor bolt loads. Other loading imposed on the model is the hydrostatic radial pressure acting on the tank wall.

Chapter 2 summarizes the overall results and conclusions of this study. Chapter 3 presents acceptance criteria used for evaluating settlement-induced tank stresses. Chapter 4 describes the tank model, boundary conditions, loading conditions and method of solution in detail. Detailed results are presented in Chapter 5. Only the worst case condition representing 1T-60 was analyzed since the worst case clearly did not result in any predicted excessive loading or permanent deformations in the tank walls, bolts or bolt chairs.

TABLE 1-1

PRE-ERECTION ELEVATION DATA FOR THE BORATED WATER STORAGE TANKS (1T60 & 2T60) FOUNDATIONS

BOLT NUMBER	ANGLE Θ (CLOCKWISE FROM NORTH)	ELEVATION (FEET) TANK 1T-60	ELEVATION (FEET) TANK 2T-60
1	355.5°	4.48	5.09
2	4.5°	4.46	5.07
3	13.5°	4.47	5.09
4	22.5°	4.47	5.09
5	31.5°	4.48	5.08
6	40.5°	4.48	5.07
7	49.5°	4.48	5.10
8	58.5°	4.47	5.09
9	67.5°	4.48	5.07
10	76.5°	4.47	5.07
11	85.5°	4.48	5.09
12	94.5°	4.47	5.08
13	103.5°	4.48	5.08
14	112.5°	4.48	5.07
15	121.5°	4.48	5.10
16	130.5°	4.48	5.08
17	139.5°	4.48	5.09
18	148.5°	4.48	5.08
19	157.5°	4.47	5.09
20	166.5°	4.48	5.08
21	175.5°	4.48	5.07
22	184.5°	4.48	5.08
23	193.5°	4.48	5.09
24	202.5°	4.47	5.08
25	211.5°	4.48	5.08
26	220.5°	4.47	5.09
27	229.5°	4.49	5.09
28	238.5°	4.48	5.08
29	247.5°	4.47	5.07
30	256.5°	4.47	5.08
31	265.5°	4.49	5.09
32	274.5°	4.47	5.08
33	283.5°	4.47	5.08
34	292.5°	4.46	5.07
35	301.5°	4.48	5.10
36	310.5°	4.47	5.08
37	319.5°	4.48	5.09
38	328.5°	4.48	5.08
39	337.5°	4.47	5.09
40	346.5°	4.47	5.08

TABLE 1-2

RING WALL ELEVATIONS TAKEN ON JUNE 15, 1981

ANGLE Θ (CLOCKWISE FROM DUE NORTH)	ELEVATION (FEET) TANK 1T-60	ELEVATION (FEET) TANK 2T-60
0°	634.86	634.94
30°	634.87	634.93
60°	634.86	634.93
90°	634.86	634.93
120°	634.87	634.94
150°	634.85	634.96
180°	634.83	634.96
210°	634.78	634.95
240°	634.77	634.96
270°	634.79	634.96
300°	634.82	634.96
330°	634.86	634.95

TABLE 1-3

MEASURED LOADS IN BOLTS ANCHORING TANK IT-60

BOLT NUMBER*	LOAD (KIPS)	BOLT NUMBER*	LOAD (KIPS)
1	0.0	21	0.0
2	0.0	22	0.0
3	0.0	23	0.0
4	0.0	24	0.0
5	0.0	25	0.0
6	0.0	26	16.44
7	17.83	27	31.31
8	14.02	28	16.10
9	21.32	29	10.13
10	22.51	30	0.02
11	16.46	31	2.32
12	0.0	32	0.0
13	0.0	33	0.0
14	0.0	34	0.0
15	0.0	35	0.0
16	0.0	36	0.0
17	0.0	37	0.0
18	0.0	38	0.0
19	0.0	39	0.0
20	0.0	40	0.0

*BOLT LOCATIONS CORRESPONDING WITH THESE BOLT NUMBERS ARE LISTED IN TABLE 1-1

TABLE 1-4

MEASURED LOADS IN BOLTS ANCHORING TANK 2T-60

BOLT NUMBER*	LOAD (KIPS)	BOLT NUMBER*	LOAD (KIPS)
1	0.0	21	0.0
2	0.0	22	0.0
3	0.0	23	0.0
4	0.0	24	0.0
5	0.0	25	0.0
6	1.19	26	0.0
7	2.82	27	0.0
8	0.24	28	0.0
9	1.15	29	0.0
10	0.0	30	0.0
11	0.0	31	0.0
12	0.16	32	0.0
13	0.0	33	0.0
14	0.0	34	0.0
15	0.0	35	0.0
16	0.0	36	0.0
17	0.0	37	0.0
18	0.0	38	0.0
19	0.0	39	0.0
20	0.0	40	0.0

*BOLT LOCATIONS CORRESPONDING WITH THESE BOLT NUMBERS ARE LISTED IN TABLE 1-1.

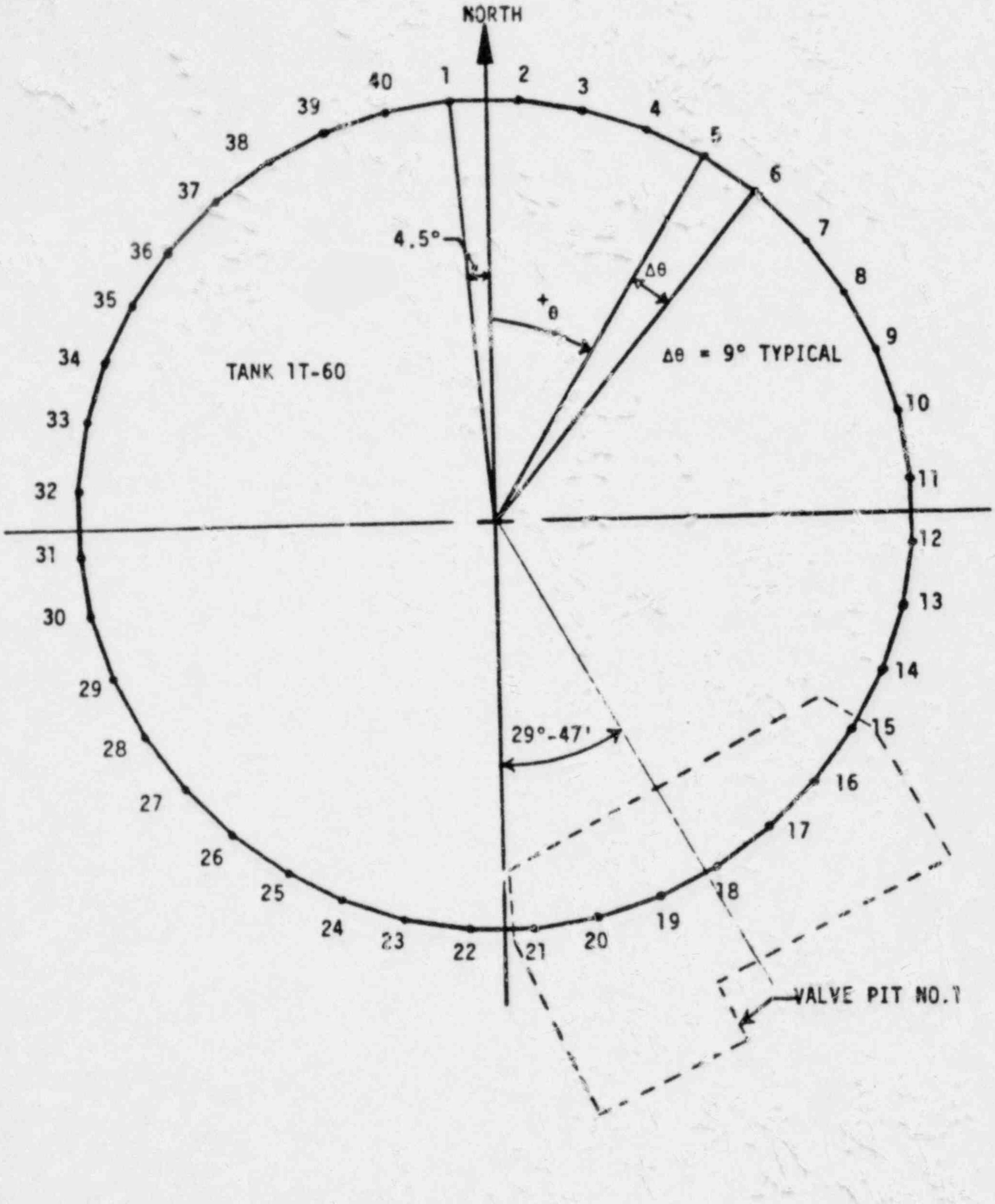


FIGURE 1-1: PLAN VIEW OF TANK 1T-60 IDENTIFYING BOLT NUMBERS AND LOCATION ANGLE θ

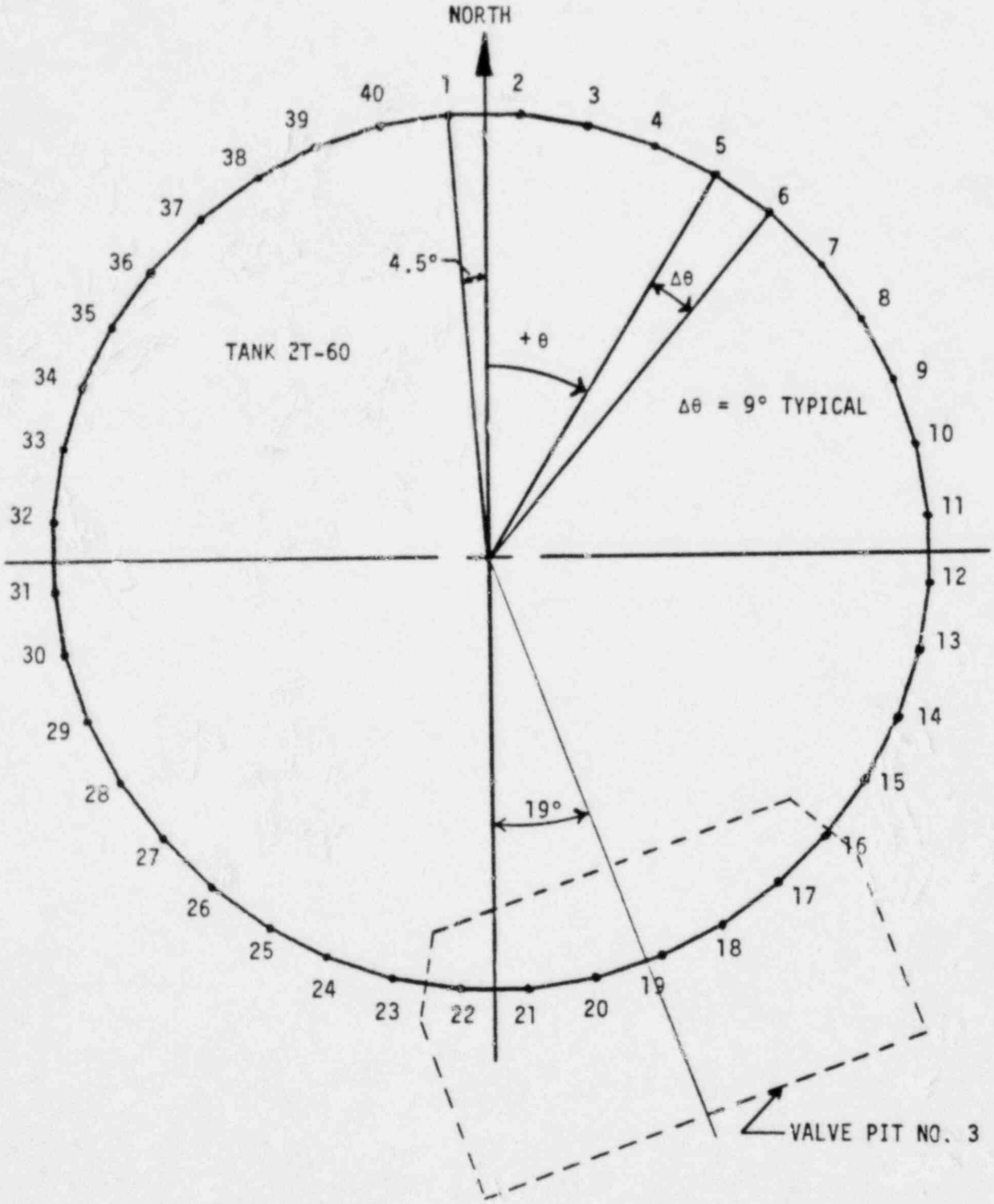


FIGURE 1-2: PLAN VIEW OF TANK 2T-60 IDENTIFYING BOLT NUMBERS AND LOCATION ANGLE θ

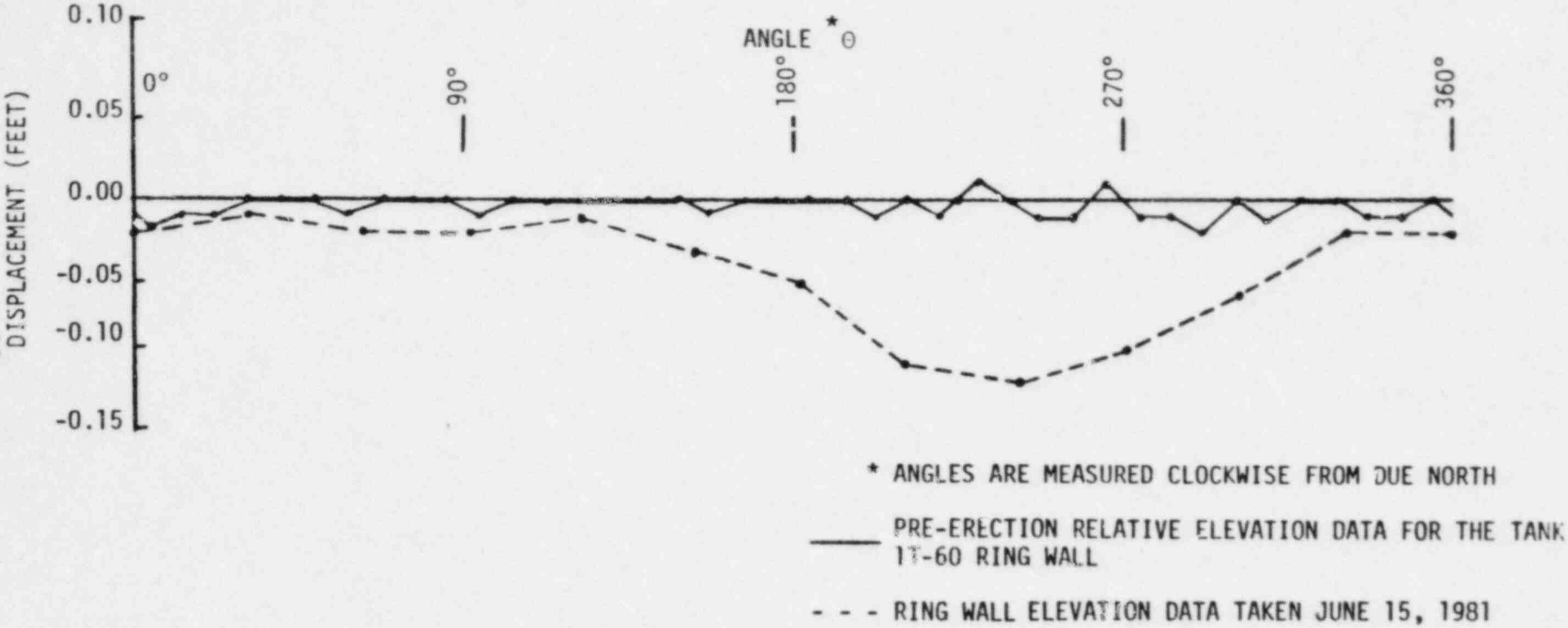


FIGURE 1-3. COMPARISON OF TANK 1T-60 RING WALL RELATIVE ELEVATIONS BEFORE AND AFTER THE GROUND SETTLEMENT

2. SUMMARY AND CONCLUSIONS

2.1 SUMMARY

The most critical case was that of tank 1T-60 where the ring wall top surface supporting the tank had the greatest deviation from a planar surface, see Figure 1-3. Since results of the evaluation of BWST IT-60 were positive, only the one tank was analyzed. Figure 2-1 is a plan view of the BWST finite element model and Figure 2-2 is an elevation view of the model. The BWST was modeled on the ANSYS computer program using 40 flat shell elements around the circumference and 8 elements along the vertical, resulting in elements that are approximately four feet square. The beam type elements shown in Figures 2-1 and 2-2 are boundary elements to represent the restraint of the cylindrical shell afforded by the tank roof and the tank bottom. The gap-type elements shown in Figure 2-2 represent the nonlinear compressibility afforded by the asphalt impregnated fiberboard at the interface between the tank bottom and ring wall.

Figure 2-3 shows the resultant displacement of the tank bottom relative to the uncompressed fiberboard positions, ie, the 1/2 inch thick fiberboard resting on top of the distorted ring wall surface. The displacement plot incorporates the nonlinear deflection of the fiberboard and the linear deflection of the tank wall. Maximum compression in the fiberboard is 0.19 inches.

Figure 2-4 plots the compressive loads at the node points along the tank wall lower end. Zero load indicates a gap between the ring wall and the tank bottom plus fiberboard thickness. Note that a gap exists from about 31 degrees to 103 degrees and from about 193 degrees to 283 degrees measured clockwise from north. The tank is being supported over about 198 degrees and being forced downward by anchor bolt loads in the regions of the gaps.

Maximum primary membrane stress intensity occurs in element number 392 which is in the first row of elements above the shell thickness change from 0.375 inches to 0.25 inches. Maximum stress intensity is calculated to be 12495 psi. Stress intensity is defined as two times the maximum shear stress in accordance with the applicable ASME code, Reference 5. Stress intensity is the appropriate value to compare to code allowables or yield strength. The tanks are constructed of SA 240, Type 304 L stainless steel and the allowable stress intensity for design and normal operating conditions is 15,700 psi. The minimum specified yield strength is 25,000 psi; thus the maximum stress intensity is only 0.5 of yield. The components of stress that make up the maximum stress intensity are 10,571 psi hoop stress from hydrostatic pressure, -1,923 psi axial stress from deadweight and support reactions and -85 psi shear stress due to the nonlinearity in support reactions. The most significant contributor to stress intensity is the hoop component which results from hydrostatic pressure. Stresses are summarized in Tables 5-1 and 5-2.

Now that the anchor bolts have been unloaded, the compressive stress component is reduced and the stress intensity is reduced.

Checks were made for buckling using NASA developed buckling formulas, Reference 6, for axially and moment-loaded thin cylinders. The NASA formulas predict lower bound values of buckling stress considering experimental data as well as theory. Calculated critical buckling stresses for cylinders stressed uniformly along the axis and around the circumference are 5050 psi axial compression in the 0.375 inch thick wall and 2690 psi axial compression in the 0.25 inch thick wall. In the situation at hand, axial stresses are not uniform around the circumference nor along the length. Each of these factors would increase the critical buckling stress. From Reference 6, critical axial stresses for buckling of a cylinder in bending would be 7950 and 4750 psi for the 0.375 and 0.25 inch thick walls, respectively. These are considered to be more representative values for the BWST condition where axial stress varies from positive to negative around the circumference and is not uniform along the length.

