DESIGN OF THE RIVER BEND STATION CONTAINMENT WITH CONCRETE ANNULUS FILL

GULF STATES UTILITIES COMPANY
Beaumont, Texas

Docket Nos.50-458
and 50-459

STONE & WEBSTER ENGINEERING CORPORATION

Cherry Hill Operations Center
Cherry Hill, New Jersey

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Prepared for
Gulf States Utilities Company
Beaumont, Texas

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1.0 INTRODUCTION

The River Bend Station Containment is a free-standing steel cylindrical shell, 1 1/2 inches thick, with a torispherical dome (Figure 1). There are presently 4 circumferential and 108 vertical T-sections welded on the outside of the steel shell to add stiffness in the lower 20 ft. During 1981, it was calculated that dynamic responses of the Containment due to hydrodynamic loadings resulting from Safety Relief Valve (SRV) discharge and from Loss-of-Coolant Accident (LOCA) events in the suppression pool require significant reductions to permit economical qualification of piping systems and equipment supported off of the Containment. Although the Containment structure is qualified under the applied loads, it has been decided that the Containment be modified to reduce dynamic acceleration of piping systems by placing concrete in the 5-ft annulus space in the lower 25-ft region between the Containment and the outside Shield Building. This report describes these modifications and provides the planned design details for this concrete annulus fill.

2.0 DESCRIPTION OF THE CONTAINMENT

Details of the Containment structure in the suppression pool area are shown in Figure 2 (reference Section 3.8.2 of the River Bend Station RBS-FSAR). The modification will consist of adding reinforced concrete in the annulus between the Containment vessel and the Shield Building to a height of 25 ft above the top of the basemat, as shown in Figure 2.

3.0 DESIGN OF THE CONCRETE ANNULUS FILL

3.1 DESIGN PARAMETERS

The approach used in the design of the modification is that the Containment vessel, the fill concrete, and the Shield Building wall will act compositely. This leads to the greatest structural stiffness and produces the greatest reduction in dynamic responses of the Containment vessel due to the hydrodynamic loads.

3.1.1 Governing Codes

The basis for the design modification is ASME Code Case N- 258 (Reference 5.3 and the Attachment to this report), which addresses this type of modification. The Containment vessel, although acting compositely with the concrete, must still meet the allowable stress values of ASME, Section III, Division 1. The concrete portion must meet the requirements of ASME, Section III, Division 2 (Reference 5.2).

Design for the transfer of interface stresses across the joint between the existing Shield Building wall and the fill concrete is based on the use of Chapter 17 of ACI 318-77 (Reference 5.4). This procedure is used because it provides criteria for composite concrete design not specified by Reference 5.2.

3.1.2 Loads and Loading Combinations

The Containment structure is subjected to a variety of loads, including dead loads, live loads, hydrostatic pressure from the suppression pool, design pressure, accident or operating temperatures, earthquake loads, and hydrodynamic loads. The hydrodynamic loads are dynamic loads resulting from a blowdown into the pool due to a Safety Relief Valve (SRV) discharge and/or Loss-of-Coolant Accident (LOCA) events. For design of the concrete fill, these loads are combined as specified by ASME, Section III, Division 2 for concrete containments. These combinations are identical to those used for the drywell wall and are listed in Section 3.8.3 of the RBS-FSAR. The three most critical combinations are shown in Figure 3.

3.2 METHODS OF ANALYSIS

The Containment structure is analyzed as an axisymmetric shell using a finite element method. The lower concrete-fill portion is given the properties of a composite section, consisting of the stiffened steel shell, the fill concrete, and the Shield Building. Above the concrete fill, individual shells representing the Containment vessel and Shield Building are used (see Figure 4).

The effect of the mat is accounted for by applying discontinuity shear and moments at the bottom of the finite element model. These boundary effects are calculated by considering the interaction of the Containment and the basemat and enforcing conditions of equilibrium and compatibility between them.

For the dynamic analysis of the finite element model under axisymmetric loads, properties of the concrete elements are considered as orthotropic to account for the amount of cracking of the concrete in the vertical and circumferential directions. The properties therefore are dependent upon the state of stress in the structure.

For the dynamic analysis of the finite element model under asymmetric loads only, uncracked sections are considered in the analysis, and cracking is accounted for in the design of individual sections. For the analysis of loads involving a combination of axisymmetric and asymmetric loads, the procedure described above for axisymmetric loads is utilized. The mechanical loads, such as dead load or pressure, are applied directly to the shell. Thermal loads are also considered. In the lower portion, where the steel heats up much more than the concrete, an equivalent temperature increase that produces the same effect on the composite section is used. An equivalent linear gradient is used which produces the same thermal moment in the cracked composite section as the actual gradient. A typical gradient for one design condition is shown in Figure 5.

3.3 RESULTS OF THE ANALYSIS

The results of the analysis of the finite element model under the applied loads consist of in-plane forces and out-of-plane shears and moments. Forces and moments in the vertical direction are shown in Figures 6 and 7 for several loading combinations. Shear stress values for three critical load combinations are shown in Figures 8 to 10. Figure 11 shows the results of tangential shear forces in the concrete fill area for the worst loading combination, while Figure 12 shows average tangential shear stresses in the steel Containment and the concrete in the suppression pool.

3.4 DESIGN DETAILS AND PROCEDURES

Design details are as shown in Figure 13.

3.4.1 Axial Forces and Moments

Reinforcing steel is provided in the vertical and circumferential directions to resist the in-plane axial forces and out-of-plane bending moments in the fill. The stresses in the reinforcing and concrete are limited by ASME, Section III, Division 2 (Reference 5.2). It should be noted that the properties of the composite section are dependent on the amount of reinforcing in the concrete fill. Therefore, several iterations of calculations were performed before arriving at the final design.

3.4.2 Shears

3.4.2.1 Radial Shear

Out-of-plane radial shear stresses are resisted by the concrete in combination with stirrups in the fill concrete as shown in Figure 13. For conservatism, only the concrete in the fill area is assumed to resist the shear carried by the entire composite section and stirrups are sized accordingly.

3.4.2.2 Tangential Shear

Tangential shear forces shown in Figure 11 are resisted by the composite section made up of the steel Containment vessel, the concrete fill, and the Shield Building. It can be seen that the average tangential shear stress (Figure 12) in the concrete is less than the allowable shear stress in concrete, Vc, of 60 psi for abnormal/extreme environmental condition (Reference 1); therefore, no shear reinforcement is required.

3.4.2.3 Interface Shear Stress

As discussed in Section 3.1.1, design of the joint between the Shield Building and the concrete fill to transfer interface shear stresses given in Figures 8, 9, and 10 is based on Chapter 17 of the ACI 318-77 Code (Reference 5.4).

The surface of the Shield Building is roughened in accordance with the ACI Code provisions, and 3/4-in diameter Maxi-Bolts are provided at an average spacing of 23 inches in the circumferential direction and 24-in spacing in the vertical direction (total number: 2, 795) (Figure 14) to satisfy the minimum amount of steel required by the ACI 318-77 Code (Section 17.5.4.3). The allowable shear capacity of the roughened surface with minimum ties (Equation 11-14 of the ACI 318-77 Code) is 298 psi, which is well above the peak shear stress of 240 psi.

Based on the average shear stress of 80 psi, there is a factor of safety of approximately 3.7, as shown in Figure 17.

Based on the amount of the steel, Av, provided by the Maxi-Bolts, the steel yield stress required is 81.9 ksi using Equation 11-14 of the ACI 318-77 Code. The minimum yield stress of the material of the Maxi-Bolts is 105 ksi, which is more than the 81.9 ksi required. Details of the Maxi-Bolts, including the design capacity and the tests performed to support the adequacy of the performance and the function of these bolts to meet the design requirements, are provided in Section 3.5.

It is to be noted that the Maxi-Bolts described in Section 3.5, with their bearing-type engagement on the concrete provide a positive anchorage system to tie the Shield Building and the concrete fill.

3.4.3 Transfer of Stresses to Basemat

The forces in the composite shell are transferred to the mat in the following manner. Vertical stresses and tangential shear stresses are transferred by the extension of Shield Building vertical reinforcing into the basemat (Figure 15) and an embedment plate at the base of the Containment vessel that is anchored into the basemat (Figure 16). Radial shear stresses are transferred through a shear key cut in the mat and shear lugs welded to the bottom of the steel Containment embedment plate.