Maximum compressive stress occurs in element 392, a 0.25 inch thick element, and is -1930 at the bottom of the element. At this point, the thickness changes to 0.375 inches for elements below. This is less than the calculated buckling stress for uniform axial compression and much less than the predicted buckling stress level for bending. The buckling factor of safety, based on elastic buckling for a shell in bending, is 2.46. In the current condition, the bolt loads have been relieved and compressive stresses have been reduced such that there is no immediate danger of buckling in the event that further settlement and ring wall distortion occur prior to retrofit of the ring walls.

Maximum anchor bolt loading measured in the field via strain gaging was 31,313 pounds, Tables 1-3 and 1-4. Review of the tank design report, Reference 7 reveals that maximum service loading is predicted for the safe shutdown earthquake event and is 20,427 pounds. Three anchor bolt loads (Table 1-3) exceeded this value. The bolts are fabricated from ASTM A-36 steel with a minimum specified yield strength of 36 ksi. The anchor bolts are 1 1/2 inches nominal diameter. Maximum anchor bolt stress is 22.32 ksi in the threaded area. The factor of safety on bolt yield is 1.61. It is therefore concluded, that no permanent deformation or damage has occurred in the anchor bolts. Anchor bolt pullout was checked and the capacity was calculted to be greater than 136 kips. The factor of safety on pullout is 4.31. Note that the bolts are currently unloaded. When the ring walls are retrofit, the tanks are shimmed and bolts are retightened, the loading conditions cited above will not exist.

Original bolt chair design analysis was conducted by methods contained in Reference 8. These methods are very conservative design methods and use beam theory as opposed to plate theory. Maximum design stress occurs in the top plate of the bolt chair. Scaling the calculations in the design report for the maximum measured bolt load of 31,313 pounds and the actual thickness of the top plate, the elastically calculated stress is 47.78 ksi and exceeds the minimum specified yield stress of 25 ksi. This is a bending stress and a plastic hinge will form when yield is exceeded by 50%. Actually for the case of a strain hardening material-like stainless steel, the effective plastic hinge will not occur until the elastically calculated load controlled stress is greater than 150% of yield. In addition, stainless steel yield strength is typically much greater than minimum specified yield.

A yield line (limit) analysis was conducted to determine the limit load for a fully plastic condition to occur. The calculated yield line analysis limit load based on minimum yield properties is 39,950 pounds. None of the measured bolt loads exceed this value. The minimum factor of safety for the maximum measured bolt load is 1.28. The calculated anchor bolt load at first yield of the top plate, using the design formula of Reference 8, is 16.38 kips. There are four bolts that exceeded this value by more than a few percent and three bolts that are essentially at this value. In Chapter 3, an acceptance criteria is developed that would allow settlement induced stresses to be comparable to stresses normally encountered in hydrostatic tests of pressure vessels without further consideration of inspection. For the case at hand, the stress in the top plate of the bolt chair is classified as primary bending and in accordance with the acceptance criteria of Chapter 3, the allowable stress could approach 1.35 S_y when calculated on an elastic basis. The allowable bolt load based on this criterion would be 1.35 (16.38) = 22.11 kips. Only bolts 10 and 27 exceed this value, bolt number 10 exceeding the value by only 1.8%. The only concern should then be addressed to bolt location 27 where the elastic analysis acceptance criterion is exceeded by 41.6%. In this case, limit analysis acceptance criteria are applied.

The yield line (lower bound limit analysis capacity) was determined to be 39.95 kips. As discussed in Chapter 3, the limit anlaysis acceptance criterion was derived from ASME code philosophy and is 0.8 of the lower bound limit analysis capacity. The maximum measured bolt load of 31,313 pounds is 0.78 of the calculated limit load capacity and the bolt chair is considered acceptable as is.

Since some yielding could have occured in the bolt chair at bolt location 27, it is suggested that a dye penetrant examination be condu..ed of welds that attach the top plate to the gussets and tank wall to assure that plastic straining has not opened up any surface cracks. If cracks are not found and/or repaired, the bolt chairs should be considered acceptable.

Local stresses in the tank wall due to bolt chair reactions were evaluated and found to be acceptable. The original design analysis, Reference 7, utilized a method contained in Reference 8 to compute local membrane plus bending stresses in the tank wall resulting from anchor bolt loading on the bolt chairs. Using the design formula in Reference 7, the combined membrane plus bending stress in the tank wall was computed to be 1.93 times the minimum specified yield strength for the maximum measured bolt loading. The governing ASME code design criteria, Reference 5, does not limit secondary stresses. Since local shell bending stresses are considered secondary by the ASME Code, only local membrane stresses are of consequence for evaluating whether any gross yielding has occurred. Reference 8 does not provide a method for calculating local membrane stresses only; thus, the methods of Reference 9 were utilized. The top plate of the bolt chair was conservatively assumed to transmit an outward radial load from the shell equal to the applied bolt chair moment divided by the height of the bolt chair (Figure 4-3). This is a conservative assumption since contribution from the bolt chair gussets are ignored in the analysis. The resulting local membrane stresses are 13.2 ksi in the hoop direction and 12.16 ksi in the axial direction. Both stresses are tensile. Other stresses resulting from hydrostatic pressure and downward bolt chair loading are 6384 psi hoop tension, 1614 psi axial tension and 159 psi shear stress. Combining all the stress components, the maximum stress intensity is 19.59 ksi. This is below the 25 ksi minimum specified yield strength of the material and significantly below the 33.75 ksi acceptance criterion of Chapter 3 for primary local membrane stress intensity.

Secondary stresses are not limited by the governing code, Reference 5. Primary plus secondary stresses should be limited to 2 S_y for cyclic load applications to demonstrate shake down. In the case at hand, the loading is a single applied load and primary plus secondary stresses calculated by the design formula of Reference 8, when combined with stresses induced by other loading are 1.93 S_y; thus, shell stresses are within acceptable limits.

The following summarizes the most severe stress condition in tank 1T-60 prior to relieving the anchor bolt loads.

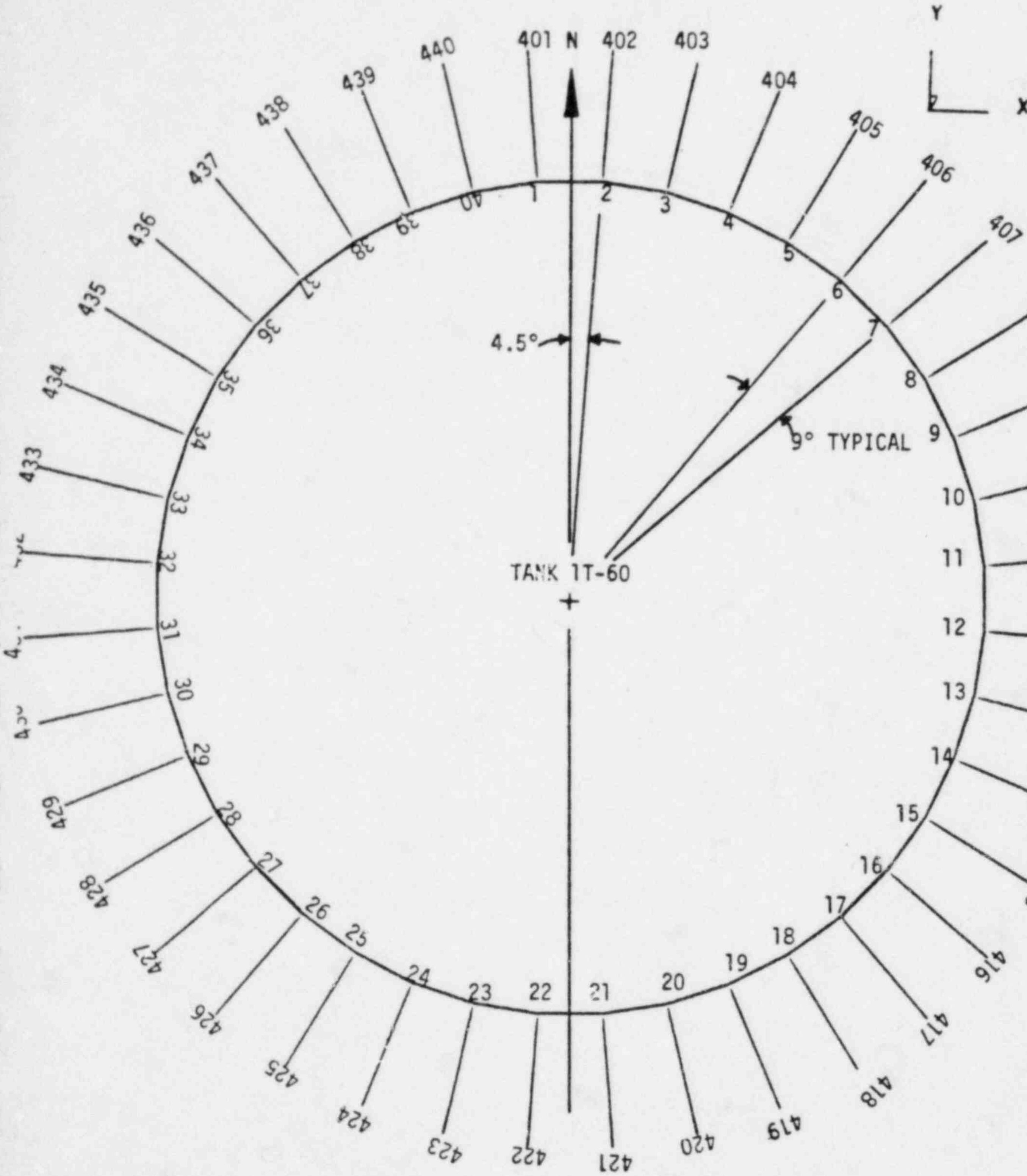
a. Maximum primary membrane stress intensity occurs in the 0.25 inch thick shell about 10 feet from the base and is 12.5 ksi compared to a yield strength of 25 ksi and an ASME code primary membrane stress allowable of 15.7 ksi for design conditions. The factor of safety on yield is 2.0.

b. Maximum membrane compressive stress occurs 8 feet above the base in the 0.25 inch thick shell and is 1930 psi compared to a lower bound elastic buckling stress of approximately 4750 psi. The resulting factor of safety on elastic buckling is 2.46.

c. One bolt chair at bolt location 27 may have yielded to a small degree. The factor of safety on collapse, based on a lower bound limit analysis, is 1.28. This meets the acceptance criterion derived in Chapter 3. For this one bolt location, it is recommended that a dye penetrant inspection be conducted for welds attaching the bolt chair top plate to the gussets and tank wall to ensure that localized inelastic strains have not resulted in cracking at the weld joints.

d. Local membrane stress intensity in the tank wall at the maximum loaded bolt chair is 19.59 ksi. The factor of safety on ASME code minimum specified yield is 1.28. Primary plus secondary stress intensity is 1.93 S_y which is less than the shakedown acceptance criterion of two times yield.

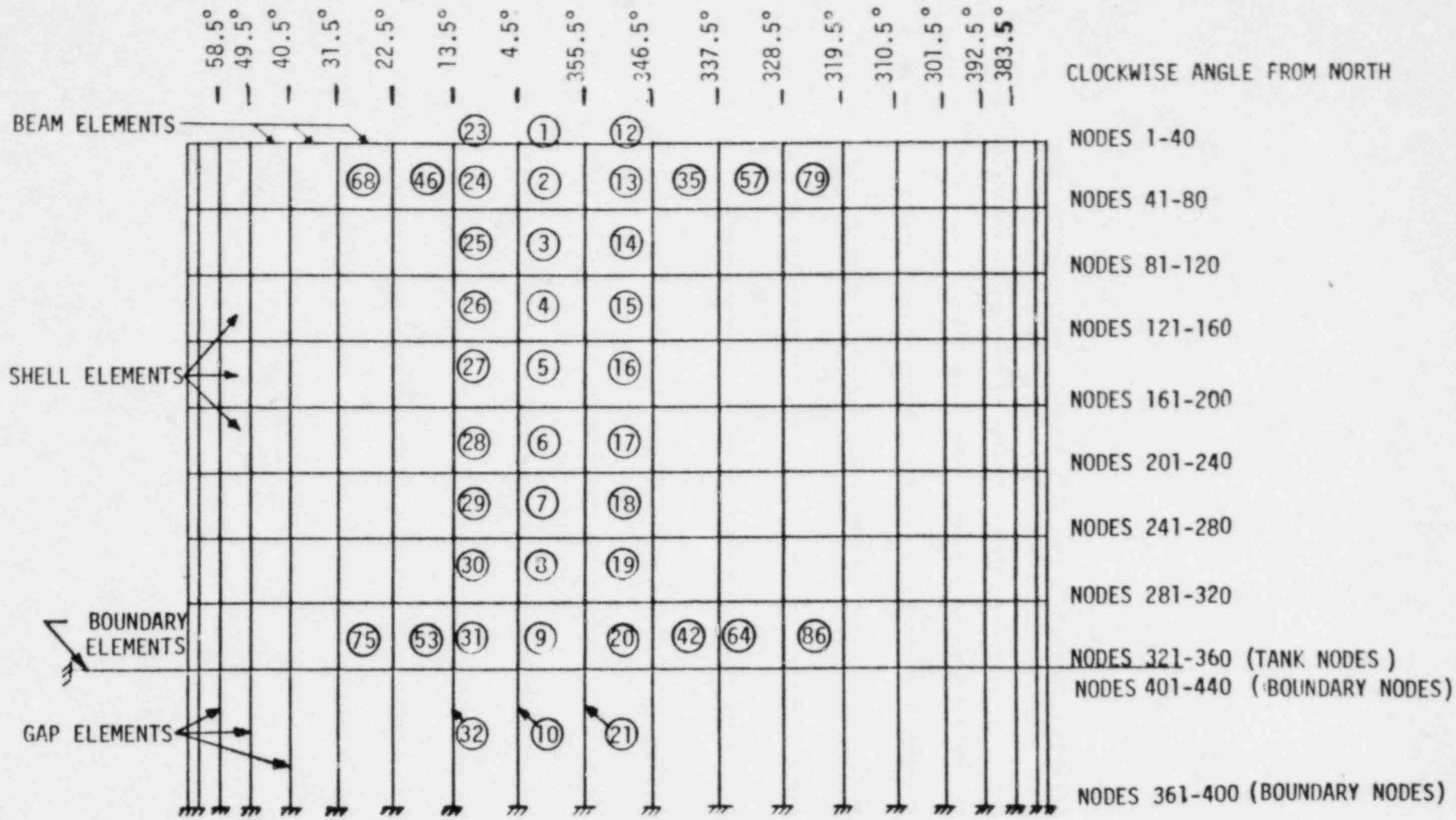
It should be concluded from this study that tank 1T-60 is acceptable as is subject to dye penetrant examination of bolt chair top plate welds at bolt location 27. Ring wall deformation and resulting anchor bolt loading in tank 2T-60 is significantly less than 1T-60 and tank 2T-60 is considered acceptable by comparison.



NOTE: INNER CIRCLE OF NUMBERS REPRESENT NODES ON THE TOP OF THE TANK. OUTER CIRCLE OF NUMBERS REPRESENT NODES AT THE OUTER END OF THE BOUNDARY ELEMENTS AT THE TANK BOTTOM.

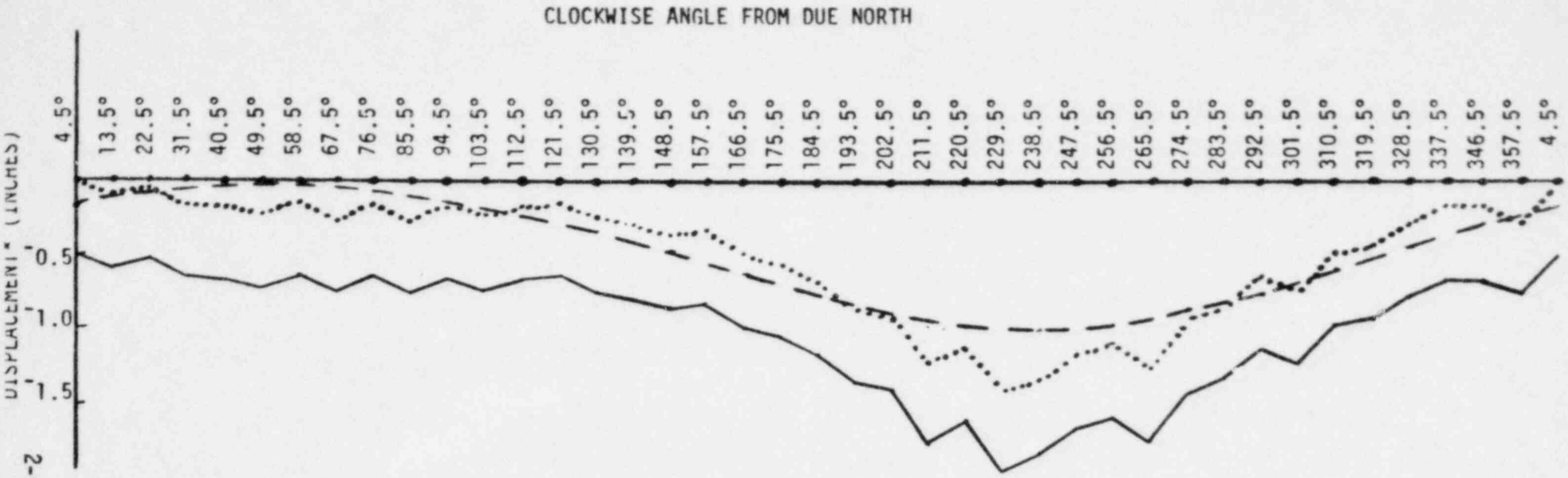
FIGURE 2-1. PLAN VIEW OF TANK MODEL

BORATED WATER STORAGE TANKS AT MIDLANDS



(xx) - Represent Element Numbers

FIGURE 2-2. ELEVATION VIEW OF TANK MODEL



* DISPLACEMENT RELATIVE TO THE AS BUILT TOP OF THE CELOTEX IN THE UNLOADED CONDITION

——— RINGWALL CONTOUR AFTER GROUND SETTLEMENT

•••••• TOP OF CELOTEX CONTOUR IN THE UNLOADED CONDITION AFTER GROUND SETTLEMENT (UNLOADED CONDITION = 1/2 INCH ABOVE RING WALL CONTOUR)