The tangential shear stress of approximately 40 psi between the concrete fill and the basemat is transferred through the roughened surface between the concrete fill and the mat concrete (Figure 13).

3.5 DESIGN FEATURES USING DRILLCO MAXI-BOLTS

3.5.1 Description of the Maxi-Bolt

The Drillco Maxi-Bolt is a ductile bearing-type anchor and has been introduced recently in the nuclear industry.

The Maxi-Bolt derives its name from the basic concept of developing maximum bolt capacity and maximum ductility. The Maxi-Bolt is a bearing anchor bolt designed specifically to comply with the requirements of Section B.7.1 of Appendix B, Steel Embedments, of ACI 349-76 (Reference 5.5). The Maxi-Bolt is made of ASTM A193 (Grade B) steel material (f_{ult}=125 ksi). Tests have been conducted to verify that the Maxi-Bolt will consistently develop the minimum specified tensile stress of the bolt and develop full ductility of the bolt to provide a favorable plastic stretch over the length of the bolt. Figure 18 explains the configuration and the components of Maxi-Bolts.

The Maxi-Bolt is installed in a predrilled hole with a conical counterbore at a predetermined depth so that the tapered nut at the end of the bolt bears against this counterbored surface. The conical hole and the nut are designed to transfer the bolt tension load into direct bearing stress between the conical nut and the expansion sleeve and between the expansion sleeve and conical hole concrete surface. This design does not depend upon the lateral expansion of a mechanism onto a concrete surface parallel to the direction of the bolt tension load and thus differs significantly from other wedge and sleeve type drilled-in expansion anchors, which depend on friction between concrete and the bolt to transmit the load. By utilizing the sloped concrete surface of the Maxi-Bolt design, the bearing area (contact surface) for the Maxi-Bolt can be duplicated for each installed anchor and will therefore produce consistent test load characteristics, performance, and capacity for any given concrete strength.

3.5.2 Description of Usage

The Maxi-Bolts will be used to tie the annulus fill concrete to the Shield Building to provide a fully composite section as described in Section 3.4.

3.5.3 Installation Procedure

Installation of this anchor is achieved by first drilling a primary hole with a drill that has very close dimensional tolerance. The bottom of this hole is undercut into a conical shape by use of a special expanding drill bit. After the drilling is completed, holes are cleaned of concrete cuttings, dust, and foreign material. The Maxi-Bolt is then inserted into the hole. The bolt is then pretensioned to 50 percent of the yield strength load. At this time, the wedge of the bolt has been fully expanded into the precut conical shape. Final tensioning is done to 81 percent of the yield strength of the bolt material by either hydraulic tensioning or torquing the hexagonal nut. The installation is complete at this point.

The installation process described above expands the bottom of sleeve B (Figure 18) and seats the Maxi-Bolt on the predrilled conical concrete shape of the hole bottom. The bolt load is transferred through the sleeve by bearing against the concrete. The depth of the primary hole is designed to develop a cone in concrete that will develop pullout capacity at least equal to the ultimate bolt tensile capacity. This ensures that the bolt will perform in a completely ductile manner.

3.5.4 Results of Test Programs

3.5.4.1 Tests at River Bend Station

To verify the performance and capacity of the Maxi-Bolts for use at River Bend Station, the following test program has been initiated:

1. In January 1982, 3/4-inch diameter anchor bolts were tested at the River Bend Site. During this testing, the bolts were installed as described in Section 3.5.3, but prior to tension testing, the pretension force was removed. This duplicated the worst condition that could be expected during plant life. The results exhibited an excellent correlation between deflection and applied tension. The load deflection curve is enclosed in

Figure 19. In all cases, the mode of failure was a ductile failure of bolts.

2. Additional tests for other diameters of Maxi-Bolts, similar to the ones described above, are scheduled for mid-May 1982.

3.5.4.2 Tests at Othe tions

Extensive tests have been performed on Drillco Maxi-Bolts at other locations. A summary of the results of static and dynamic tests (for tension and shear loads) performed at the University of Tennessee, Tennessee Valley Authority (TVA), Pittsburgh Testing Laboratory, and Rockport Generating Station is enclosed (the Appendix of this report). Results of static and dynamic tests indicate that failure occurred in the bolts at loads exceeding the specified ultimate load-carrying capacity of the bolt. Not a single Maxi-Bolt failed prematurely due to slippage or due to any sort of malfunction of the anchorage mechanism. It was concluded that the anchorage mechanisms of the Maxi-Bolts tested were adequate to develop the full strength of the bolt in tension or in shear and that the failure mode is a ductile one. Additional static and dynamic tests performed at the University of Tennessee (Reference 5.6) on various types of anchors, including the Maxi- Bolts, in cracked concrete indicated clearly that the Maxi-Bolts performed exceptionally well under the most rigorous test conditions.

3.5.5 Design Capacity of Maxi-Bolts

3.5.5.1 Evaluation of IE Bulletin 79-02

NRC Bulletin IE 79-02, Revision 2, was issued in November 1979 and has imposed constraints on the use of drilled-in wedge and sleeve type expansion anchor bolts so that if they are used, a safety factor of 4 must be provided. Until 1979, the expansion-type anchors were the only drilled- in anchor bolts available for use in the power plant construction.

Drillco's Maxi-Bolt anchor is, in contrast to the expansion-type designs, a bearing-type anchor and is designed using provisions of Appendix B, Steel Embedments, of ACI 349-76. Key design features of the design and performance are outlined as follows:

1. Design Practice

   Loads for Drillco anchors are calculated using the procedure outlined in Appendix B of ACI 349-76. Proper reduction, if applicable, in allowable stresses of anchor bolts for cyclic loading is made to avoid fatigue failures in the bolt material.

   An extensive amount of testing for static and dynamic loads has been done (Appendix), and it has been established that the failure of the Maxi-Bolt material takes place at or above the Maxi-Bolt's specified ultimate capacity and that it does not pull out at failure, which assures that the mode of failure is ductile. Therefore, the anchor can be designed in accordance with the requirements of ACI 349-76.

2. Installation

Installation of these bolts will be in accordance with procedures as described above. Inspection criteria have been developed by the Engineers for monitoring the installation under the River Bend Station QA program.

3. Material

The material used for Drillco anchors is ASTM A193, Grade B, which has a minimum yield stress of 105 ksi and a minimum ultimate strength of 125 ksi. The bolts will be procured in accordance with a QA program that meets the intent of 10CFR50, Appendix B.

4. Slippage

The means by which this bolt transfers loads to concrete is by bearing against the concrete. All tests conducted to date have shown no bolt failure due to slippage during either static or cyclic loading.

In view of the above, the Maxi-Bolts are not subject to the requirements of the IE 79-02 Bulletin, but are correctly governed by the requirements of Appendix B of ACI 349-76.

3.5.5.2 Capacity of Maxi-Bolts

The design pullout capacity of Maxi-Bolts is calculated in accordance with Appendix B, Steel Embedments, of ACI 349-76, as shown as follows:

Pure tension P_t in kips = 0.81 (lesser of fy or 0.8 fult) x Ab

where fy = minimum yield stress of bolt, 105 ksi
fult = minimum ultimate stress of bolt, 125 ksi
Ab = area of bolt, in^2

Bolt capacity and other data are listed in Table 1.

TABLE 1

PULLOUT CAPACITY OF MAXI-BOLTS

Diameter of Bolt (inches)	Area of Bolt (sq inches)	ACI 349-76 Appendix B (capacity in kips)	Ultimate Bolt (capacity in kips)
1/2	0.142	11.5	17.8
5/8	0.226	18.3	28.3
3/4	0.334	27.0	41.8

3.5.6 Verification of Installed Maxi-Bolts

To ensure that the variability of concrete in the zone of the shear cone will not adversely affect the capacity of these bolts, we have taken the following steps:

1. The Maxi-Bolt is subjected to a stress of 0.81 fy during the installation, thereby assuring the adequate strength of the concrete in the undercut area and verifying that the Maxi-Bolt will not pull out of the concrete.