– – – TANK BOTTOM CONTOUR IN LOADED CONDITION AFTER GROUND SETTLEMENT

FIGURE 2-3. DISPLACEMENT OF TANK BOTTOM RELATIVE TO RINGWALL AND CELOTEX CONTOURS AFTER GROUND SETTLEMENT

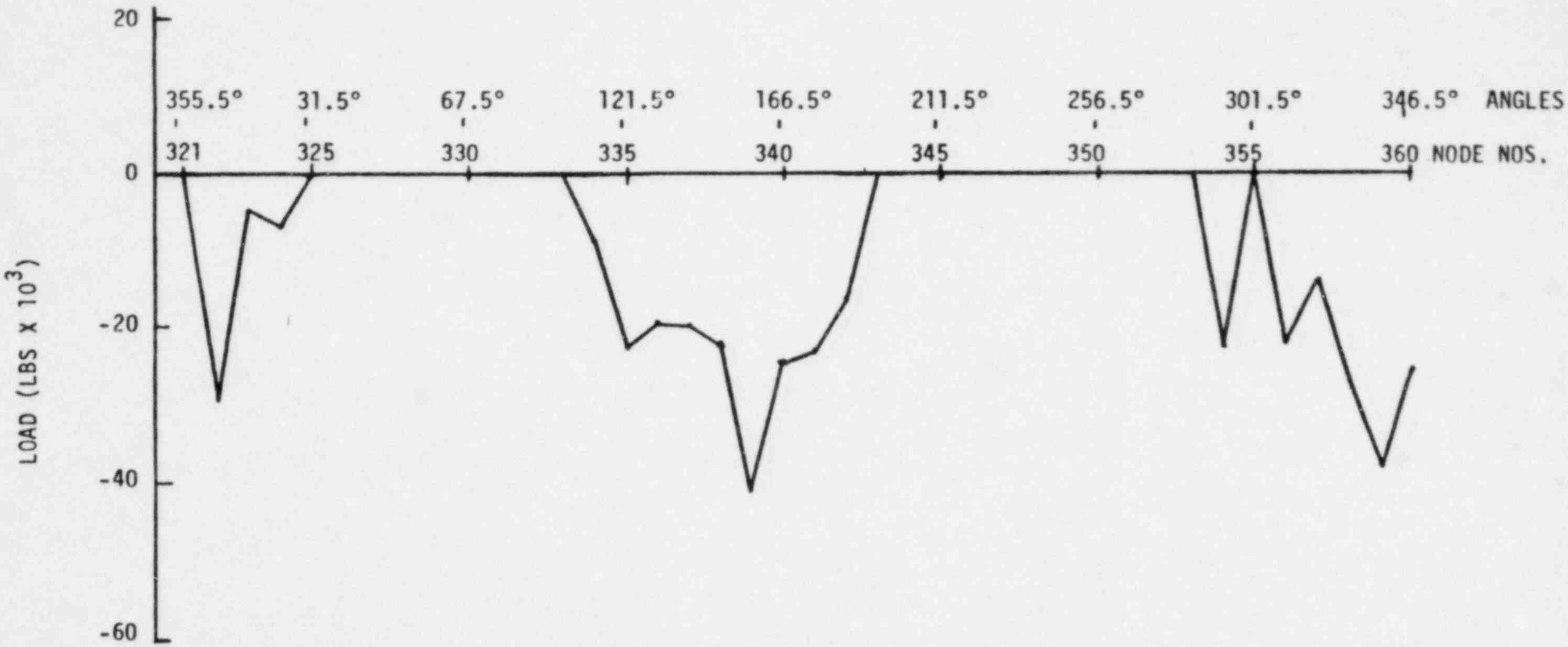


FIGURE 2-4. COMPRESSIVE LOADS AT TANK BOTTOM (SEE TABLE 5-3)

3. ACCEPTANCE CRITERIA

3.1 GOVERNING CODES AND STANDARDS

Governing codes and standards are delineated in the design specification, Reference 4. The BWSTs are designed and code stamped to the ASME code, Section III, Nuclear Power Plant Components, Subsection NC, Class 2 Components, Paragraph NC3300, Design of Vessels. The 1974 code with no addenda are applicable and Code Case 1607-1 is applicable for upset, emergency and faulted condition stress allowables. The API 650 code is also specified for design, Reference 10. In cases of conflict, the ASME Code governs.

The basic design is conducted using API-650 criteria since NC3300 of the ASME code does not specifically address flat bottom storage tank designs. NC3800 does provide criteria for flat bottom storage tanks and is essentially identical to API-650. The ASME code stress acceptance criteria from code case 1607-1 is used for evaluation of the OBE and SSE events.

Under the governing criteria the following stress intensities are allowed.

Loading Condition	Primary Membrane, P_m	Primary Local Membrane plus Primary Bending, $P_L + P_b$
Design and Normal	S	1.5S
Upset	1.1S	1.65S
Emergency	1.5S	1.8S
Faulted	2.0S	2.4S
Testing	1.25S*	1.87S**

* Not to exceed 0.9 S_y

** Not to exceed 1.35 S_y

S is the allowable stress intensity of 15.7 ksi for 304 L stainless steel. Secondary stresses do not require evaluation for Class 2 components designed by rule (NC3300 criteria). Minimum specified yield strength is 25 ksi. It can be seen from the allowable stress criteria that any stress in excess of 1.59 S exceeds the minimum specified yield strength and that primary local membrane and primary bending stress allowables all exceed the yield strength for all service conditions except design and normal.

3.2 STRESS CRITERIA FOR SETTLEMENT LOADING

Loading due to soil settlement is considered a one time application event that will be relieved when the ring wall foundations are retrofit and the tanks are shimmed to provide uniform support. There is no clear analogy to operating service conditions, however, the testing event would be the most applicable as acceptance criteria if we were to choose a similar stress category for which no further examination would be required. If the higher emergency condition allowables were used, some additional inspection may be warranted to assess deformation.

Flat bottom storage tanks are not hydrotested, but the basic ASME code philosophy regarding stress and permanent strain limits during pressure vessel hydrotesting is considered a rational philosophy for setting a conservative stress acceptance criteria for comparison to elastic stress analysis results. If elastically calculated stresses due to soil settlement do not exceed code acceptance criteria for testing conditions, then the component should be considered acceptable with no additional examination required.

ASME code philosophy for hydrostatic testing is to test the vessel at 1.25 times design pressure (formerly the hydrostatic test pressure was specified to be 1.5 times design pressure). Primary membrane stress intensity for design pressure is limited to S for Class 2 vessels and the primary local membrane plus primary bending stress intensity is limited to 1.5S. At a hydrostatic test pressure of 1.25 times the design pressure, the implied limit is 1.25S for primary membrane and 1.87S for primary local membrane plus primary bending. A limit is placed on yield for those cases where the allowable stress intensity, S, can be as high as 0.9 times yield. The limit is set at 0.9 times yield for primary membrane stress intensity and 1.35 times yield for primary local membrane plus primary bending stress intensity for cases where the primary membrane stress intensity is less than 0.67 times yield.

The ASME code philosophy in placing stress limits on hydrostatic testing induced stresses is to guard against gross deformation. The same philosophy is considered applicable to the BWST settlement problem. If stress intensities from settlement combined with hydrostatic pressure, weight and anchor bolt loading do not exceed hydrostatic testing allowables, then no detrimental effects are considered to have occurred.

Secondary stresses are not considered to be detrimental due to the single loading applications and are not considered to govern for acceptance criteria. This is consistant with ASME code design pholosophy for class 2 and 3 components and even for Class 1 components for infrequent (emergency) events. Note also, that if secondary stresses were limited to the shakedown regime (2 S_y) that during hydrostatic testing at 1.25 times the design pressure, primary plus secondary stress intensity could reach 2.5 S_y. Thus, code philosophy would allow primary plus secondary stress intensity to exceed the shakedown limit for single or limited numbers of events.

Areas where the above stress acceptance criteria may be exceeded should be treated on a case-by-case basis by applying ASME code limit analysis acceptance criteria, going to emergency condition allowables with some additional inspection requirements or limiting inelastic strains.

There is only one case where calculated stress intensities exceed the recommended allowable stress limits. This is in the bolt chair top plate for the most severely loaded bolt (bolt location 27). In this case, limit analysis concepts are employed. The ASME code for Class 1 components and component supports, References 12 and 13, allow limit analyses to be used in lieu of meeting elastic stress criteria for primary local membrane and primary bending stress intensities (Paragraphs NB 3228.2 and NF 3224 (a)). The bolt chair top plates are considered to be plate and shell-type component supports and limit analysis concepts from the component support code, (NF 3224 a) are considered to be applicable.

Under Level C Service (Emergency Conditions), Reference 13 allows 0.8 of the lower bound collapse load. The same allowable is also specified for Class 1 components (Reference 12). Design philosophy for Level C Service Condition allowables is that some permanent deformation may be experienced but the component is still serviceable. Deformation limits may be specified if deformation is a controlling factor for function. In the case of BWST bolt chairs, deformation is not a limiting factor and a limit analysis acceptance criteria analogous to that for Level C Service for Class 1 plate and shell-type component supports is considered a valid concept for evaluation of settlement induced loading on bolt chairs. As long as the bolt loads do not exceed 0.8 times the lower bound collapse load of the bolt chairs, they are considered to be acceptable without further analysis or retrofit being required. It is suggested, however, that for the one chair where the maximum bolt load occurs and is close to the recommended acceptance criteria, that a dye penetrant examination be conducted of fillet welds that attach the bolt chair top plates to the tank wall and gussets. This will ensure that any plastic deformation that may have occurred did not initiate any cracking.

Because of uneven support conditions, axial compressive stresses exist in the tank wall. For the large diameter thin wall storage tank, buckling will occur in the elastic range. The ASME code buckling criterion for axially loaded cylinders nominally contains a safety factor of 3 for sustained design loads. Since the condition under consideration is local and is more of a strain controlled condition than load controlled, a more liberal buckling criteria is recommended.

Reference 6 provides thin shell buckling formulae modified to reflect extensive test data. Formulae are provided for the case of uniformly axially loaded cylinders and cylinders subjected to bending moment wherein the axial compressive stress peaks at one location. The formulae contain correction factors based upon experimental data and are considered to be lower bound formulae. Since the axial compressive stresses are local and vary from compression to tension around the circumference, the bending formula for buckling is considered to be more appropriate. For the geometry under consideration, the applicable formula is:

$$\sigma_{cr} = \frac{0.6\ \gamma\ Et}{R}$$ where

γ = correction factor for cylinders in bending
E = Young's modulus
t = shell thickness
R = mean radius of shell

For the bending case and the geometry under consideration:

γ = 0.35 for the 0.25 inch thick shell
γ = 0.39 for the 0.375 inch thick shell

Critical buckling stresses are:

4750 psi for the 0.25 inch thick shell
7950 psi for the 0.375 inch thick shell

For strain controlled conditions the allowable axial compressive stress should be limited to $\sigma_{cr}/1.67$ resulting in allowable axial compressive stresses of:

2845 psi for 0.25 inch thick wall
4760 psi for 0.375 inch thick wall.

In the event that the above allowables are exceeded, a geometry check should be made to determine if elastic buckles have actually occurred. It should be noted that if elastic buckles do occur in a strain controlled condition, they will spring back upon removal of the applied loading. After the anchor bolt nuts have been loosened, part of the loading condition that could potentially have caused buckling was removed. Currently, the anchor holts are unloaded. During a site visit in September, while the anchor bolts were still loaded, there was no visual evidence of any buckling in the most critically loaded tank (1T-60). Subsequent analysis also demonstrates that buckling would not occur under the prior conditions.

4. ANALYTICAL MODELS AND ANALYSIS METHODS

A finite element model was contructed to represent the cylindrical shell portion of the BWSTs. Boundary conditions were applied at the top and bottom of the model to represent the restraint offered by the umbrella roof, the tank bottom and the asphalt impregnated fiberboard between the ring wall and tank bottom. Analysis of bolt chairs and the tank wall adjacent to the bolt chairs was conducted by hand methods using empirical design analysis approaches and classical stress analysis techniques.

4.1 FINITE ELEMENT MODEL

The tank wall finite element model was constructed for the ANSYS computer program (Reference 11). Plan and elevation views of the model are shown in Figures 2-1 and 2-2. ANSYS element 63 was used to represent the tank wall. Element 63 is a quadrilateral flat thin shell element with both membrane and bending stiffnesses. There are 40 elements around the circumference and 8 elements along the vertical axis. The elements are almost square being 49 inches X 48 inches. Each tank has 40 anchor bolts and, thus, the choice of 40 elements around the circumference. Anchor bolt locations correspond to nodal points along the bottom of the model.

The first two rows of elements from the bottom are 0.375 inches thick and the remaining six rows are 0.25 inches thick to represent the nominal tank wall thickness. At the top of the tank, beam elements are placed around the circumference of the tank to keep the tank round at the roof/tank wall junction. The umbrella roof is welded to a ring girder at the tank top and would keep the tank wall from ovaling under asymmetric loading.

At the tank bottom two types of elements are used to represent boundary conditions. Beam elements (element 4) are placed radially outward from the tank to represent the radial restraint of the tank wall afforded by the 0.25 inch thick tank bottom, the rotational stiffness of the tank bottom and to restrain the model from rotation about the vertical centerline of the tank. Vertical gap elements (element 10) are placed along the tank bottom to represent the stiffness of the asphalt impregnated fiberboard and any gaps that may exist between the tank bottom and ring wall. The gap elements have linear force displacement characteristics in the compression direction and zero load capacity in tension.

The actual asphalt impregnated fiberboard (Celotex) is nonlinear. Force-displacement properties were determined by laboratory testing of material taken from the site. Two tests were conducted to determine force-displacement relationships. One specimen was 3" X 3" and one was 3" X 6". The laboratory test report is included as Appendix A. Results from both tests were similar and 3" X 3" specimen results were used in developing an appropriate model for the boundary elements.

In order to model the force-displacement properties of compressible asphalt impregnated fiberboard, a beam on an elastic foundation model was used to find an appropriate equivalent linear stiffness for the boundary elements. The beam on an elastic foundation model is shown in Figure 4-1. Figure 4-2 compares a plot of the beam on an elastic foundation model results versus the equivalent linear stiffness used in the computer model for boundary elements. In developing the beam on an elastic foundation model, the resulting stiffness was biased on the conservative (stiff) side. Stiffer boundary elements result in the reaction loads being concentrated in more localized regions. Assumptions used in the beam on elastic foundation models are described as follows.

Referring to Figure 4-1, a rigid boundary is assumed on the tank bottom at the junction of the Celotex and the oil impregnated sand. This tends to be conservative in that the oil impregnated sand is assumed to be rigid, thus, forcing all deformation in the tank bottom to occur in a relative short span of 6.81 inches, the distance from the tank inside wall to the inner edge of the Celotex.

Several beam on elastic foundation solutions were carried out in order to include the nonlinear force-deflection characteristics of the Celotex. Equivalent linear springs were assumed for the Celotex material for discrete displacement ranges. For larger displacements, the equivalent spring rates were increased to approximately follow the force-deflection relationship determined in laboratory tests (Appendix A). For each displacement increment, the equivalent linear spring stiffness was selected to be the best fit of the nonlinear force-displacement relationship of the Celotex.

Referring to Figure 4-1, for a given force P, water pressure, w, shell rotational stiffness, $k\theta$, and Celotex effective stiffness, $\overline{k}$, a displacement directly under the load P was computed. Figure 4-2 plots four points determined by this method. Maximum displacement was determined to be less than 0.26 inches; therefore, a best fit linear stiffness for the three beam on an elastic founation solutions for 0.26 inches and less displacement was found. This equivalent linear stiffness of 3800 lbs/in/in of circumference was used to develop appropriate stiffnesses for the vertical boundary elements that represent the Celotex force-displacement characteristics.

4.2 FINITE ELEMENT MODEL LOADING

Loading conditions present in the BWSTs are:

- Dead weight of the tank roof
- Dead weight of the tank wall
- Anchor bolt loading
- Hydrostatic pressure acting radially on the tank
- Hydrostatic pressure acting downward on an annulus of the bottom plate adjacent to the tank wall.

Loads were applied to the model at nodal points that would best represent the physical forces acting on the tanks.

Tank roof weight was applied at the top of the model and was uniform around the circumference, i.e., equal loads at each of 40 nodal points. Dead weight of the tank was applied by specifying a 1g downward gravitational load. The program computes the proper nodal forces from the specified geometry, material density and g loading. Anchor bolt loading, tabulated in Table 1-3 was applied at nodal points along the bottom of the tank wall finite element model. Radial hydrostatic pressure was specified to be acting on the face of each shell element. Pressure must be specified as constant on element faces and the average pressure was used for each row of elements. In order to eliminate any fictitious bending stresses that could be computed due to use of flat plate elements to represent a curved shell, an option in the ANSYS code was used to transfer pressure loads on the element face to nodal forces. A pressure analysis check case was conducted to verify the model geometry and assure that hoop stresses and displacements were properly computed.

Most of the tank bottom rests on soil. There is an effective annulus of water adjacent to the tank wall that is not supported by soil. The resultant shear force on the tank bottom that is transferred to the tank wall and ultimately to the ring wall was derived and applied as nodal forces along the tank bottom.

In areas where gaps exist between the tank bottom and ring wall, the weight of an effective annulus of water that extends radially inward beyond the inside ring wall boundary is carried partially by the tank wall. Figure 4-3 shows the effective radial length, Δr, of this annulus of water. The effective annulus radial length, Δr, and the resulting vertical reaction load, P, at the tank bottom/shell junction are a function of the gap, δ, between the tank bottom and the free surface of the Celotex. Figure 4-4 plots the reacton, P, as a function of δ. The P versus δ plot was made using classical equations developed for seismic design of flat bottom storage tanks. Loading due to effe tive water weight is, thus, nonlinear, depending upon the gap between the tank bottom and its support. For the range of gaps calculated in the final solution, a weighted average of water weight was less than 110 lbs/in/inch of circumference. The final computer run conservatively used a water weight equal to 120 lbs/in/inch of circumference.

In regions where the tank bottom is supported, the effective annulus of water carried by the tank support is less than at gap locations. In order to avoid several iterative solutions that would require changing the effective water weight at points where the tank bottom was supported, the vertical spring boundary effective stiffness included the effect of the 120 lb/in/inch of circumference water weight. Consequently, a conservative constant value of water weight could be used without requiring iteration. Figure 4-5 portrays this pictorially. At points where the tank is in contact with the Celotex, the effective water weight of 120 lbs/in/in of circumference corresponds to a compression in the Celotex of 0.010 inches. Using a linear boundary element stiffness of 3800 lbs/in/in of circumference, the zero load point corresponds to a downward displacement of -0.021 inches. In other words, the starting point of compression in the Celotex is shifted 0.021 inches and all gaps are shifted 0.021 inches. The computer program boundary element reactions carried into the shell are correct and resulting shell stresses are correct. However, computed displacements relative to the starting point are 0.021 inches greater than actual and the sum of the vertical reactions is greater than actual reactions due to fictitious water weight being applied at points where the tank is supported, i.e., the excess water weight and excess reactions at the points of tank support cancel each other out and are not reacted by the shell.

Actual effective water weight is computed after completion of the finite element analysis and is equal to the unsupported circumferential distance times the water weight of 120 lbs/in plus a fraction of this weight at a few points where the displacement is so small that the full water weight cannot develop. This occurred at four boundary elements. At points where the tank is supported, the water weight is reacted directly into the soil and ring wall and there is no effective loading on the tank wall.