2. The River Bend Project adheres to stringent quality control measures and techniques in mixing, placing, and curing of concrete.

3. The design capacity of the theoretical shear cone is based on capacity reduction factors in accordance with ACI 349-76, which take into account the possibility of variation in the strength of concrete.

4. Since the theoretical shear cone covers a large area of concrete around the bolt, the variability of concrete strength averages out over the total area of the shear cone and minor variations are not significant.

5. To demonstrate that the adequate concrete shear cone capacity exists, a statistical sample of a minimum of 125 (Reference 5.7) bolts in installed condition will be randomly selected and tensioned to 0.81 fy, with supports on concrete surface outside the theoretical shear cone area.

Stone & Webster Engineering Corporation believes that the above measures will adequately demonstrate that the variability of concrete in the shear cone area will not adversely affect the capacity of the Maxi-Bolts and will ensure that the Maxi-Bolts will function as designed.

4.0 SUMMARY AND CONCLUSIONS

As explained in detail in Sections 1.0 through 3.0, the design of the concrete fill and the method of tying the existing Shield Building with

Maxi-Bolts are conservative and provide adequate assurance that the structure will perform satisfactorily as designed, meeting the safety requirements of the Containment design.

5.0 REFERENCES

5.1 Standard Review Plan, Section 3.8.1 Concrete Containment, Revision 1, dated July 1981, issued by the United States Nuclear Regulatory Commission.

5.2 ASME Boiler and Pressure Vessel Code, Section III, Division 2, Code for Concrete Reactor Vessels and Containment, July 1, 1980, American Society of Mechanical Engineers.

5.3 ASME Code Case N-258 (Attachment to this Report).

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

5.5 Code Requirements for Nuclear Safety Related Concrete Structures, American Concrete Institute (ACI) Standard 349-76, 1976.

5.6 Expansion Anchor Performance in Cracked Concrete, Journal of the American Concrete Institute, November - December 1981, pages 471 through 479.

5.7 Sampling Procedure and Tables for Inspection by Attributes, Military Standard, MIL-STD-105D, April 1963, U.S. Department of Defense, U.S.A.