Weights of the tank, roof and internal bracing were taken from tank drawings. Tank shell weight applied to the model was checked against weights on the tank drawings and a total force balance was conducted to assure correct input for loadings. The relative contributions of downward loading from dead weight, anchor bolt loads and vertical water pressure loading reacted by the shell are:

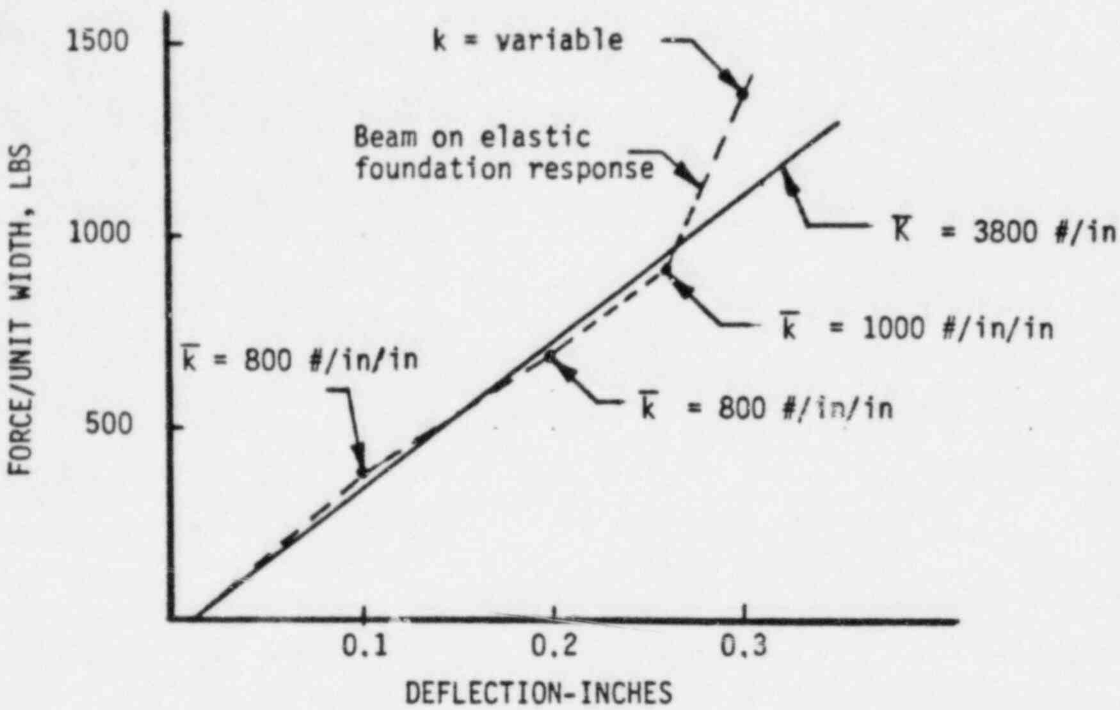
Tank Roof	36,000	lbs
Tank Shell and Hardware	67,090	lbs
Effective Water Weight Reacted by Shell	115,045	lbs
Total of Bolt Loads	168,470	lbs
	386,605	lbs

From the vertical force summary, it can be seen that the anchor bolt loads are 43.5% of the total vertical load. Now that the anchor bolt loads have been released, the total downward forces are considerably lower than the case evaluated.

4.3 BOLT CHAIR MODEL

Bolt chairs were evaluated by hand calculations. Initially, the method of Reference 8 used in design of the bolt chairs, was used to determine stress response in the top plate for a maximum bolt load of 31,310 pounds. The method assumes that a beam of width equal to the edge distance from the hole to the plate outside edge carries 1/3 of the total bolt load. Figure 4-6 shows the analytical model.

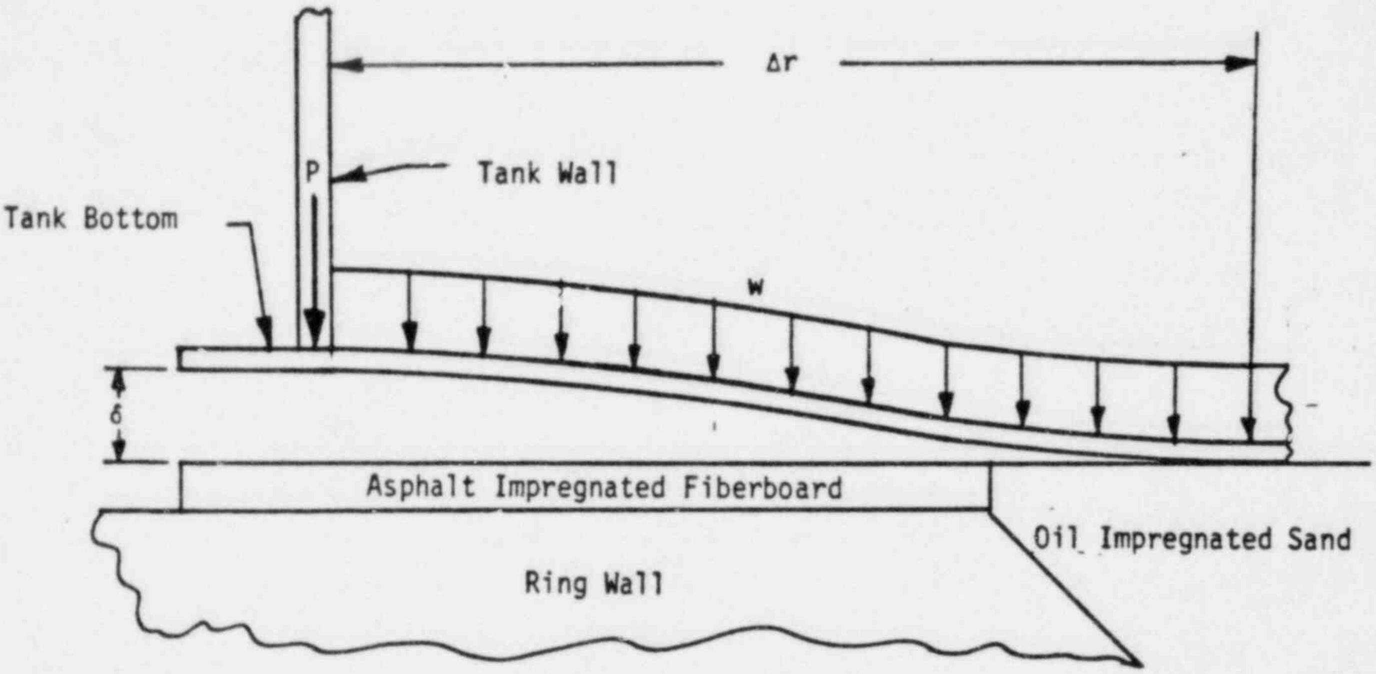
Since this model resulted in yielding of the top plate for some of the measured bolt loads in Tank 1T-60, a yield line analysis (limit analysis) was conducted to develop a limit load capacity. Figure 4-7 is the yield line model. Minimum limit load capacity was calculated to be 39.95 kips which is above the maximum measured bolt loading of 31.3 kips by a factor of 1.28. The acceptance criterion developed in Chapter 3 requires a safety factor of 1.25 on the lower bound collapse load, thus, the criterion is satisfied.



$\overline{k}$ = Average spring rate at asphalt impregnated fiberboard over finite deflection range, lbs/in/in/unit width

$\overline{K}$ = Average spring rate of tank support, lbs/in/unit width

FIGURE 4-2. LINEARIZATION OF BOUNDARY SPRINGS



w = water force, lbs/in/unit width

P = tank and roof weight, lbs/unit width

δ = gap between tank bottom and asphalt impregnated fiberboard, in.

Δ_r = effective width of water annulus, in.

FIGURE 4-3. EFFECTIVE WATER ANNULUS

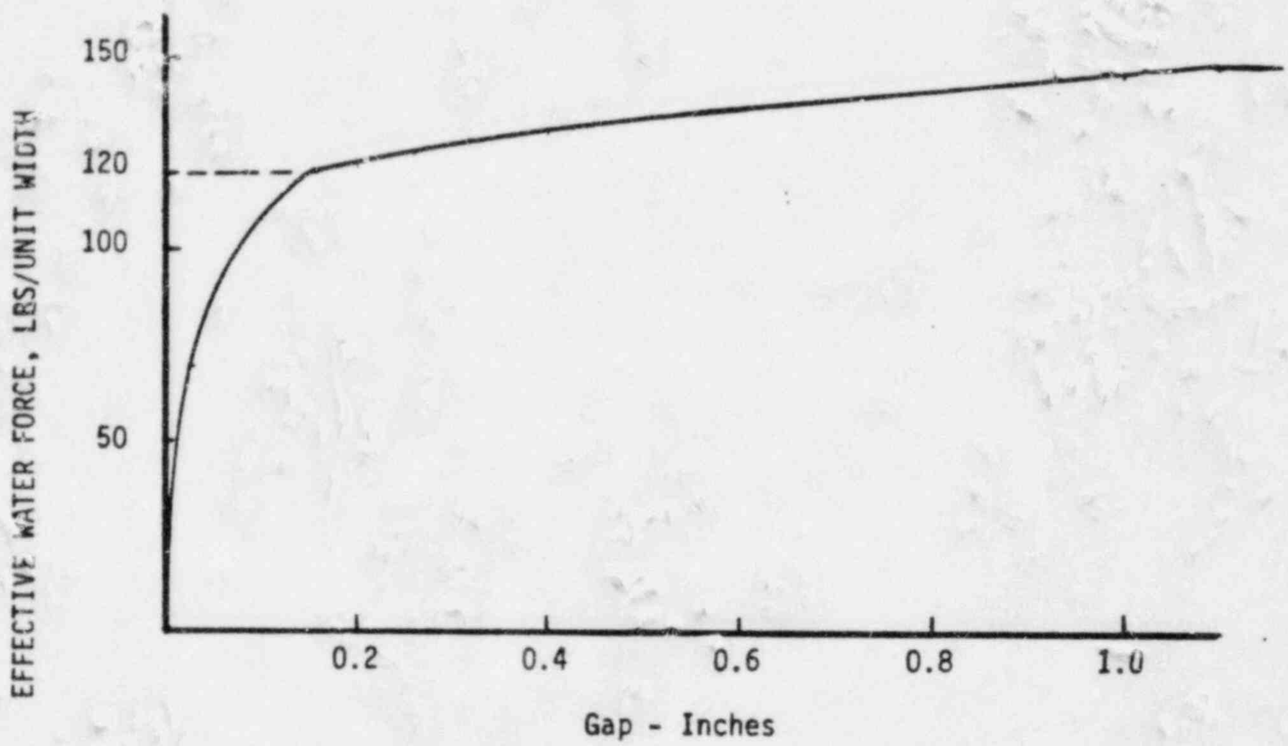


FIGURE 4-4. EFFECTIVE WATER FORCE/UNIT WIDTH OF CIRCUMFERENCE VS GAP

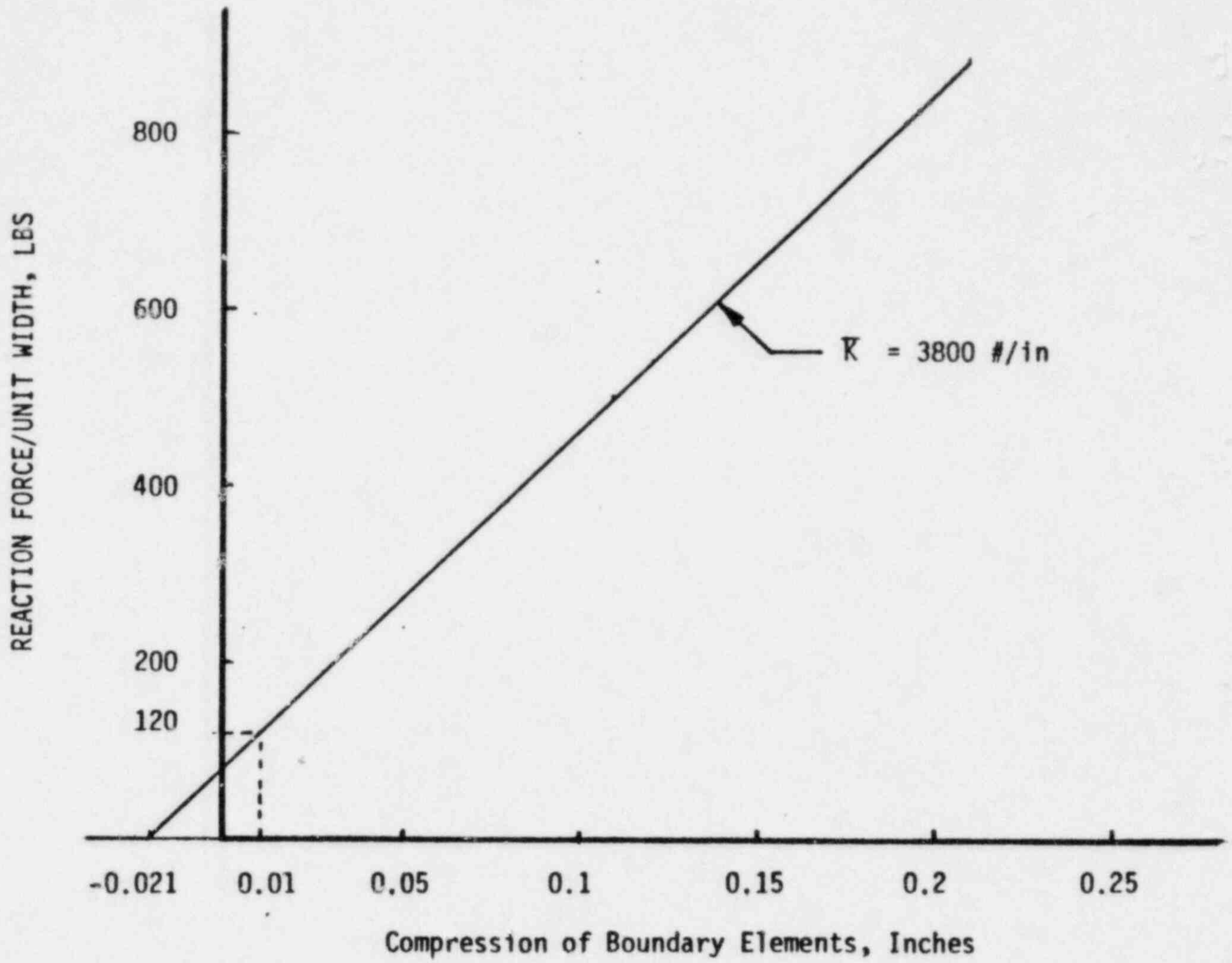


FIGURE 4-5. FORCE VS DEFLECTION AT BOUNDARY ELEMENTS

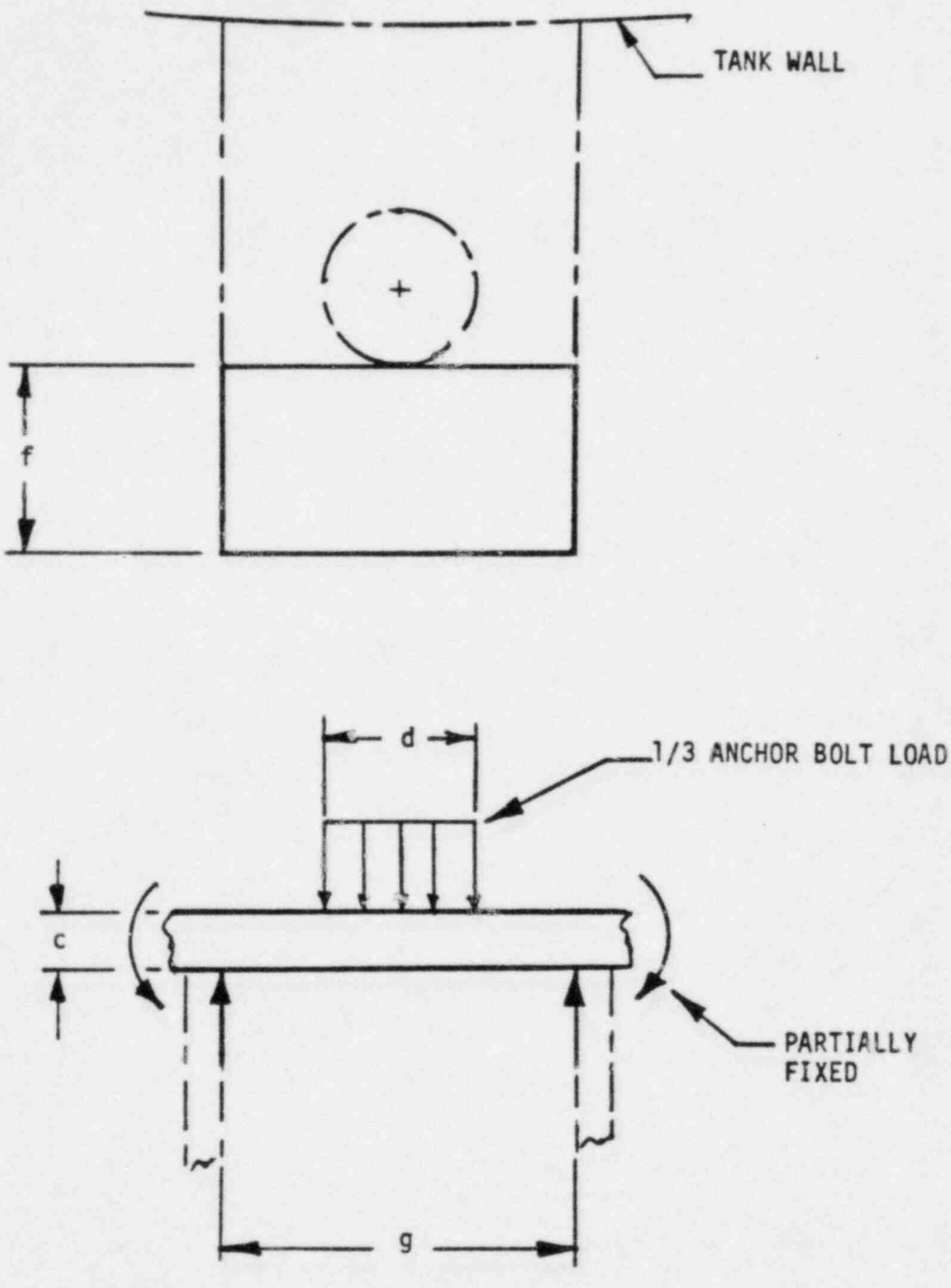


FIGURE 4-6: BEAM MODEL FOR BOLT CHAIR DESIGN

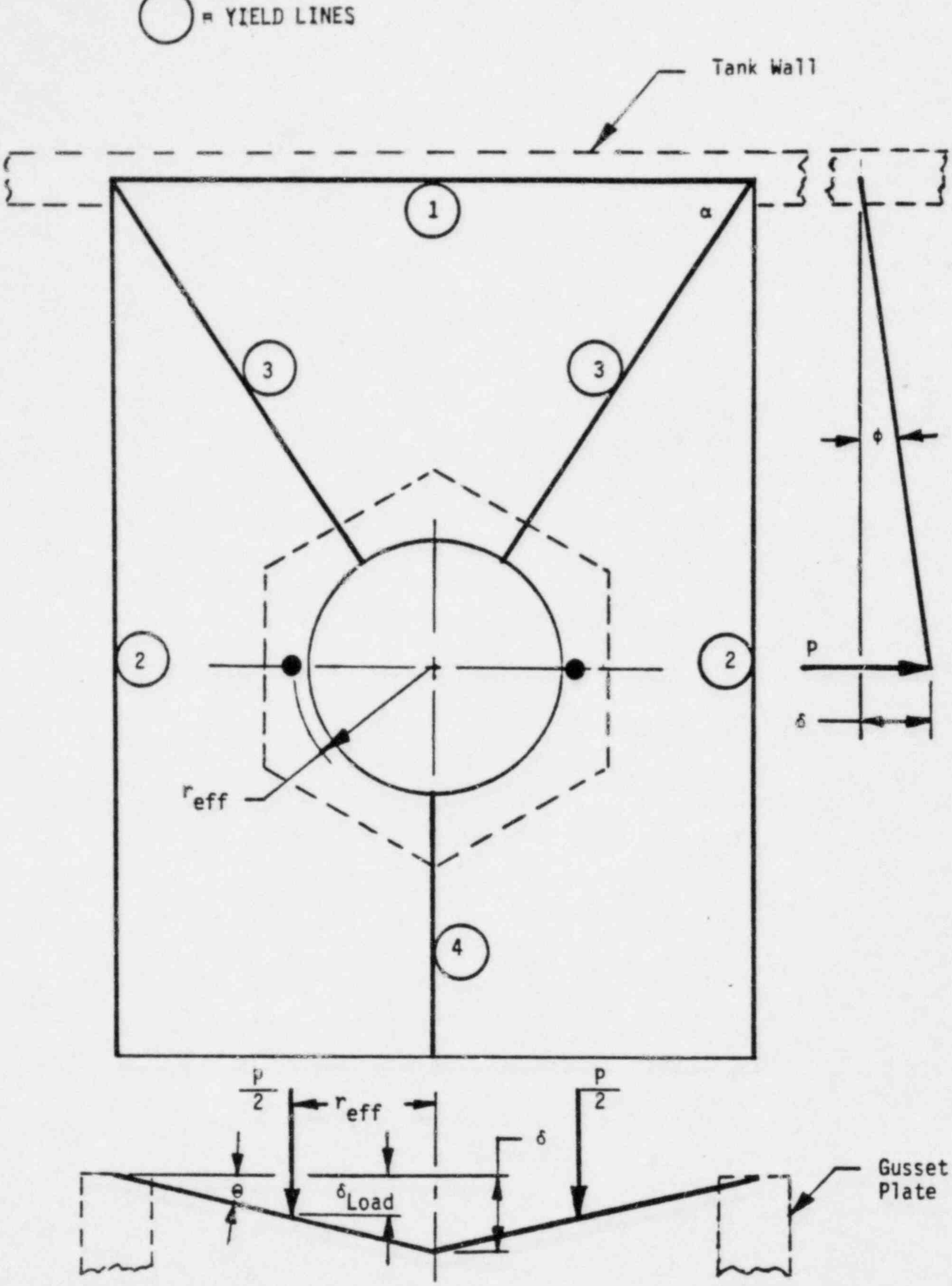


FIGURE 4-7. YIELD LINE MODEL FOR BOLT CHAIR

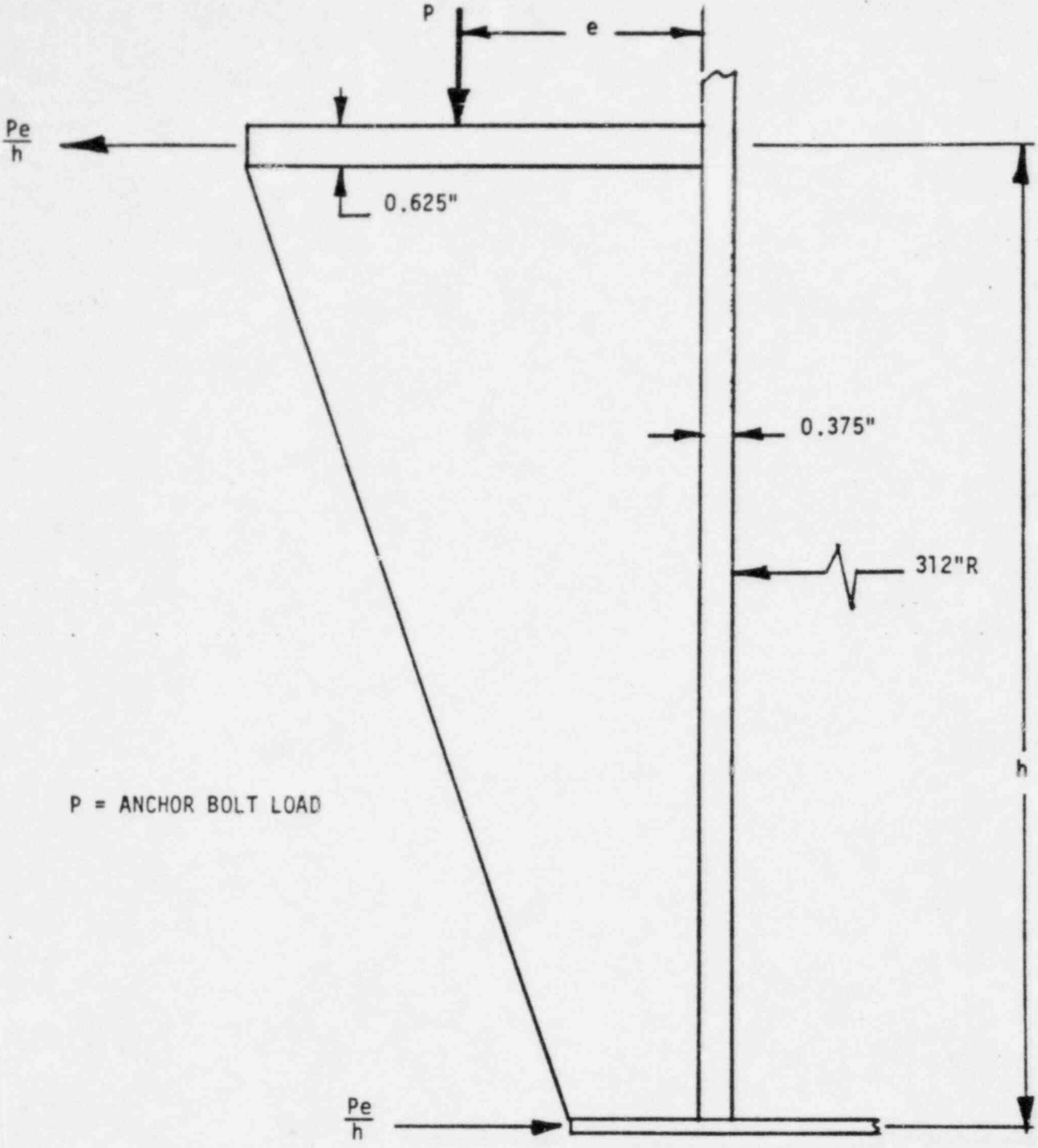


FIGURE 4-8. ANALYSIS MODEL FOR LOCAL MEMBRANE STRESSES IN SHELL DUE TO ANCHOR BOLT LOADING

5. ANALYTICAL RESULTS

Results from the finite element model analysis and hand calculations are summarized in this chapter. Computer output is too voluminous to include, consequently, only important highlights of the output are summarized.

5.1 RESULTS FROM FINITE ELEMENT MODEL

Important stress results from the finite element model are from the bottom 3 rows of elements representing the lower 12 feet of the tank. The highest loading occurs in the bottom row of elements where the shell thickness is 0.375 inches. Since the bottom elements are restrained from radial movement by the tank bottom, hoop stresses in the second row of elements are greater than for the bottom row. At the junction of the second to third row of elements, the tank wall thickness changes from 0.375 inches to 0.25 inches. Maximum stress intensity occurs in the third row of elements at element 392 where the hoop stress is tensile and negative axial compressive stress exists. Above row 3, element stresses diminish with decreasing hydrostatic pressure and a more even distribution of the asymmetric vertical reaction loads.

Because of the wave front solution techniques used in ANSYS, elements in a column are in numerical suequence while elements in a row (around the circumference) are numbered in steps of 22 (Figure 2-2). Table 5-1 tabulates the three stress components and resulting stress intensity for the first three rows of elements beginning at row 3 and working down, row 1 being the bottom row.

Stress output is given at the center of each element and represents the average stress in the element. In addition to output at the center of each element, stress output at the bottom edge of each element was requested. Table 5-2 summarized stress components for each element along the tank wall bottom. Along the bottom, tank hoop stresses are low due to radial restraint offered by the tank bottom. Axial stresses are more concentrated along the tank bottom resulting from concentrated anchor bolt loading and uneven vertical reactions from the distorted ring wall.

Vertical reactions carried into the shell along the bottom row of nodes (nodes 321 through 360) are tablulated in Table 5-3 and were visually displayed in Figure 2-4. Displacements of the tank bottom and surface of the asphalt impregnated fiberboard were previously plotted in Figure 2-3.

5.2 BOLT CHAIR TOP PLATE

Figure 4-7 shows the yield line model used to evaluate limit load capacity of the top plate of the bolt chairs. In this model, plastic hinges were assumed to form in the 0.5 inch thick gusset plates, the 0.375 inch thick tank wall and the 0.625 inch thick top plate. For a 25,000 psi minimum yield material, and assuming elastic-perfectly-plastic material behavior, the plastic hinge moment capacities are computed to be:

2440 in-lbs/in. in the .625 inch thick top plate
1560 in-lbs/in. in the 0.50 inch thick gussets
879 in-lbs/in. in the tank wall.

Work-energy relationships were used to solve for the limit load capacity. The angle α in Figure 4-2 was varied to determine a minimum value of capacity. Resulting values at minimum capacity are:

$$\alpha = 56.8^{\circ}$$
$$P_{limit} = 39.95 \text{ kips}$$

The limit load is a factor of 1.28 greater than the maximum measured bolt load of 31.3 kips and meets the acceptance criteria for limit analysis developed in Chapter 3.

5.3 TANK WALL AT BOLT CHAIR LOCATION

Design calculations contained in Reference 7 indicate tank wall stresses in excess of yield for the faulted condition loading (SSE plus normal operating loading). Calculations are based on a formula given in Reference 8. The formula in Reference 8 is empirical and combines membrane and bending stresses. For local loadings on shells, the ASME code defines the resulting membrane stress as a primary local membrane stress and the bending stress as secondary. The code stress acceptance criteria, Reference 5, places a limit on primary local membrane stress intensity but ignores secondary stress intensity. This is justifiable if the components do not undergo a large number of cycles of loading. For the case under consideration, the loading is a one-time-only event and secondary stresses can justifiably be ignored.

In order to compute the membrane components of stress in the tank wall due to bolt chair loading, the methods of Reference 9 were applied in a conservative manner. The top plate was assumed to be loaded radially outward, resisting the applied moment from anchor bolt loading. Figure 4-8 shows the analytical model. This model is very conservative since no credit is taken for load distribution into the shell from the gusset plates.

In applying Reference 9 to the problem, non-dimensional membrane forces $R_m N\phi/P$ and $R_m N_x/P$ are given for R/t ratios up to 300. The R/t ratio of the tank wall is 832. Consequently, non-dimensional membrane forces were extrapolated via log-log plots of $R_m N/P$ vs γ. The value of β, which is a function of the ratio of the lug dimensions to the shell radius, is small and was conservatively taken to be near zero for the extrapolation process.

For a maximum experimentally determined anchor bolt load of 31,310 pounds, the local primary membrane stress components were determined to be:

$$\sigma_\Theta = 13.20 \text{ ksi}$$
$$\sigma_x = 12.16 \text{ ksi}$$

These stress components were combined with stress components from the finite element analysis. The finite element analysis resultant stresses account for all loading conditions except for the bolt chair moment. Maximum bolt chair loading occurs between elements 130 and 152. Membrane stresses were averaged between these elements and added to local membrane stresses computed for the bolt chair moment.

The finite element model stresses are:

Element	σ_θ	σ_x	$\tau_{\theta x}$
130	6385	1640	331
152	6383	1589	-13
Avg.	6384	1614	159

Resulting local membrane stress components are:

σ_θ = 19,584 psi
σ_x = 13,744 psi
$\tau_{\theta x}$ = 159 psi

Primary local membrane stress intensity derived from the stress components is:

S = 19,588 psi

The derived stress intensity is conservative for two reasons:

1. Finite element stresses are taken at the center of the element. Hoop stress diminishes toward the bottom of the element due to the radial restraint offered by the tank bottom. The point of application of the radial bolt chair reaction load is about midway between the tank bottom and the mid point of the element; thus, actual hydrostatic pressure induced hoop stresses would be lower than assumed above.

2. Computed local membrane stresses are conservative since the total moment reaction is assumed to be taken out by a concentrated radial load on the tank wall at the bolt chair top plate. Load distribution into the tank wall from the gusset plates is ignored.

Using the design stress formula of Reference 8, the maximum primary local membrane plus secondary stress components in the axial tank wall direction is computed to be 41.94 ksi. Since a major portion of the stress is bending the resultant stress is assumed for design purposes to be negative and when combined with the positive hoop stress of 6.38 ksi from hydrostatic pressure loading, the maximum primary plus secondary stress intensity is:

$$S = 48.32 \text{ ksi}$$

This stress combination is not restricted by the governing design criteria, Reference 5. ASME code philosophy regarding shakedown to elastic action suggests that primary plus secondary stress intensities be limited to 2 S_y (2x25,000 psi) for cyclic loading. The above stress intensity meets the 2 S_y shakedown philosophy and is of no concern.

TABLE 5-1

STRESSES AT CENTER OF ELEMENTS
BOTTOM THREE ROWS - PSI

ROW 3

ELEMENT NO.	σ_θ	σ_x	$\tau_{x\theta}$	S
7	10581	-1429	544	12060
18	10600	-1106	761	11805
40	10619	-406	918	11176
62	10633	127	858	10703
84	10642	199	799	10703
106	10661	768	721	10713
128	10685	1708	460	10708
150	10689	1743	38	10689
172	10669	1056	-208	10674
194	10650	523	-404	10666
216	10646	361	-495	10670
238	10648	448	-599	10683
260	10634	193	-694	10680
282	10621	-536	-757	11258
304	10615	-509	-532	11174
326	10614	-457	-649	11146
348	10608	-894	-434	11536
370	10587	-1223	-314	11826
392	10571	-1923	-85	12495
414	10582	-1778	301	12374
436	10604	-575	543	11232

ROW 3

ELEMENT NO.	σ_θ	σ_x	$\tau_{x\theta}$	S
29	10573	-1885	332	12475
51	10516	-1807	84	12385
73	10590	-1189	281	11792
95	10596	-1093	459	11726
117	10593	-1262	-638	11924
139	10606	-893	-851	11624
161	10624	-116	-912	10894
183	10641	188	-814	10704
205	10657	650	-790	10719
227	10676	1431	-526	10706
249	10684	1594	-197	10689
271	10679	1337	107	10681
293	10670	1199	337	10682
315	10659	824	619	10698
337	10645	311	702	10692
359	10632	-19	786	10766
381	10614	-432	743	11146
403	10607	-974	733	11674
425	10613	-605	425	11250

TABLE 5-1 (Continued)

ROW 2

ELEMENT NO.	σ_Θ	σ_x	$\tau_{x\Theta}$	S
8	10866	-1081	349	11966
19	10872	-895	502	11810
41	10830	-309	627	11210
63	10801	129	564	10831
85	10805	153	529	10831
107	10781	598	484	10804
129	10731	1376	318	10741
151	10737	1375	7	10737
173	10767	830	-126	10768
195	10783	434	-278	10790
217	10796	288	-325	10806
239	10796	347	-404	10812
261	10792	201	-448	10811
283	10848	-459	-544	11360
305	10831	-378	-299	11225
327	10827	-324	-484	11192
349	10870	-752	-258	11632
371	10854	-908	-215	11770
393	10897	-1503	-67	12400
415	10908	-1450	196	12364
437	10882	-381	404	11232

ROW 2

ELEMENT NO.	σ_Θ	σ_x	$\tau_{x\Theta}$	S
30	10900	-1488	248	12396
52	10899	-1433	-80	12332
74	10862	-907	-177	11774
96	10864	-857	-310	11738
118	10869	-985	-417	11882
140	10858	-714	-573	11628
162	10808	-52	-618	10930
184	10807	146	-520	10832
206	10787	503	-547	10817
228	10742	1156	-344	10755
250	10741	1263	-133	10743
272	10757	1043	79	10758
294	10753	968	207	10758
316	10774	659	434	10793
338	10800	246	451	10819
360	10816	-19	539	10888
382	10819	-290	469	11148
404	10867	-799	539	11716
426	10845	-479	220	11332

TABLE 5-1 (Continued)

ROW 1

ELEMENT NO.	σ_{θ}	σ_x	$\tau_{x\theta}$	S
9	5891	-1173	332	7096
20	5925	-1092	498	7086
42	6042	-341	642	6510
64	6135	190	554	6186
86	6164	159	526	6210
108	6252	664	485	6294
130	6385	1640	331	6408
152	6383	1588	-13	6383
174	6297	944	-109	6300
196	6230	522	-290	6245
218	6188	329	-317	6205
240	6176	378	-412	6205
262	6155	319	-425	6186
284	6029	-615	-594	6750
306	6039	-405	-231	6461
328	6033	-312	-546	6438
350	5937	-964	-218	6914
372	5922	-954	-227	6892
394	5824	-1733	- 71	7558
416	5822	-1784	184	7614
438	6008	-307	456	6380

ROW 1

ELEMENT NO.	σ_{θ}	σ_x	$\tau_{x\theta}$	S
31	5812	-1739	280	7572
53	5818	-1683	-109	7504
75	5905	-997	-162	6910
97	5918	-995	-317	6942
119	5920	-1130	-408	7097
141	5972	-863	-578	6932
163	6010	6	-630	6165
185	6150	149	-493	6190
207	6229	553	-571	6286
229	6350	1381	-333	6372
251	6372	1466	-137	6376
273	6338	1175	91	6339
295	6322	1151	184	6329
317	6255	765	459	6294
339	6173	281	428	6204
361	6109	-43	561	6254
383	6065	-251	433	6375
405	5961	-995	601	7058
427	5994	-568	145	6568

TABLE 5-2

STRESSES AT CENTER OF LOWER EDGE OF ELEMENT, BOTTOM ROW - PSI

ROW 1

ELEMENT NO.	σ_θ	σ_x	$\tau_{x\theta}$
9	2987	-1180	332
20	3044	-1098	498
42	3233	-348	641
64	3384	184	554
86	3433	151	526
108	3576	657	484
130	3788	1633	331
152	3785	1582	-13
174	3650	937	-109
196	3541	515	-290
218	3472	322	-316
240	3452	371	-412
262	3414	312	-425
284	3216	-622	-594
306	3228	-412	-231
328	3217	-318	-546
350	3064	-971	-218
372	3038	-961	-227
394	2881	-1740	-71
416	2879	-1790	184
438	3174	-314	456

ROW 1

ELEMENT NO.	σ_θ	σ_x	$\tau_{x\theta}$
31	2861	-1746	280
53	2871	-1690	-109
75	3010	-1004	-162
97	3032	-1001	-317
119	3036	-1136	-408
141	3120	-869	-577
163	3327	0	-630
185	3410	143	-493
207	3539	546	-571
229	3732	1374	-332
251	3769	1459	-137
273	3715	1168	90
295	3687	1144	184
317	3580	758	459
339	3447	274	428
361	3343	-50	561
383	3269	-258	432
405	3104	-1001	601
427	3154	-574	145

TABLE 5-3

VERTICAL REACTIONS INTO SHELL- LBS.