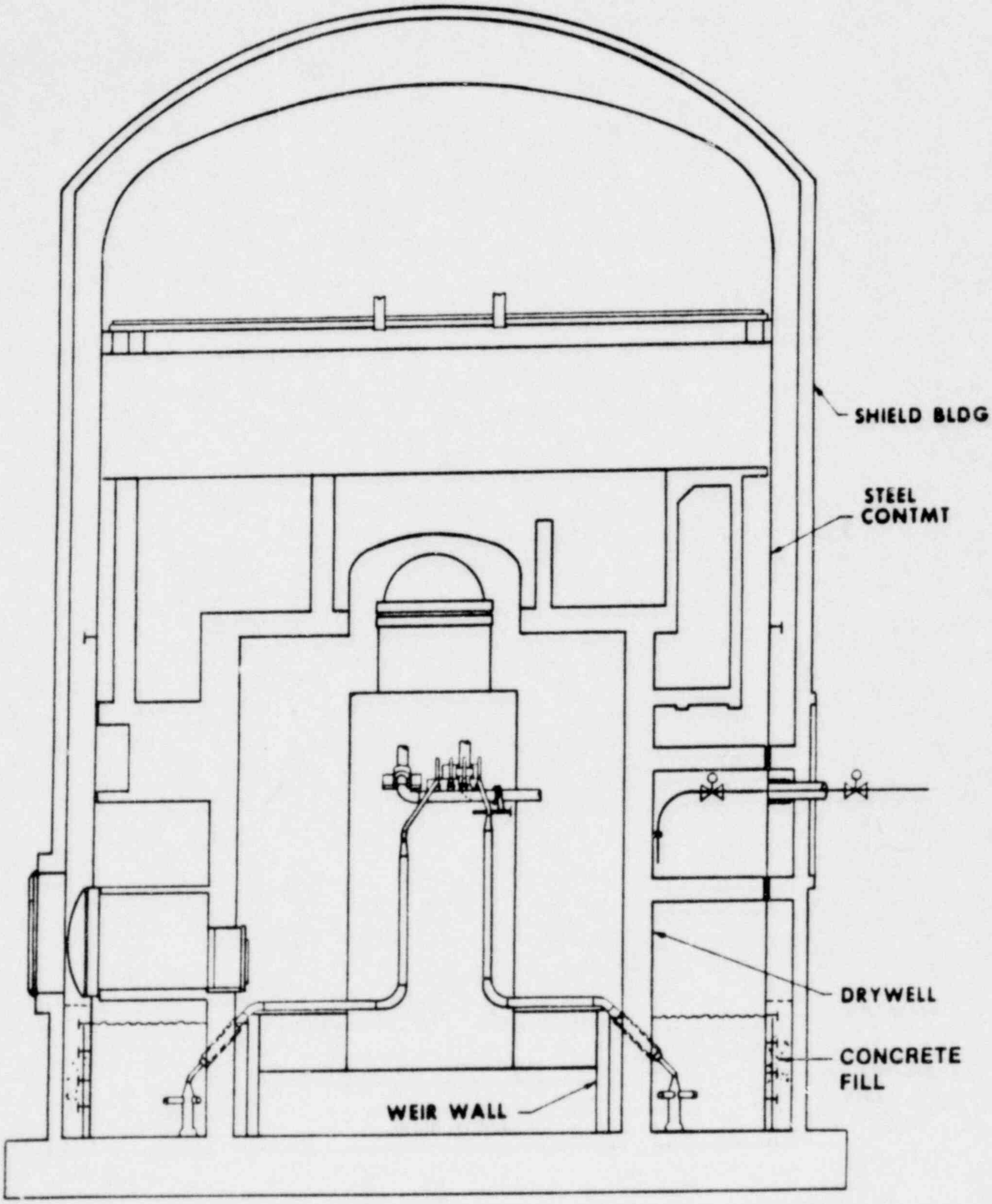


FIG. 1 RIVER BEND STATION MARK III CONTAINMENT

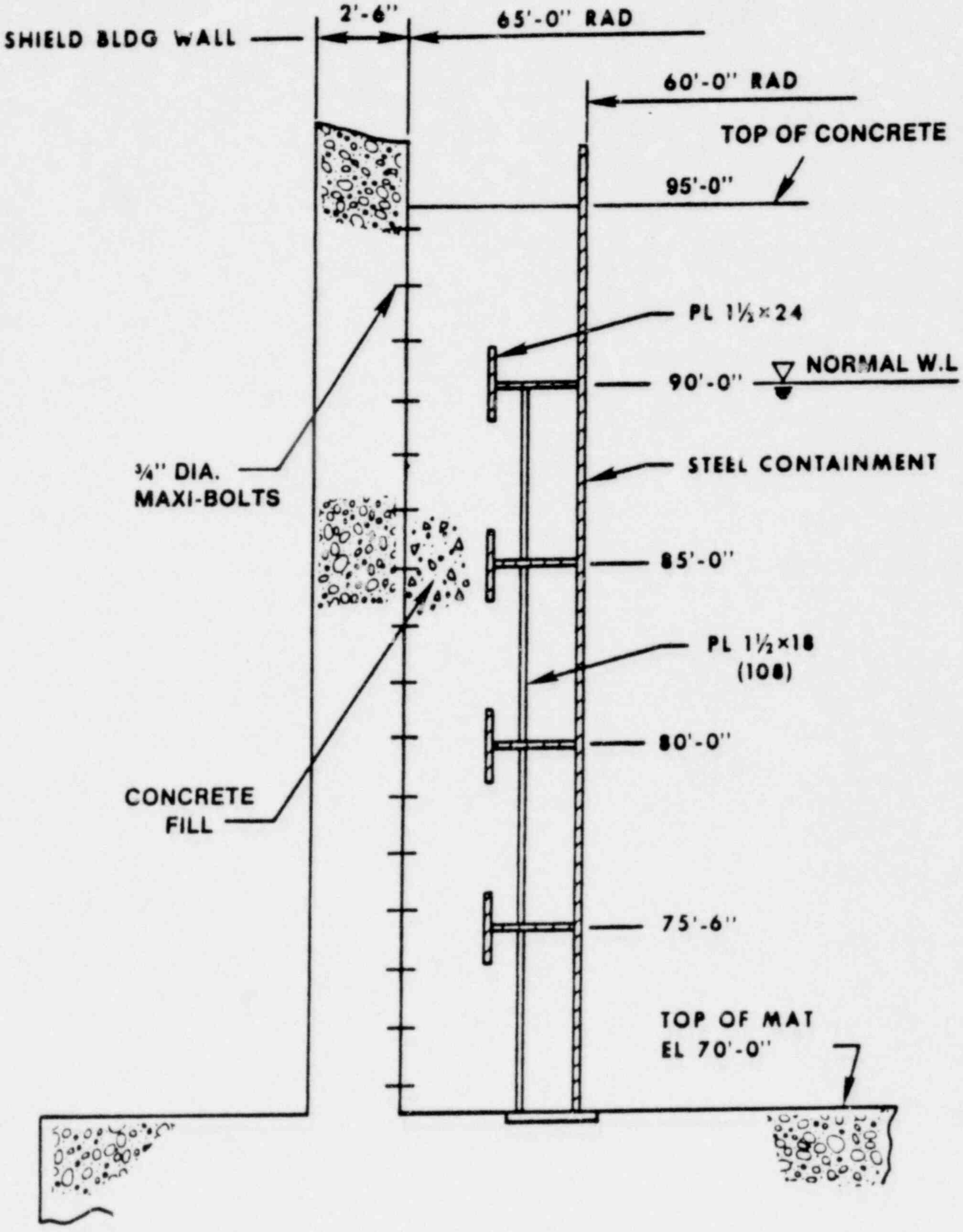


FIG. 2 DETAILS OF THE CONTAINMENT IN SUPPRESSION POOL

REFERENCE: FSAR SECTION 3.8.3.3.1

13.1 $D+L+Pa_1+Ta_1+SSE+LOCA+SRV_2+Ra+Rm+Rj+Rr$

13.2 $D+L+Pa_2+Ta_2+SSE+LOCA+SRV_3+Ra+Rm+Rj+Rr$

8.2 $D+L+1.5\ Pa_2+Ta_2+1.5\ LOCA+1.25\ SRV_3+Ra$

FIG.3 CRITICAL LOAD COMBINATIONS

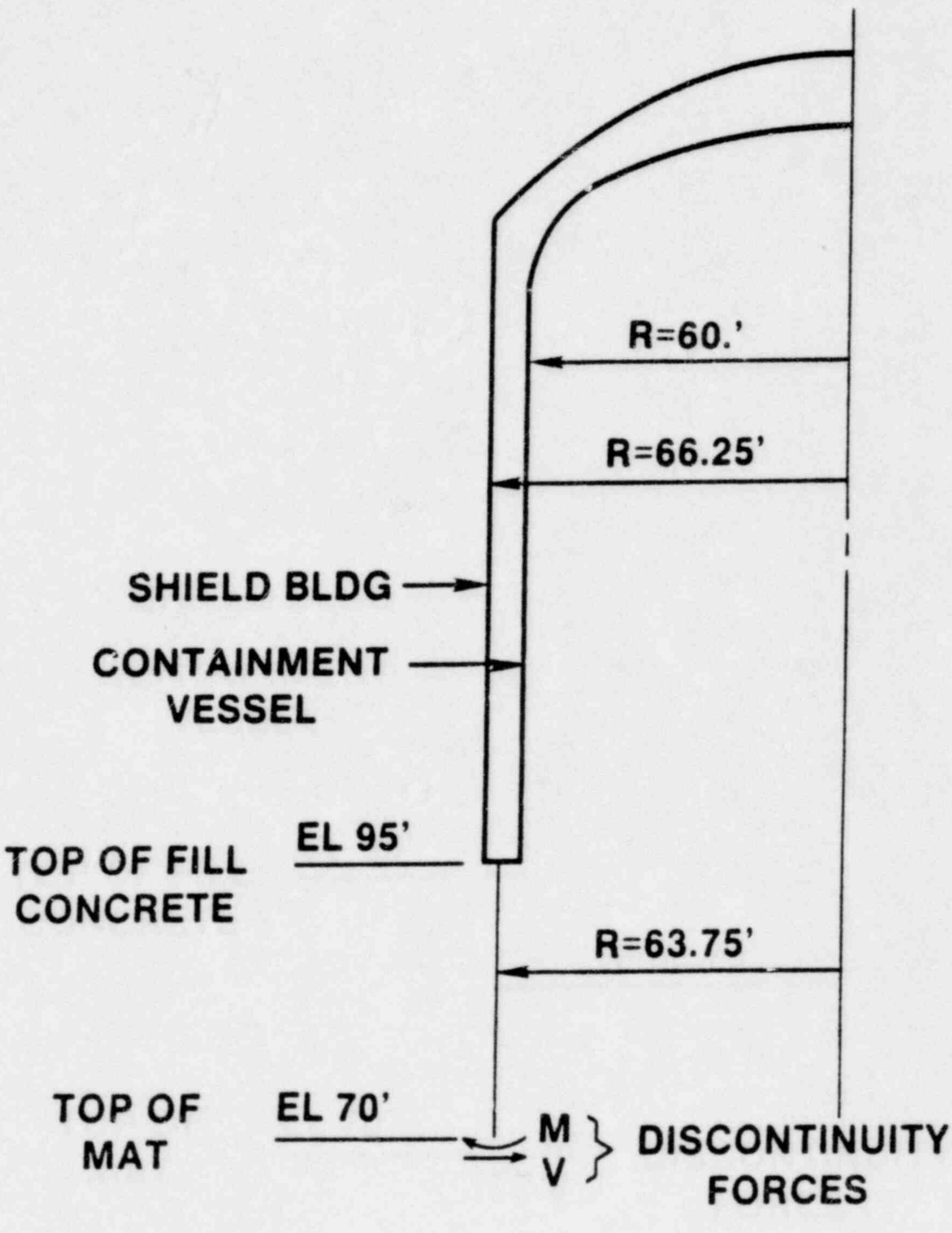


FIG.4 SHELL MODEL

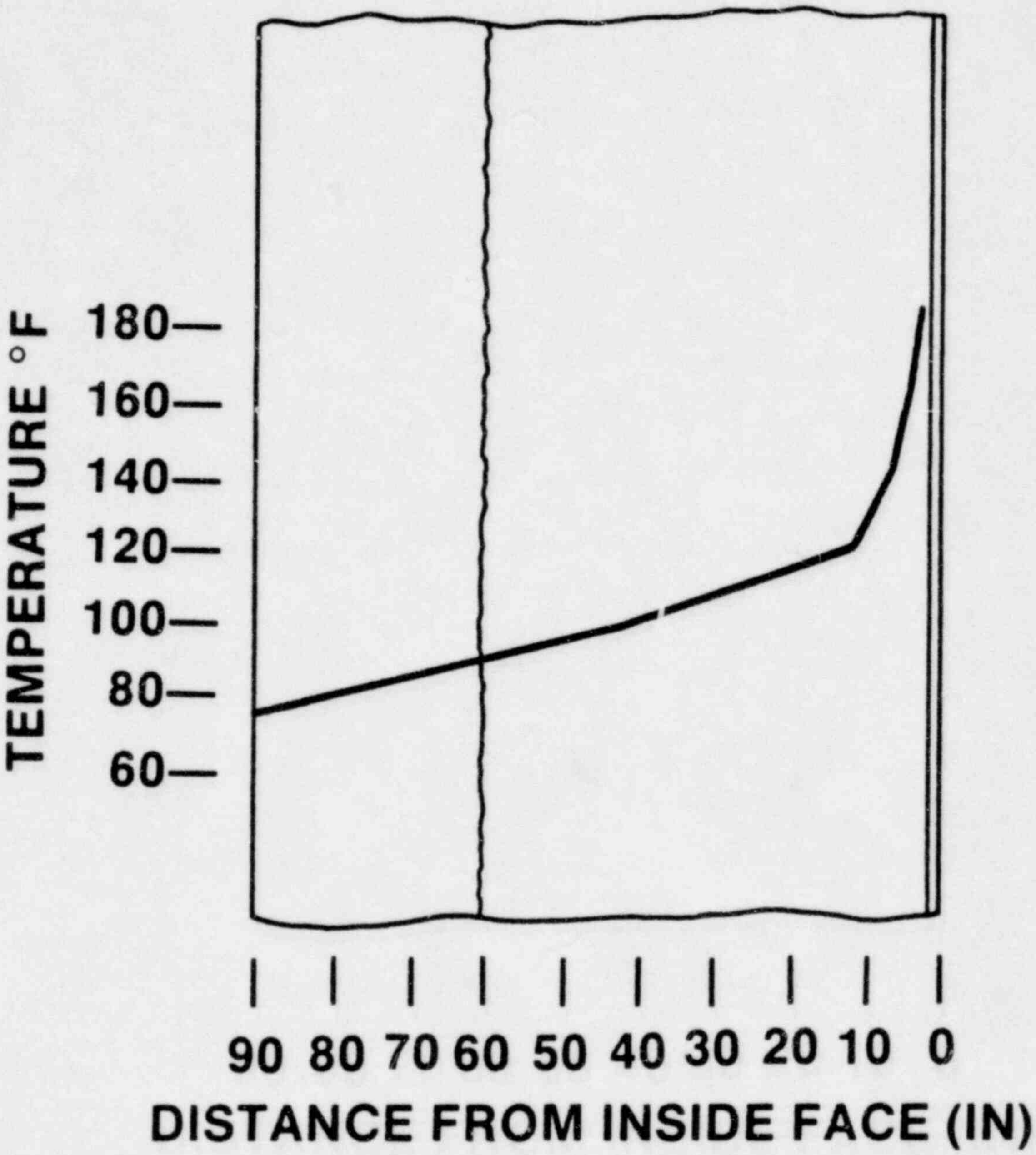
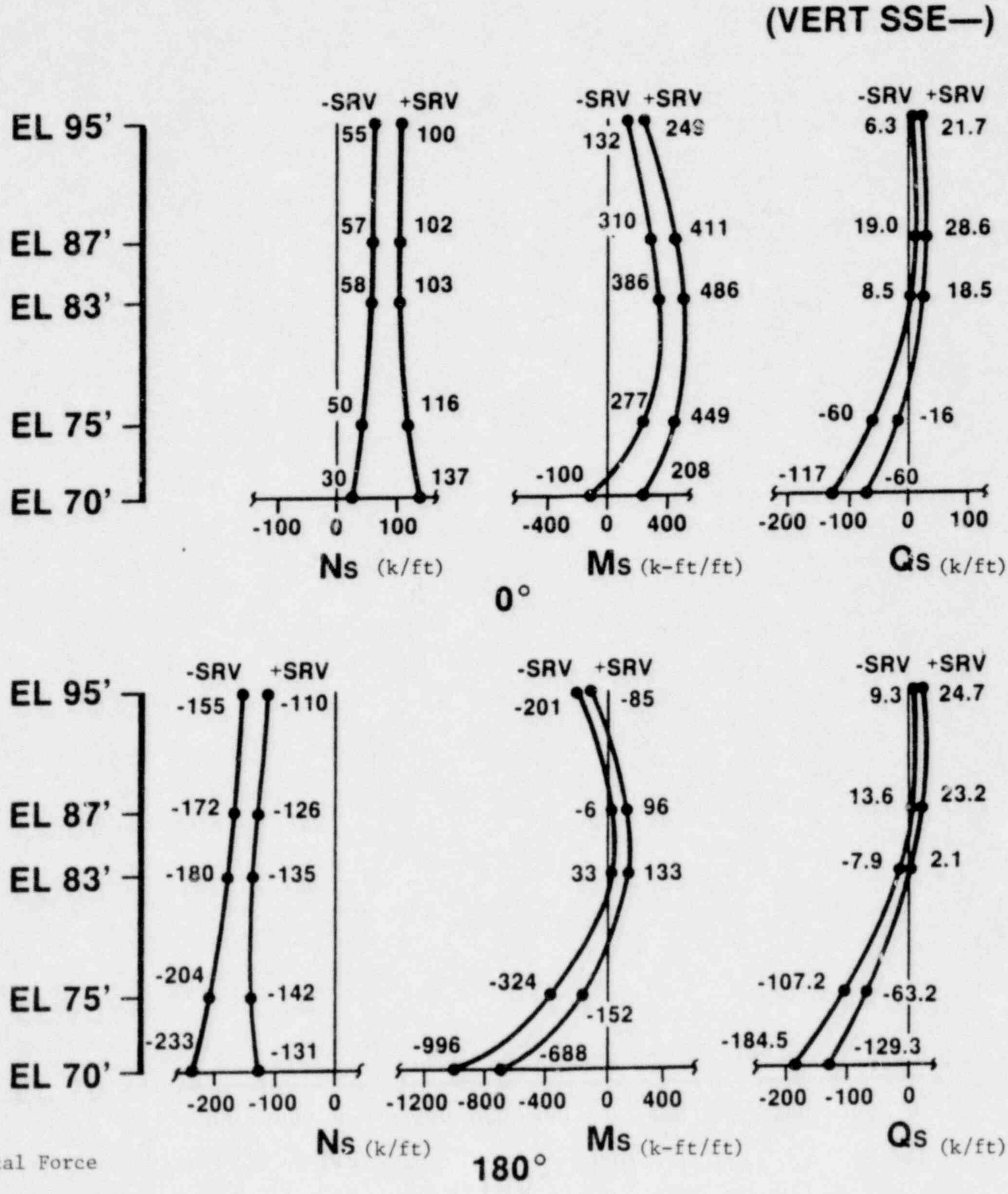