NODE	REACTION
321	0
322	-29163
323	-4671
324	-7183
325	0
326	0
327	0
328	0
329	0
330	0
331	0
332	0
333	0
334	-8701
335	-22443
336	-19812
337	-19971
338	-22206
339	-40898
340	-24302
341	-23464
342	-16538
343	0
344	0
345	0
346	0
347	0
348	0
349	0
350	0
351	0
352	0
353	0
354	-22081
355	0
356	-21393
357	-14061
358	-26501
359	-37680
360	-25537

Sum of vertical reactions into shell = 386605 lbs

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ADDENDA

BORATED WATER STORAGE TANK ANALYSIS
FOR END-OF-LIFE SOIL SETTLEMENT
AND SEISMIC MARGIN EARTHQUAKE
LOADING CONDITIONS

A1. INTRODUCTION

A1.1 STATEMENT OF PROBLEM

Ring walls supporting the Borated Water Storage Tanks (BWSTs) are being reinforced by Bechtel Power Corporation and the tanks will be releveled to a condition of uniform support. Subsequent to the proposed ring wall retrofits and tank releveling, further soil settlement is projected to occur. This addenda evaluates the effects of the projected future soil settlement on the BWSTs. Bechtel Power Corporation has undertaken a study to quantify the ring wall distortion resulting from soil settlement over the 40-year life of the plant. Tabulated distortion data were provided to SMA by Bechtel personnel (Reference A1) and are presented within Table A1-1. Figure A1-1 from Reference A2 is a graphical presentation of predicted future settlement and distortion.

A1.2 PURPOSE OF STUDY

The purpose of this study is to evaluate the effects of future soil settlement loading conditions combined with the seismic margin earthquake and to demonstrate compliance with applicable design codes for the combined loading.

A1.3 SCOPE OF WORK

The scope of work consists of performing a finite element analysis of the BWSTs to determine the stress conditions in the tank wall, anchor bolts and bolt chairs caused by soil settlement and consequent ring wall distortion. Tank models, with and without a ½ inch layer of asphalt impregnated fiberboard (Celotex) between the tank bottom and ring wall, are analyzed in order to assess support stiffness effects on stress levels in the tanks. Calculated stress from the static load cases are combined with seismically induced stress to produce a combined response. The seismic loads are taken from the Midland Seismic Margin Study as described in Section A4.4. The combined stresses due to the seismic and settlement response are compared to 1974 ASME Code acceptance criteria as described in Section A3.

A1.4 GENERAL APPROACH

The general approach to analyzing the effects of the end-of-life settlement condition on the BWST is very similar to the analysis performed on the BWSTs for current settlement condition analysis, as described in Section 1.5. The finite element model constructed to analyze the end-of-life soil settlement condition is shown in Figure A1-2. This model is nearly identical to the model which was constructed for the analysis of the current soil settlement condition (Figure 2-2), with the exception of the addition of 40 three-dimensional spar elements to simulate anchor bolts. Spar elements within the ANSYS program are defined as uniaxial tension-compression elements with three displacement degrees of freedom at each node. These spar elements have been placed in parallel to the gap elements. Each set of nodes which formerly defined the endpoints for the gap elements, now define the endpoints for a parallel set of a gap element and a spar element. The gap element and spar element combinations model the anchor bolt, Celotex and ring wall interface, as described in Section A4.

The loading conditions on the tank cylindrical shell, which are reacted by the ring wall, include the weight of the tank roof, the weight of the cylindrical shell, the weight of an effective annulus of water, and the bolt loads which are caused by the ring wall deflection. Another loading imposed on the model is the hydrostatic radial pressure acting on the tank wall.

Section A2 summarizes the overall results and conclusions of this study. Section A3 presents acceptance criteria used for evaluating seismic and settlement-induced tank stresses. Section A4 describes the tank model, boundary conditions, loading conditions and methods of solution in detail. Detailed results are presented in Section A5. Only one tank was analyzed since future settlement and seismic loading are applicable to both BWSTs.

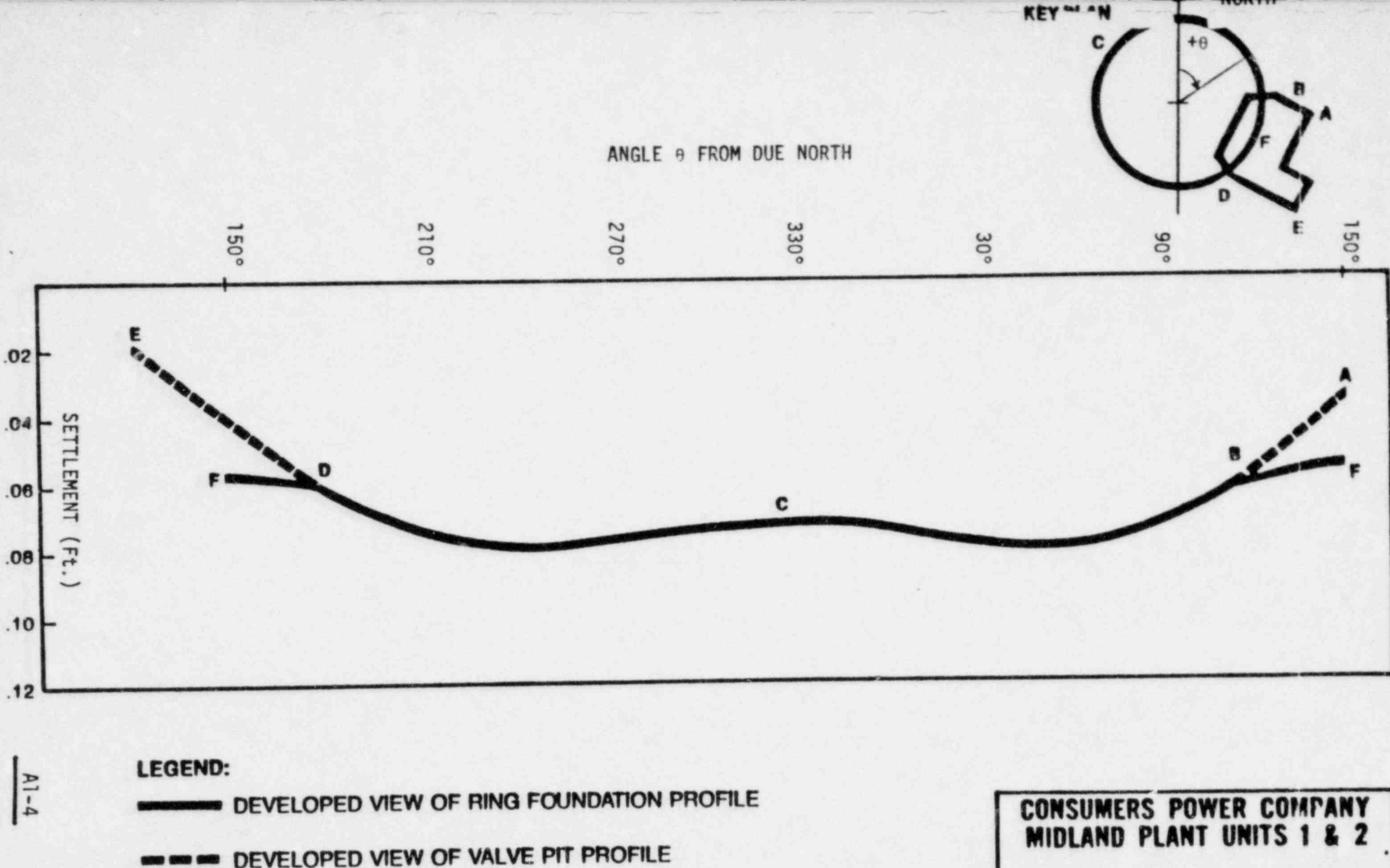
TABLE A 1-1

PREDICTED

END-OF-LIFE SETTLEMENT DATA FOR BWST #1T-60

Angle θ from North	Settlement (ft.)
0.22°	0.0745
7.72°	0.0754
15.22°	0.0764
22.72°	0.0773
30.22°	0.0782
37.72°	0.0789
45.22°	0.0794
52.72°	0.0796
60.22°	0.0794
67.72°	0.0787
75.22°	0.0774
82.72°	0.0756
90.22°	0.0729
97.72°	0.0699
105.22°	0.0669
112.72°	0.0640
120.22°	0.0616
127.72°	0.0597
135.22°	0.0583
142.72°	0.0575
150.22°	0.0573
157.72°	0.0576
165.22°	0.0586
172.72°	0.0601

Angle θ from North	Settlement (ft.)
180.22°	0.0622
187.72°	0.0648
195.22°	0.0676
202.72°	0.0706
210.22°	0.0732
117.72°	0.0752
225.22°	0.0765
232.72°	0.0776
240.22°	0.0780
247.72°	0.0781
255.22°	0.0779
262.72°	0.0774
270.22°	0.0767
277.72°	0.0759
285.22°	0.0751
292.72°	0.0743
300.22°	0.0736
307.72°	0.0731
315.22°	0.0727
322.72°	0.0725
330.22°	0.0725
337.72°	0.0727
345.22°	0.0732
352.72°	0.0738



A1-4

CONSUMERS POWER COMPANY
MIDLAND PLANT UNITS 1 & 2

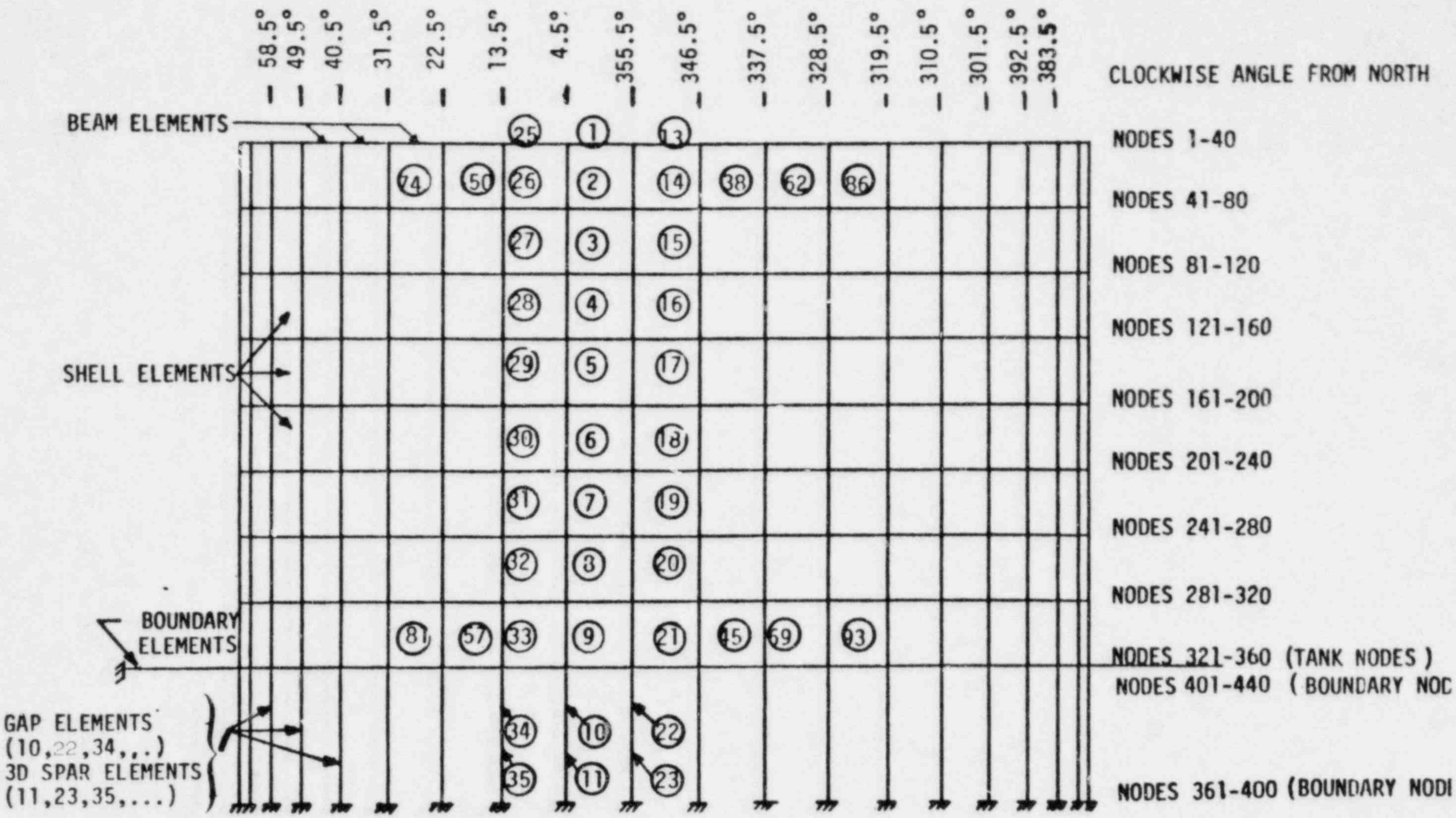
BORATED WATER STORAGE TANK FOUNDATION
FOUNDATION SETTLEMENT FROM FINITE ELEMENT ANALYSIS

FIGURE BWST-12

G-1834-15

FIGURE A 1-1: RINGWALL SETTLEMENT CURVE FOR TANK 1T-60 END OF LIFE CONDITION

BORATED WATER STORAGE TANKS AT MIDLANDS



(xx) - Represent Element Numbers

FIGURE A 1-2: ELEVATION VIEW OF TANK MODEL (END OF LIFE CONDITION)

A2. SUMMARY AND CONCLUSIONS

A2.1 SUMMARY OF RESULTS

The five critical areas of the Borated Water Storage Tank for combined seismic and settlement type loading conditions have been determined to be:

1) Compression in 3/8 inch tank wall
2) Compression in 1/4 inch tank wall
3) Local membrane stress in tank wall at the bolt chair
4) Bolt chair top plate
5) Anchor bolts

Table A2-1 contains a summary of the stress resultants for the tank model configuration with a ½ inch layer of Celotex, along with the appropriate allowable stress limits for each of these five critical areas. As shown in Table A2-1, each of the total response stresses falls within the allowable stress limit. The factor of safety relative to the allowable response for each critical area is given below:

1) Factor of Safety on Allowable Buckling Stress in the 3/8 inch shell = 2.22.
2) Factor of Safety on Allowable Buckling Stress in the 1/4 inch shell = 1.53.
3) Factor of Safety on Allowable Local Membrane Stress in the Shell at the Bolt Chair = 1.71
4) Factor of Safety on Allowable Bending Stress in the Bolt Chair Top Plate = 1.08
5) Factor of Safety on Allowable Anchor Bolt Loading = 2.65

Note that there is an additional factor of safety included in the design code allowable stress limits.

Table A2-2 contains a Stress Resultant Summary for the tank model configuration without the Celotex layer along with the appropriate allowable stress limits for the five critical areas of the tank. Table A2-2 shows that the maximum compressive stress in the 1/4 inch shell is 3,657 psi compared to an allowable stress of 2,834 psi and the bolt chair top plate bending stress is 41,386 psi compared to an allowable of 37,680 psi. Thus, the use of Celotex is necessary in order for combined settlement and seismic margin loading to remain within design code allowables. The stress resultants at the three remaining critical areas on the BWST are below the allowable stresses and would be acceptable for the case of no Celotex.

Section A5 of this addenda contains a more detailed description of the results presented within Table A2-1 and Table A2-2.

A2.2 CONCLUSIONS

The following conclusions are made from the study presented within this addenda:

a) The BWST configuration without the Celotex fiberboard layer results in the allowable buckling stress and bolt chair top plate stress being exceeded when subjected to the combined Seismic Margin Study Earthquake and future settlement loadings.

b) The BWST configuration with a 1/2 inch Celotex layer between the tank bottom and ring wall is acceptable, and all design code stress allowables are met for combined Seismic Margin Earthquake and settlement loading.

Table A2-1: TANK MODEL WITH CELOTEX

STRESS COMBINATIONS - SME + DW + SETTLEMENT

Parameter	Seismic Response from SME (Faulted Condition)	DW + Settlement Response	Total Response	Allowable Response (See Section A3)
Compression in 3/8" Shell	853 psi	1,066 psi	1,919 psi	4,252 psi
Compression in 1/4" Shell	684 psi	1,164 psi	1,848 psi	2,834 psi
Local Membrane in Shell at Bolt Chair	8,332 psi	13,720 psi	22,052 psi	37,680 psi
Bolt Chair Top Plate Bending	25,120 psi	9,637 psi	34,757 psi	37,680 psi
Anchor Bolt Load	15.7 kips	6 kips	21.7 kips	57.4 kips

SME = Seismic Margin Earthquake

DW = Deadweight + Hydrostatic Pressure Loads

Table A2-2: TANK MODEL WITHOUT CELOTEX

STRESS COMBINATIONS - SME + DW + SETTLEMENT

Parameter	Seismic Response from SME (Faulted Condition)	DW + Settlement Response	Total Response	Allowable Response (See Section A3)
Compression in 3/8" Shell	853 psi	2,661 psi	3,514 psi	4,252 psi
Compression in 1/4" Shell	684 psi	2,973 psi	3,657 psi	2,834 psi
Local Membrane in Shell at Bolt Chair	8,332 psi	15,469 psi	23,801 psi	37,680 psi
Bolt Chair Top Plate Bending	25,120 psi	16,266 psi	41,386 psi	37,680 psi
Anchor Bolt Load	15.7 kips	10 kips	25.7 kips	57.4 kips

SME = Seismic Margin Earthquake

DW = Deadweight + Hydrostatic Pressure Loads

A3. ACCEPTANCE CRITERIA

The governing codes and standards for design of the Midland BWSTs are delineated within Section 3.2 of this report. Section 3.2 contains the ASME code stress acceptance criteria. The ASME code does not specify loading combinations. In order to assure satisfactory performance of the BWSTs in a seismic event, response to a load combination of dead weight plus settlement plus the seismic margin earthquake was compared to faulted condition allowable stress limits. The critical areas of the tank for stress analysis purposes are:

1) Local Membrane Stress in Vessel Wall
2) Vessel Wall in Compression (Buckling)
3) Bolt Chair Top Plate
4) Anchor Bolts

The specific acceptance criteria for each of these areas is addressed below.

A3.1 CRITERIA FOR BOLT CHAIR AND VESSEL WALL IN TENSION

The basic design criteria for the bolt chair and the vessel wall in tension is the 1974 ASME Code, Section III (Nuclear Power Plant Components), Subsection NC (Class 2 Components). In addition, ASME Code Case 1607-1 is applicable for upset, emergency and faulted condition stress allowables. Under these governing criteria the following principal stresses are allowed.

Loading Condition	Primary Membrane, σ_m	Primary Local Membrane plus Primary Bending, $\sigma_L + \sigma_b$
Design and Normal	1.0S	1.5S
Upset	1.1S	1.65S
Emergency	1.5S	1.8S
Faulted	2.0S	2.4S

S is an allowable stress and has a value of 15.7 ksi for 304L stainless steel. The Seismic Margin Study earthquake loads represent safe shutdown (SSE) loads, and, thus, must be compared to the faulted allowable stress limits. Secondary stresses do not require evaluation for Class 2 components designed by rule (NC 3300 criteria).