FIG. 5 TYPICAL THERMAL GRADIENT

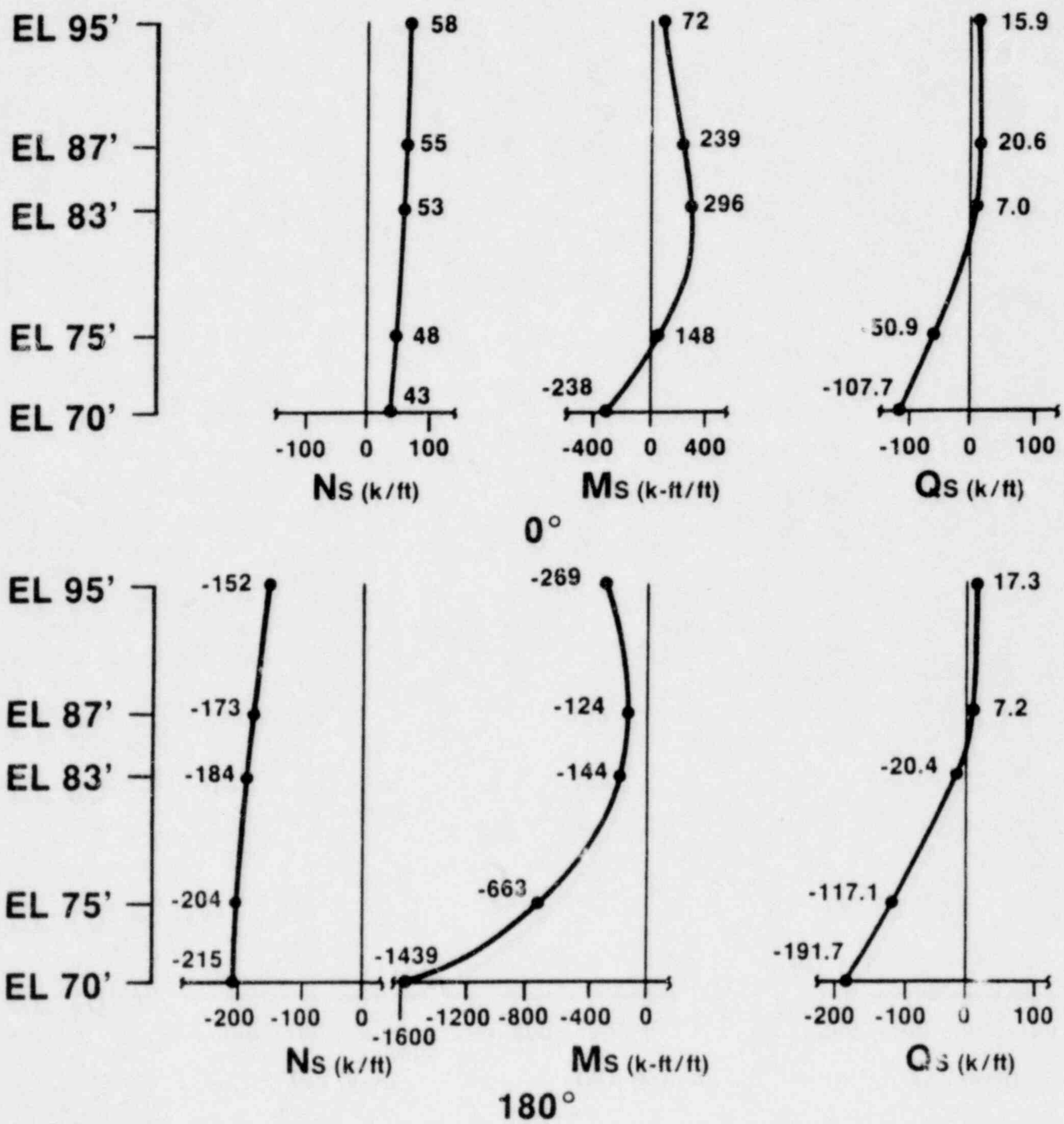


Ns=Vertical Force

Ms=Radial Bending Movement

Qs=Radial Shear Force

FIG. 6 MOMENTS & SHEARS FOR LOAD COMBINATIONS 13.1



Ns=Vertical Force

Ms=Radial Bending Movement

Qs=Radial Shear Force

FIG. 7 MOMENTS & SHEARS FOR LOAD COMBINATION 13.2

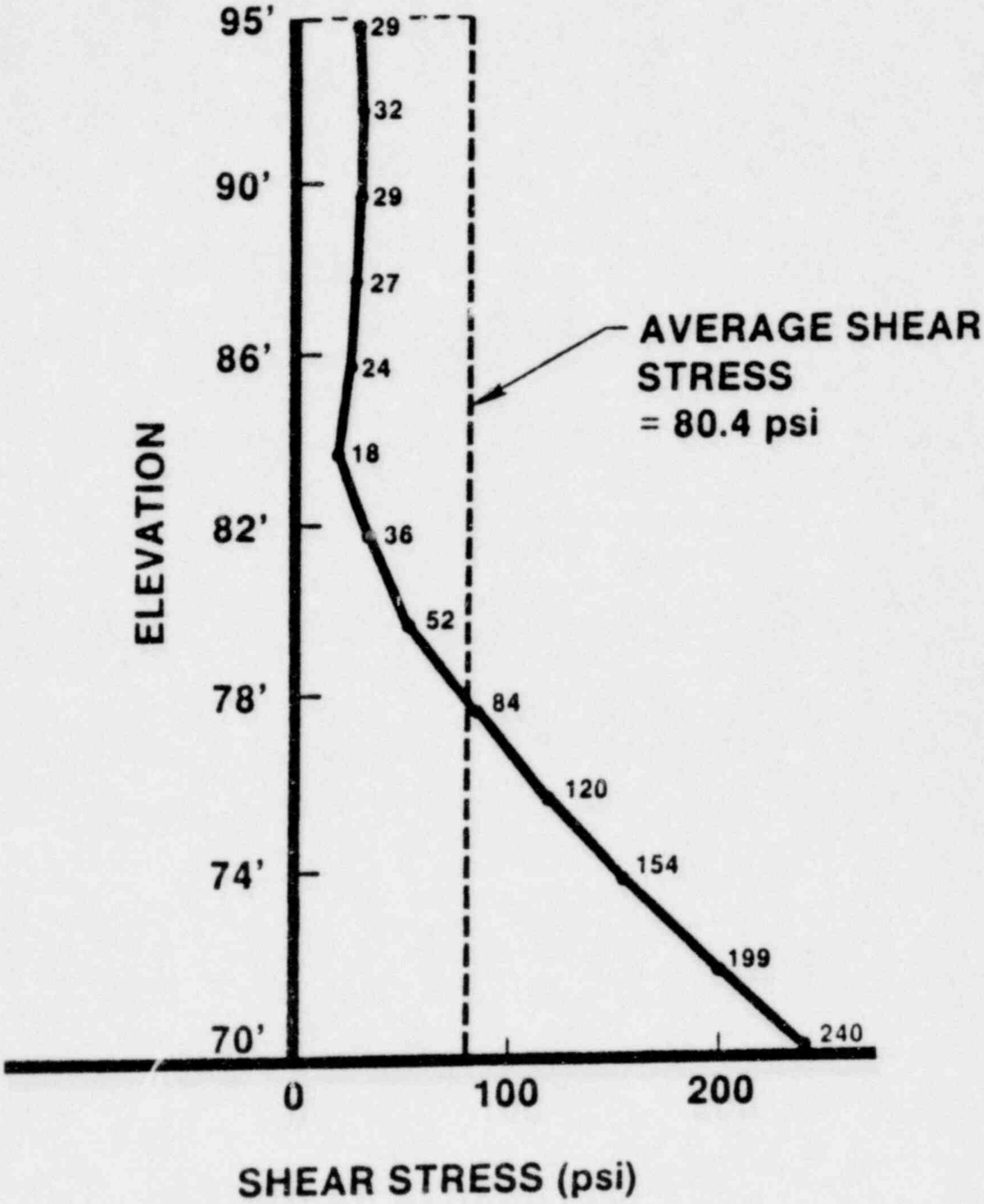


FIG. 8 INTERFACE SHEAR STRESS LOAD COMBINATIONS 13.1

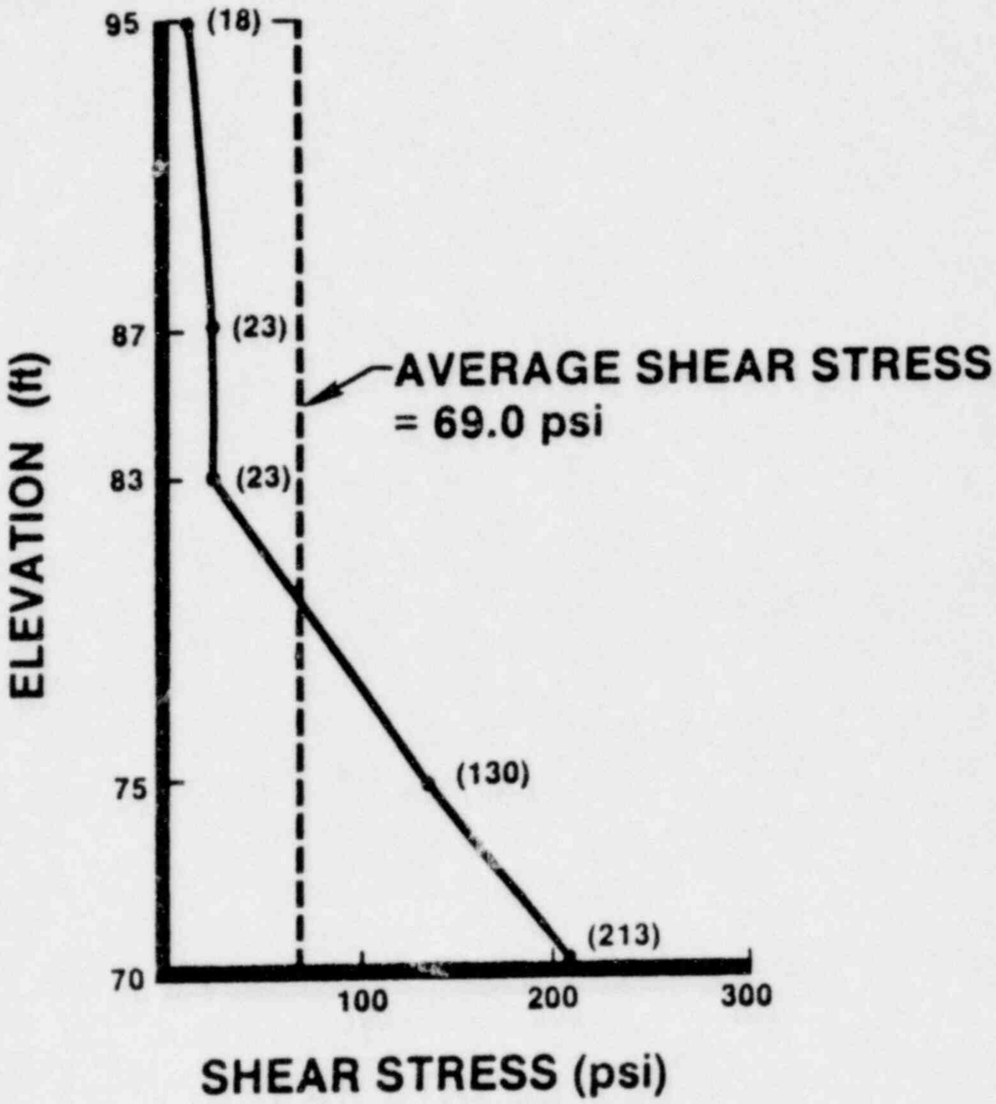


FIG. 9 INTERFACE SHEAR STRESS LOAD COMBINATIONS 13.2

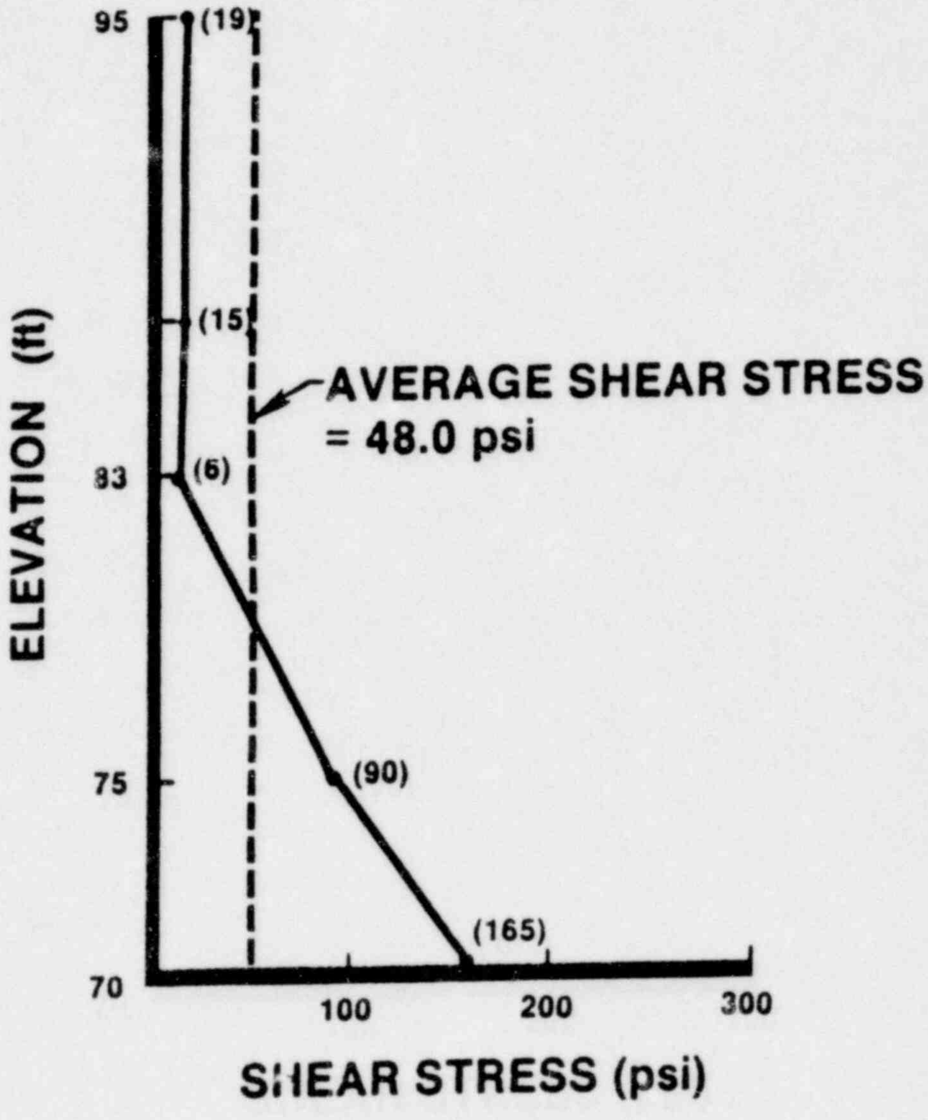


FIG. 10 INTERFACE SHEAR STRESS LOAD COMBINATIONS 8.2

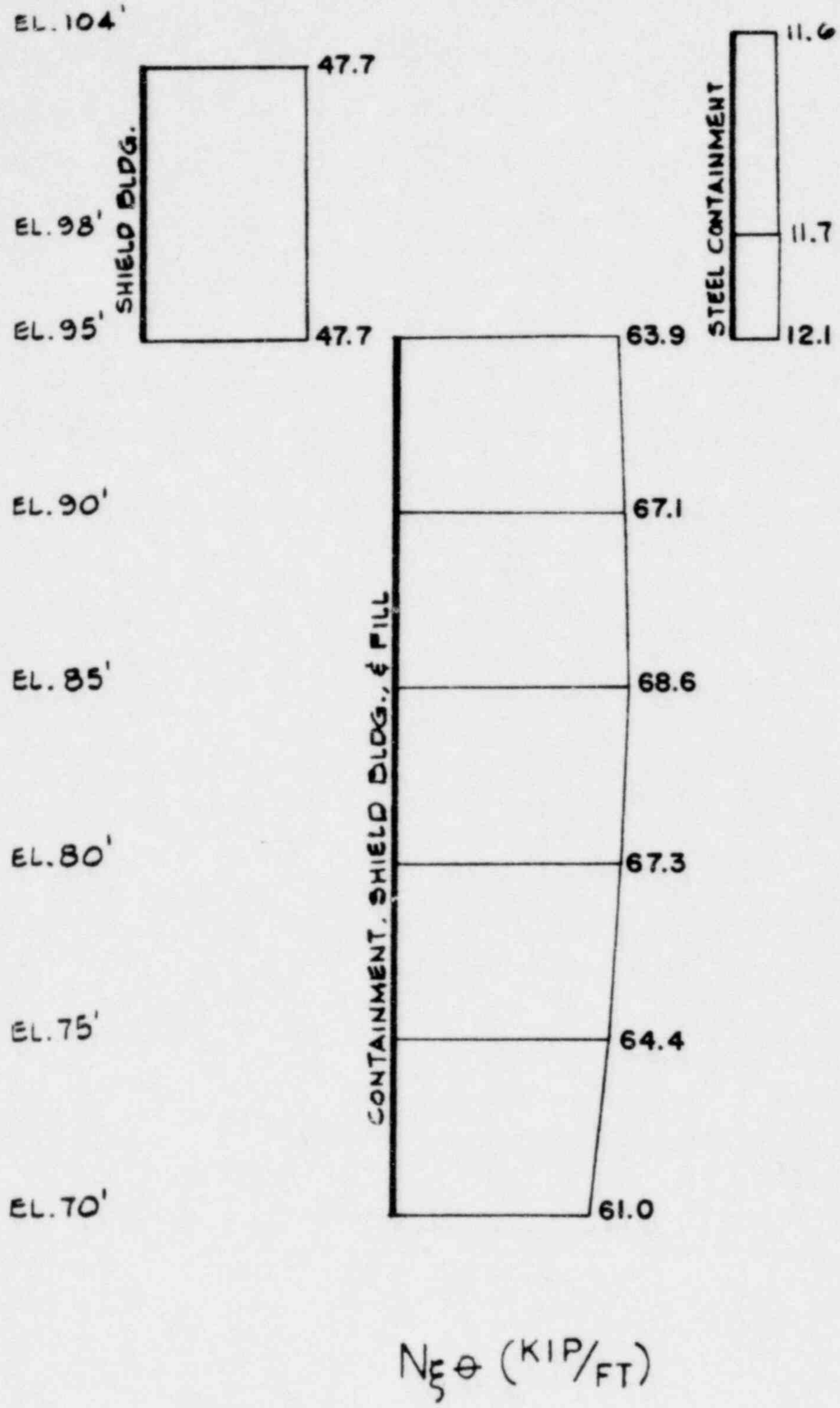


FIG. 11 TANGENTIAL SHEAR FORCES IN THE SUPPRESSION POOL REGION (SSE)