A3.2 CRITERIA FOR VESSEL WALL IN COMPRESSION

For the large diameter thin wall storage tank, buckling will occur in the elastic range. The ASME Code buckling criterion for axially loaded cylinders nominally contains a safety factor of 3 for sustained design loads. The ASME Code specifies in Article NC-3000 that the maximum allowable compressive stress to be used in the design of cylindrical shells shall be the lesser of:

a) The allowable S value given in Tables I-7.0.

b) The value of B determined from the applicable chart in Appendix VII.

For the case under consideration, the latter criterion governs.

This B value for elastic buckling in Appendix VII can be obtained with a higher degree of accuracy by using the design formula shown below, taken from the 1977 ASME Code.

$$B = \frac{0.0625\ E\ T}{R}$$

where

E = modulus of elasticity

T = shell thickness

R = inside radius of shell

This formula applies to the linear portion of the buckling curves in Appendix VII and is applicable for the BWST analysis. The buckling allowables for the design and normal loading conditions are therefore:

σ_{cr} = 1417 psi (for 1/4" shell)
σ_{cr} = 2126 psi (for 3/8" shell)

Based on Code Case 1607-1, the Faulted Condition allowables can be increased by a factor of 2.0 for primary membrane stresses. Thus, the buckling allowables for faulted conditions are:

σ_{cr} = 2834 psi (1/4" shell)
σ_{cr} = 4252 psi (3/8" shell)

A3.3 CRITERIA FOR ANCHOR BOLTS

The allowable load criteria for anchor bolt pullout of the concrete is based on the provisions of ACI 349-80 (Reference A7). An allowable bolt load of 136 kips has been conservatively calculated for concrete pullout. The allowable load for the failure of the bolt itself is based on the AISC criteria (Reference A6). Section 1.5.2.1 of the AISC Code states that for tension on the nominal bolt area, the allowable stress is 1/3 of the ultimate tensile stress. Further, Part 2 of the AISC Code states that an addition factor of 1.7 may be applied for the safe shutdown earthquake. Thus, for 1½ inch diameter A36 bolts with an ultimate tensile strength of 58 ksi, the allowable load is 57.4 kips. This bolt failure allowable of 57.4 kips will govern for loading of the anchor bolts.

A4. ANALYTICAL MODELS AND ANALYSIS METHODS

A finite element model was constructed to represent the cylindrical shell portion of the BWSTs. Boundary conditions were applied at the top and bottom of the model to represent the restraint offered by the umbrella roof, the tank bottom and the asphalt impregnated fiberboard between the ring wall and tank bottom. Analysis of bolt chairs and the tank wall adjacent to the bolt chairs was conducted by hand methods using empirical design analysis approaches and classical stress analysis techniques.

A4.1 FINITE ELEMENT MODEL

The basic tank model has already been described in Section 4.1. The finite element model generated for the end-of-life settlement loading differs from this original model only in the vertical boundary elements placed along the bottom of the tank. As Figure A1-2 illustrates, the interface between the tank bottom, the asphalt impregnated fiberboard (Celotex) and the ringwall have been modeled as a gap element and a spar element acting vertically at each of the 40 nodes located at the bottom of the tank. The spar elements model the 40 anchor bolts as linear springs. The gap elements represent the rigid ringwall for the case where the Celotex layer is not present, and they model the Celotex layer for the case where it is present.

The equivalent linear spring stiffness for the forty anchor bolts was calculated by adding the flexibilities of the bolt, the bolt chair top plate and the tank wall out-of-plane rotation due to the moment created by bolt chair loading. These three items act as springs in series, and their individual stiffnesses were calculated by classic strength of material equations and by the methods given within Reference A8. An equivalent spring stiffness of 1.1×10^5 lb/in was calculated and used for each of the 40 spar elements in the model. As a check on the derived bolt effective stiffness, the stiffnesses of each of the loaded bolts within the original BWST analysis were evaluated by dividing the measured bolt load by the calculated bolt stretching caused by the ring wall deflection relative to the tank bottom. All but one of these derived bolt stiffnesses were lower in magnitude than the 1.1×10^5 lb/in value used in the current model. This higher stiffness value is conservative since it results in higher bolt stresses and tank wall stresses.

The gap element stiffnesses, which represent the behavior of the ½ inch thick asphalt impregnated fiberboard, were determined using a beam on an elastic foundation analysis as reported in Section 4.1 of this report. For the model configuration without a Celotex layer, the gap elements were modeled as rigid once the gap has closed and the tank bottom meets the ring wall.

A4.2 FINITE ELEMENT MODEL LOADING

Loading conditions present in the BWSTs are:

1) Dead weight of the tank roof
2) Dead weight of the tank wall
3) Anchor bolt loading
4) Hydrostatic pressure acting radially on the tank
5) Hydrostatic pressure acting downward on an annulus of the bottom plate adjacent to the tank wall.

Loading conditions 1, 2 and 4 above are identical to the loads applied to the original tank settlement analysis, and their descriptions can be found in Section 4.2 of this report. The anchor bolt loadings had to be applied differently for this end-of-life settlement analysis since the actual bolt loads had to be computed rather than measured by use of strain gages as they were for the original tank settlement analysis. The tank is initially modelled with a planar bottom, and 40 preloaded springs attached at each of the anchor bolt locations. The preload value in each of these springs was determined by multiplying the bolt stiffness by the distance from the tank bottom to the deflected ring wall position beneath it. After these preloads, along with the deadweight loads and hydrostatic pressure loads, have been applied to the tank, the computer will iterate the model into an equilibrium position. Actual bolt loads can be obtained from this computer solution by subtracting the amount of relieving force occurring due to tank deflection from the original spring preload.

The deadload attributed to the hydrostatic pressure acting downward on an annulus of the bottom plate was determined from Figure 4-4, as in the previous analysis. For the model configuration without the Celotex layer, a conservative value of 100 lbs/in/in. of circumference was used for the water annulus weight. For the model configuration with the Celotex layer, the resultant gaps are smaller than for the case without Celotex. Thus, a conservative water annulus load of 90 lb/in/in. of circumference was used. This deadweight load was placed on those nodes where a gap still exists after equilibrium has been satisfied. The water load will be carried directly by the ring wall and soil beneath the tank bottom for those nodes which have contacted the Celotex layer.

The relative contributions of downward loading from deadweight, anchor bolt loads and vertical water pressure loading reacted by the shell are:

With Celotex Layer

Tank Roof		36,000 lbs
Tank Shell and Hardware		67,090 lbs
Effective Water Weight Reacted by Shell		70,560 lbs
Total of Bolt Loads		60,340 lbs
	TOTAL	233,990 lbs

Without Celotex Layer

Tank Roof		36,000 lbs
Tank Shell and Hardware		67,090 lbs
Effective Water Weight Reacted by Shell		132,300 lbs
Total of Bolt Loads		149,760 lbs
	TOTAL	385,150 lbs

From the vertical force summary, it can be seen that the anchor bolt loads are 26% and 39% of the total vertical load for the "with Celotex" and the "without Celotex" cases, respectively. Thus, a considerable amount of the load for each of these configurations could be avoided by periodically backing off the load developed on these bolts. This is not necessary, however, as the analysis results indicate that end of settlement loading combined with the seismic margin earthquake result in responses within the design code allowable limits.

A4.3 BOLT CHAIR MODEL

The bolt chair model is identical to that presented within Section 4.3 of this report.

A4.4 SEISMIC LOADING CONDITIONS

A seismic analysis of the Borated Water Storage Tank was conducted in addition to the soil settlement analysis. The earthquake excitation used for this seismic analysis is the ground response spectra developed for the Midland Seismic Margin Earthquake Structural Evaluation Program. The Seismic Margin Study response spectra consist of an envelope of the site specific spectra developed by Weston Geophysical Corporation (Reference A3) for structures founded at the top of fill, and Housner response spectra (Reference A4) which are anchored to a 0.12 peak ground acceleration. The seismic analysis for this loading condition is contained within Reference A5. A Seismic Margin Earthquake (SME) overturning moment of 8154 ft-kips and an SME base shear of 539 kips were taken from Reference A5 to develop the seismic stresses.

A5. ANALYTICAL RESULTS

Results from the ring wall future settlement analysis and the Seismic Margins Study Earthquake analysis are summarized in this chapter. Computer output is too voluminous to include, consequently, only important highlights of the output are summarized.

A5.1 RESULTS FROM THE FUTURE SETTLEMENT ANALYSIS

Because of the wave front solution techniques used in ANSYS, tank model elements in a column are in numerical sequence, while elements in a row (around the circumference) are numbered in steps of 24 (see Figure A1-2). The bottom 3 rows of elements representing the lower 12 feet of the tank contain the important stress results from the finite element model. The highest loading occurs in the bottom row of elements where the shell thickness is 0.375 inches. Since the bottom elements are restrained from radial movement by the tank bottom, hoop stresses in the second row of elements are greater than for the bottom row. At the junction of the second to third row of elements, the tank wall thickness changes from 0.375 inches to 0.25 inches. Above row 3, element stresses diminish with decreasing hydrostatic pressure and a more even distribution of the asymmetric vertical reaction loads.

The maximum compressive stresses in the tank wall can be taken directly from the computer printout. These maximum compressive stresses in both the 1/4 inch wall and the 3/8 inch wall are listed below for each of the tank model configurations.

Shell Thickness	Compressive Stress, psi No Celotex	With Celotex
1/4"	-2973 (Element 55)	-1164 (Element 55)
3/8"	-2661 (Element 57)	-1066 (Element 57)

Element 55 is located on the bottom row of the 1/4 inch elements, and element 57 is located on the bottom row of the 3/8 inch thick elements. Both of these elements are directly above the highest point on the settled ring wall contour (Node #338).

The bolt force and gap results from the computer analysis are tabulated in Table A5-1 and Table A5-2. Table A5-1 contains data for the tank configuration without Celotex and Table A5-2 contains data for the tank configuration with Celotex. The maximum bolt loads for the "with" and "without" Celotex configurations are 6 kips and 10 kips, respectively, as shown in these two tables. Figure A5-1 shows the resultant displacement of the tank bottom relative to the uncompressed fiberboard position for the tank model with the Celotex layer. The displacement plot incorporates the deflections of both the fiberboard and the tank wall. Maximum compression in the fiberboard is 0.09 inches.

The local membrane stress in the shell at the bolt chair location is the combined hoop stress due to hydrostatic pressure and anchor bolt loads. The hydrostatic pressure hoop stress is calculated at a depth of 31 ft to be 11,177 psi.

The localized hoop stress induced by anchor bolt loading was derived by linear scaling from previous analysis, i.e., future settlement stress is equal to the bolt load for future settlement divided by the bolt load for current settlement times the computed stress for current settlement from Section 5.0. The local membrane hoop stress due to deadweight plus settlement was then obtained by absolute summation.

The bending stress located in the top plate of the bolt chair is a linear function of the bolt load. Thus, the bending stresses in Table A2-1 and Table A2-2 were scaled from bolt load ratios times the stresses derived in Section 5.0.

A5.2 RESULTS FROM THE SEISMIC MARGIN STUDY EARTHQUAKE ANALYSIS

The complete results of the BWST seismic analysis using Seismic Margin Study loading conditions is documented in Reference A5. A brief summary of the results from Reference A5 which were utilized in this addenda will be included in this section.

Seismic Margin Earthquake (SME) overturning moments from Reference A5 were utilized to calculate compressive stresses in the tank shell. The maximum overturning moment is reported to be 8154 ft-kips in the 3/8 inch shell, and 4358 ft-kips in the 1/4 inch shell. Using standard bending stress equations from beam theory, stresses of 853 psi and 684 psi are calculated for the 3/8 inch shell and the 1/4 inch shell, respectively.

The reported local membrane stresses in the tank wall at the bolt chair are hoop stresses due to 2 different sources. The first source of hoop stress is the seismic induced bolt loads acting on the bolt chair which tend to want to stretch the shell in hoop membrane. This stress is scaled from Section 5.0 for the Seismic Margin bolt load computed in Reference A5 and is computed to be 6629 psi. The second source of hoop stress is made up of three pressure induced components. These three components are:

1) the hydrostatic pressure due to the 0.1g vertical earthquake which produce a hoop stress of 1118 psi. This stress was calculated by taking one tenth of the 11,177 psi hydrostatic pressure induced hoop stress calculated for a water column 31 feet above the bolt chair top plate.

2) sloshing induced pressure of 12.9 lb/ft^2 (Reference A5) which produces a 74.5 psi hoop stress. The sloshing induced pressure is caused by that portion of the tank water mass which moves in a sloshing motion.

3) convective induced pressure of 222 lbs/ft^2 (Reference A5) which produce a 1283 psi hoop stress. The convective induced pressure is caused by that portion of the tank water mass which moves as a rigid body.

These three pressure induced hoop stresses were combined by SRSS since they will generally be out of phase with each other and the resulting stress level is 1703 psi. The total hoop stress at the bolt chair location was conservatively calculated to be 8332 psi by absolute summation of the 6629 psi bolt load induced stress and the 1703 psi pressure induced stress.

The seismic-induced bending stress of 25.12 ksi in the bolt chair top plate was calculated by scaling results in Section 5.0 for predicted future settlement bolt loading.

A maximum anchor bolt load of 15.7 kips for the Seismic Margin Earthquake was taken directly from the calculations in Reference A5.

A5.3 TOTAL RESPONSE CALCULATIONS

The total stress response for the faulted condition was conservatively obtained by absolute summation of the SME induced stresses and the ring wall settlement (including deadweight and hydrostatic pressure) induced stresses.

Table A5-1

BOLT FORCE AND GAP RESULTANTS FOR

BWST CONFIGURATION WITHOUT CELOTEX LAYER

Node Number	Gap (Inches)	Bolt Force (lbs)
321	0	0
322	0.0037	407
323	0.0169	1,863
324	0.0312	3,432
325	0.0462	5,100
326	0.0619	6,810
327	0.0756	8,322
328	0.0863	9,499
329	0.0926*	10,190*
330	0.0906	9,974
331	0.0798	8,785
332	0.0597	4,566
333	0.0373	4,103
334	0.0134	1,473
335	0	0
336	0	0
337	0	0
338	0	0
339	0	0
340	0	0
341	0.0002	24
342	0.0174	1,916
343	0.0396	4,354
344	0.0622	6,850
345	0.0789	8,690
346	0.0872	9,600
347	0.0875	9,636
348	0.0827	9,096
349	0.0730	8,033
350	0.0600	6,606
351	0.0449	4,937
352	0.0297	3,269
353	0.0164	1,799
354	0.0039	426
355	0	0
356	0	0
357	0	0
358	0	0
359	0	0
360	0	0

*Maximum Value

Sum of the Bolt Force = 149,760 lbs.

Table A5-2

BOLT FORCE AND GAP RESULTANTS FOR

BWST CONFIGURATION WITH CELOTEX LAYER

Node*	Gap (Inches)	Bolt Force (lbs)
321	0	0
322	0	0
323	0	0
324	0	0
325	0.0094	975
326	0.0252	2818
327	0.0399	4383
328	0.0500	5501
329	0.0549*	6037*
330	0.0506	5567
331	0.0368	4028
332	0.0122	1393
333	0	0
334	0	0
335	0	0
336	0	0
337	0	0
338	0	0
339	0	0
340	0	0
341	0	0
342	0	0
343	0	0
344	0.0158	1771
345	0.0366	4032
346	0.0478	5292
347	0.0507	5577
348	0.0470	5194
349	0.0381	4185
350	0.0246	2700
351	0.0080	890
352	0	0
353	0	0
354	0	0
355	0	0
356	0	0
357	0	0
358	0	0
359	0	0
360	0	0

*Maximum Valve

Sum of the Bolt Forces = 60,343 lbs.

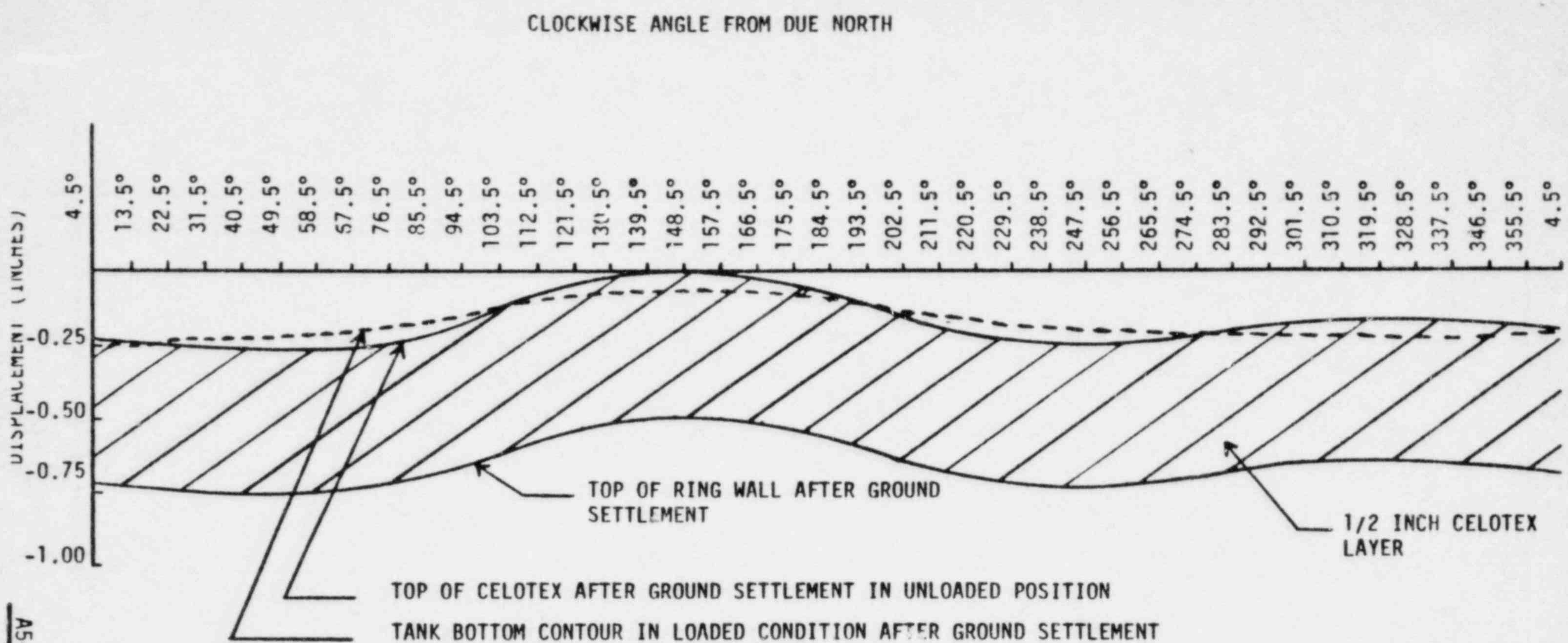


FIGURE A5-1. DISPLACEMENT OF TANK BOTTOM RELATIVE TO RINGWALL AND CELOTEX CONTOURS AFTER GROUND SETTLEMENT

REFERENCES

A1. Tabulated Data presented to R. D. Campbell during January 13, 1982 meeting between Consumers Power Company, Bechtel Power Corportion, Nuclear Regulatory Commission and Structural Mechanics Associates held at NRC Headquarters, Phillips Building in Bethesda, Maryland.

A2. "Testimony of Alan J. Boos and Dr. Robert D. Hanson on behalf of the Applicant Regarding Remedial Measures for the Midland Plant Borated Water Storage Tank", Atomic Safety and Licensing Board Docket Numbers 50-329 OM, 50-330 OM, 50-329 OL, and 50-330 OL, February 16, 1982.

A3. "Site Specific Response Spectra Midland Plant - Units 1 and 2, Part II - Response Spectra Allicable for the Top of Fill Material at the Plant Site", Weston Geophysical Corporation, Westboro, Massachusetts, May 1, 1981.

A4. "Nuclear Reactors and Earthquakes", TID-7024, prepared by Lockheed Aircraft Corporation and Holmes & Narver, Inc., for the Division of Reactor Development, U.S. Atomic Energy Commission, Washington, D. C., August, 1963.

A5. "Midland Safety Margin Earthquake Structural Evaluation of the Borated Water Storage Tank", SMA Report #13701.01-R002, prepared for Consumers Power Company, April, 1982.

A6. Manual of Steel Construction (Seventh Edition), American Institute of Steel Construction, Inc., 1973.

A7. ACI 349-80. Code Requirements for Nuclear Safety Related Concrete Structures, American Concrete Institute, April, 1981.

A8. Bijlaard, P. P., Stresses From Radial Loads and External Moments in Cylindrical Pressure Vessels, Welding Journal Research Supplement, December, 1955.

APPENDIX A

TEST LABORATORY RESULTS FROM COMPRESSION TESTING OF ASPHALT IMPREGNATED FIBERBOARD (CELOTEX)

RICHMOND 9-3411

FORM P-1 1-42 L

SMITH-EMERY COMPANY

CHEMISTS · TESTING · INSPECTION · ENGINEERS

781 EAST WASHINGTON BOULEVARD · LOS ANGELES, CALIFORNIA 90021 · (213) 749-3411
3148-D LA PALMA AVENUE · ANAHEIM, CALIFORNIA 92806 · (714) 630-4910

FILE NO.: 8246
LABORATORY NO.: 81-809

DATE: October 16, 1981

Structural Mechanics Associates
5160 Birch Street
Newport Beach, California 92660

RECEIVED OCT 2 0 1981
13704.01C003

Attention: Greg S. Hardy

SUBJECT: COMPRESSION TESTS ON 1/2" THICK CELOTEX FIBERBOARD MATERIAL

SOURCE: Submitted to our Laboratory

ITEM Q

REPORT OF TESTS

In compliance with your request, we have conducted compression tests on 3-inch by 3-inch, and 3-inch by 6-inch specimens of Celotex fiberboard cut from a 6-inch by 11½-inch by 1/2-inch thick sample submitted to our laboratory.

The tests were conducted in a universal testing machine with the specimen laid flat. Rigid steel plates slightly larger than each specimen were placed on the top and bottom of the specimen to assure full and uniform loading over the surface of the specimen. Dial indicators having 0.001 inch accuracy were placed on each side of the specimen, evenly spaced from the center of the specimen to measure the deformation of the specimen.

A load was applied to the specimen and the corresponding average deformation reading recorded. The load was then released and the deformation again recorded. The load was increased and the deformations again recorded, followed by unloading. This sequence was continued with increasing loads until a reasonable curve could be established and the deformation exceeded 50% of the original thickness.

Load deflection readings are shown on the attached Plates A and B with accompanying load-deformation curves.

File No. 8246

Laboratory No. 81-809

Structural Mechanics Associates
CELOTEX FIBERBOARD COMPRESSION TESTS

October 16, 1981

We are also enclosing a copy of the most recent calibration of our Tinius-Olsen universal testing machine used for these tests.

Respectfully submitted,

SMITH-EMERY COMPANY

By

PAUL LINSTROM
Civil Engineer

2-Addressee

Attachments
PL:ks

SMITH-EMERY COMPANY

File No. 8246 Laboratory No. 81-809

STRUCTURAL MECHANICS ASSOCIATES October 16, 1981

CELOTEX COMPRESSION TEST

Sample Size: 3" x 3" x 1/2"

Load, Lbs.	Deflection, In.	Load, Lbs.	Deflection, In.
0	.000	1000	.151
240	.050	0	.083
0	.017	1500	.184
300	.066	0	.108
0	.026	2000	.210
350	.068	0	.129
0	.029	2500	.232
400	.075	0	.151
0	.034	3000	.251
450	.083	0	.173
0	.036	3500	.264
500	.090	0	.190
0	.039	4000	.274
550	.097	0	.200
0	.046	4500	.295
600	.104	0	.235
0	.048	5000	.307
650	.110	0	.243
0	.055	5500	.312
700	.120	0	.249
0	.057	6000	.316
750	.124	0	.251
0	.063	7000	.323
800	.129	0	.267
0	.070	8000	.334
850	.134	0	.277
0	.070	9000	.342
900	.144	0	.291
0	.078	Set After 5-Minutes:	
950	.147	0	.274
0	.081		

PLATE "A"

SMITH-EMERY COMPANY

File No. 8246 | Laboratory No. 81-809

STRUCTURAL MECHANICS ASSOCIATES | October 16, 1981

CELOTEX COMPRESSION TEST

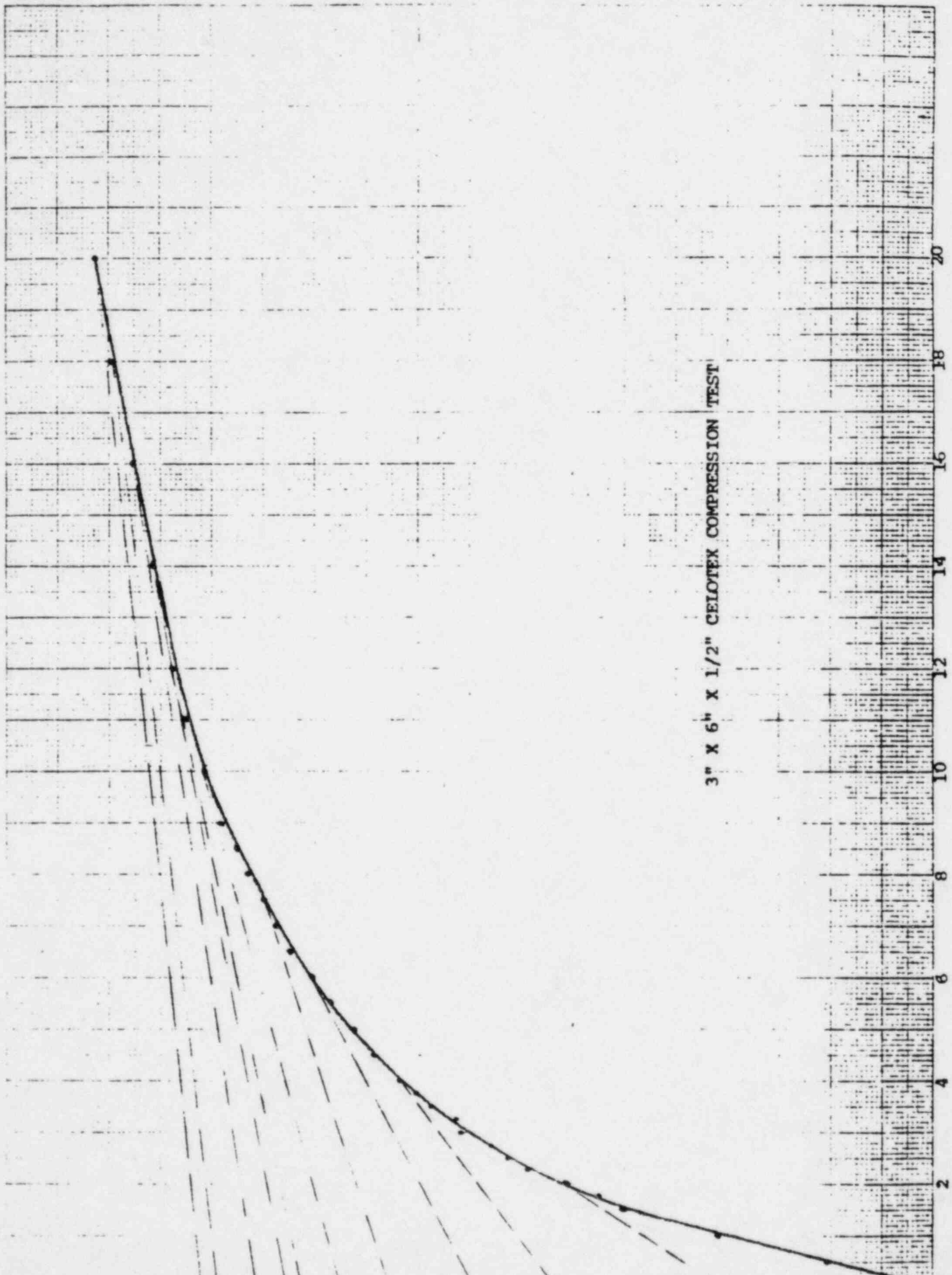
Sample Size: 3" x 6" x 1/2"

Load, Lbs.	Deflection, In.	Load, Lbs.	Deflection, In.
0	.000	5,500	.234
0	.000	0	.172
500	.041	6,000	.241
0	.017	0	.179
1000	.084	6,500	.250
0	.045	0	.189
1500	.120	7,000	.255
0	.060	0	.198
1750	.130	7,500	.260
0	.072	0	.203
2000	.143	8,000	.267
0	.083	0	.212
2250	.158	8,500	.271
0	.095	0	.217
2500	.166	9,000	.277
0	.102	0	.223
2750	.173	10,000	.283
0	.111	0	.231
3000	.184	11,000	.290
0	.118	0	.237
3250	.186	12,000	.295
0	.126	0	.245
3500	.196	14,000	.303
0	.131	0	.253
3750	.201	16,000	.311
0	.139	0	.262
4000	.208	18,000	.319
0	.148	0	.278
4500	.218	20,000	.325
0	.153	0	.284
5000	.225	Set After 5-Minutes:	
0	.162	0	.270

PLATE "B"

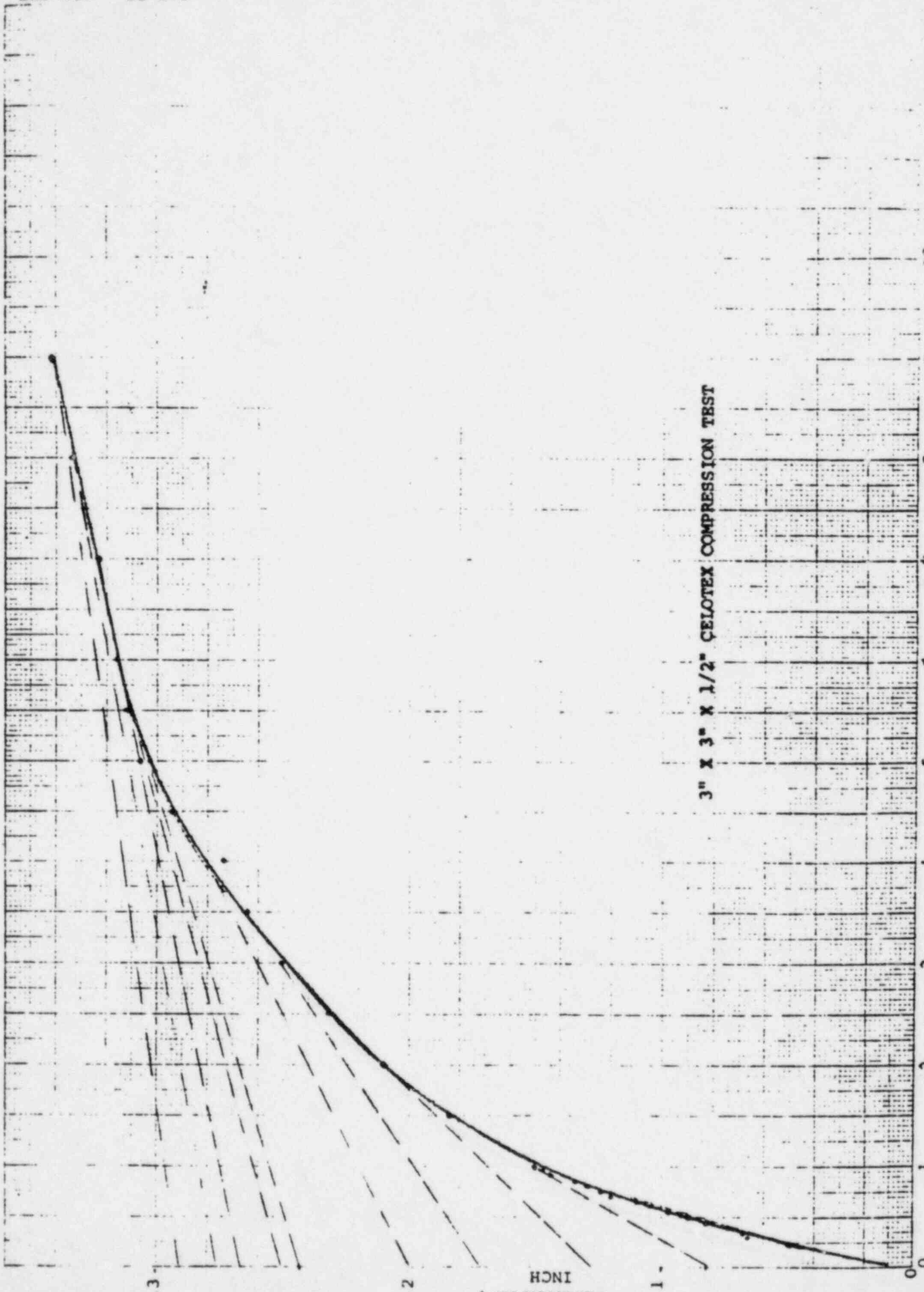
File No. 8246
Lab No. 81-809

SMITH-EMERY COMPANY



File No. 8246
Lab No. 81-809

SMITH-EMERY COMPANY



UNITED CALIBRATION CORP.

12761 MONARCH STREET · GARDEN GROVE, CALIFORNIA 92641 PHONE (714) 893-1821 · 638-2322

CERTIFICATE OF CALIBRATION

FOR: - SMITH EMERY
Anaheim, Ca.

TYPE OF MACHINE OR APPARATUS
T.O. SUPER "L" U.T.M.
CAP: 120,000 Lbs. S/N 80700-4

AMBIENT TEMPERATURE 68°F

DATE February 26, 1981

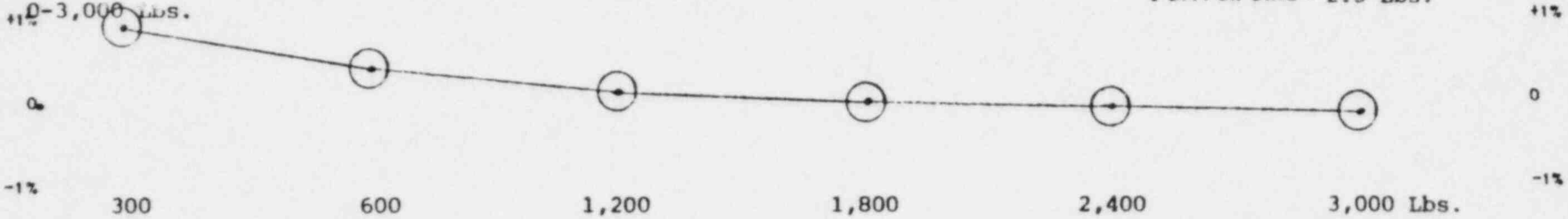
S.O. NO.

P.O. NO.

B-7

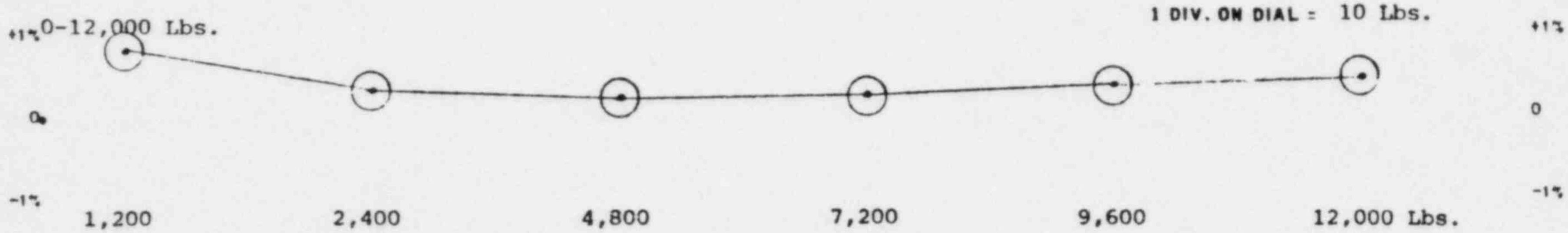
0-3,000 Lbs.

1 DIV. ON DIAL = 2.5 Lbs.



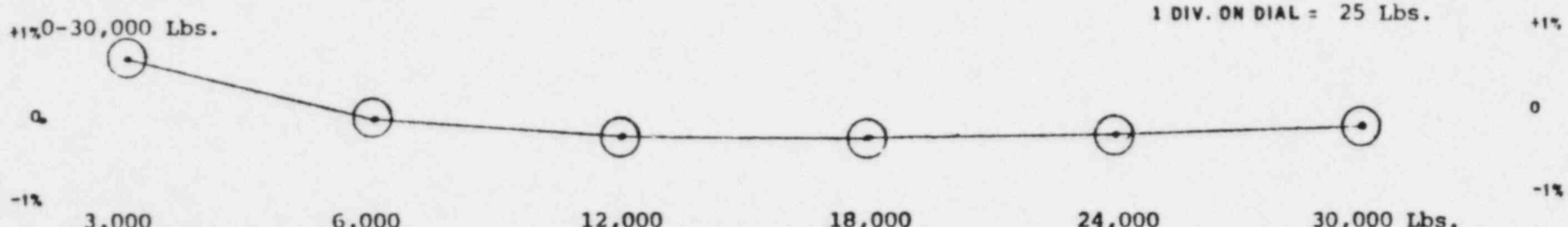
0-12,000 Lbs.

1 DIV. ON DIAL = 10 Lbs.



0-30,000 Lbs.

1 DIV. ON DIAL = 25 Lbs.



CALIBRATION APPARATUS USED
N.B.S. TRACEABLE MOREHOUSE PROVING RINGS calibrated
by National Standards Test Lab:
100,000 Lbs., Serial #1431, 3/29/79, 1/20th of 1%, NBS #SJT.Ø1/101468.
20,000 Lbs., Serial #2820, 3/21/79, 1/20th of 1%, NBS #SJT.01/101469
2,000 Lbs. Serial #3010, 3/22/79, 1/20th of 1%, NBS #SJT.01/101469.

Method Used
☒ ASTM E4-79
☐ ______________

Machine Meets
XX E4-79
XX ± 1%

Field Service Engineer
Walter I. Brown

Technical Director
Jere H. Watson