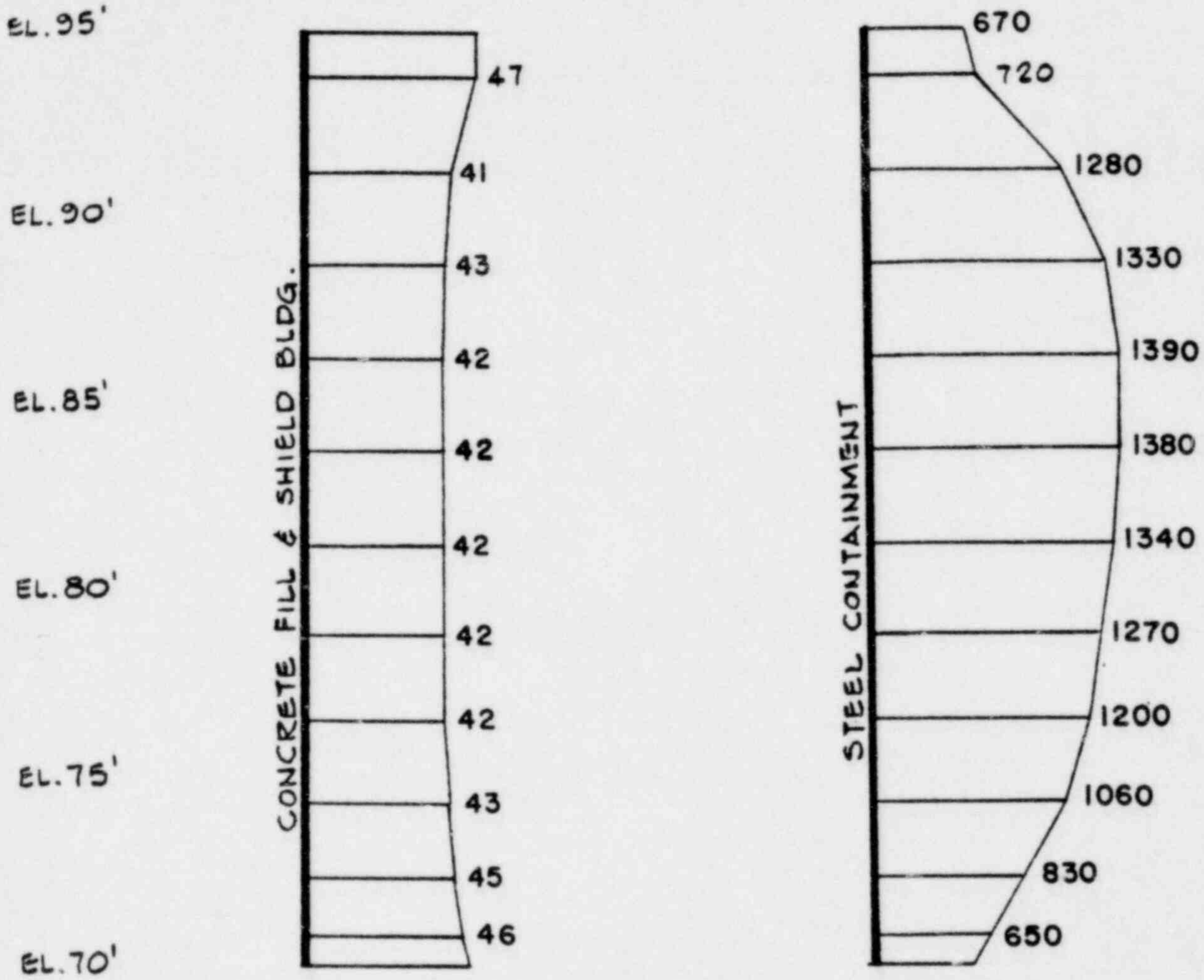


FIG. 12 AVERAGE TANGENTIAL SHEAR STRESS (PSI) IN THE SUPPRESSION POOL REGION

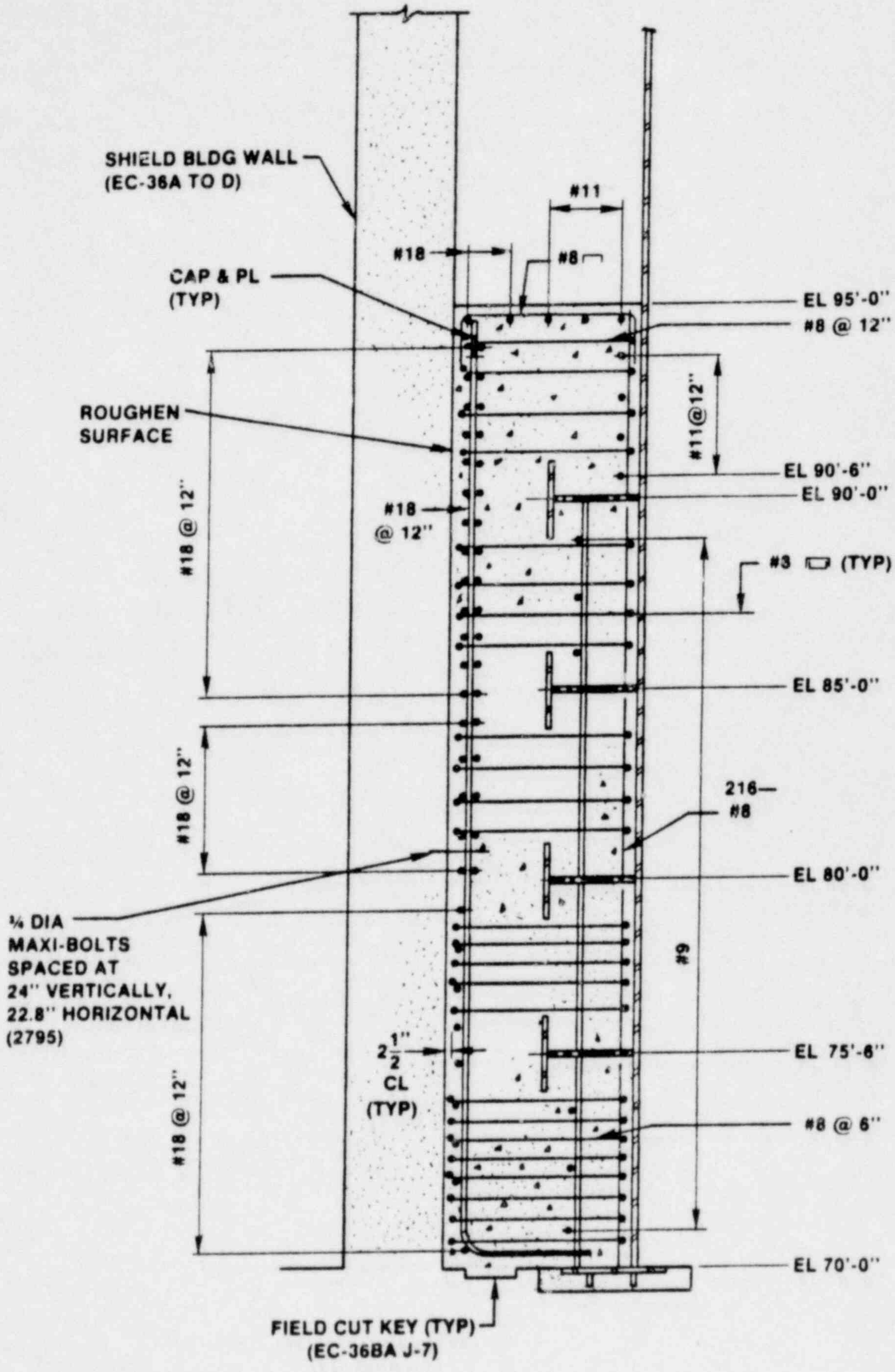


FIG. 13 DESIGN DETAILS OF THE CONCRETE FILL

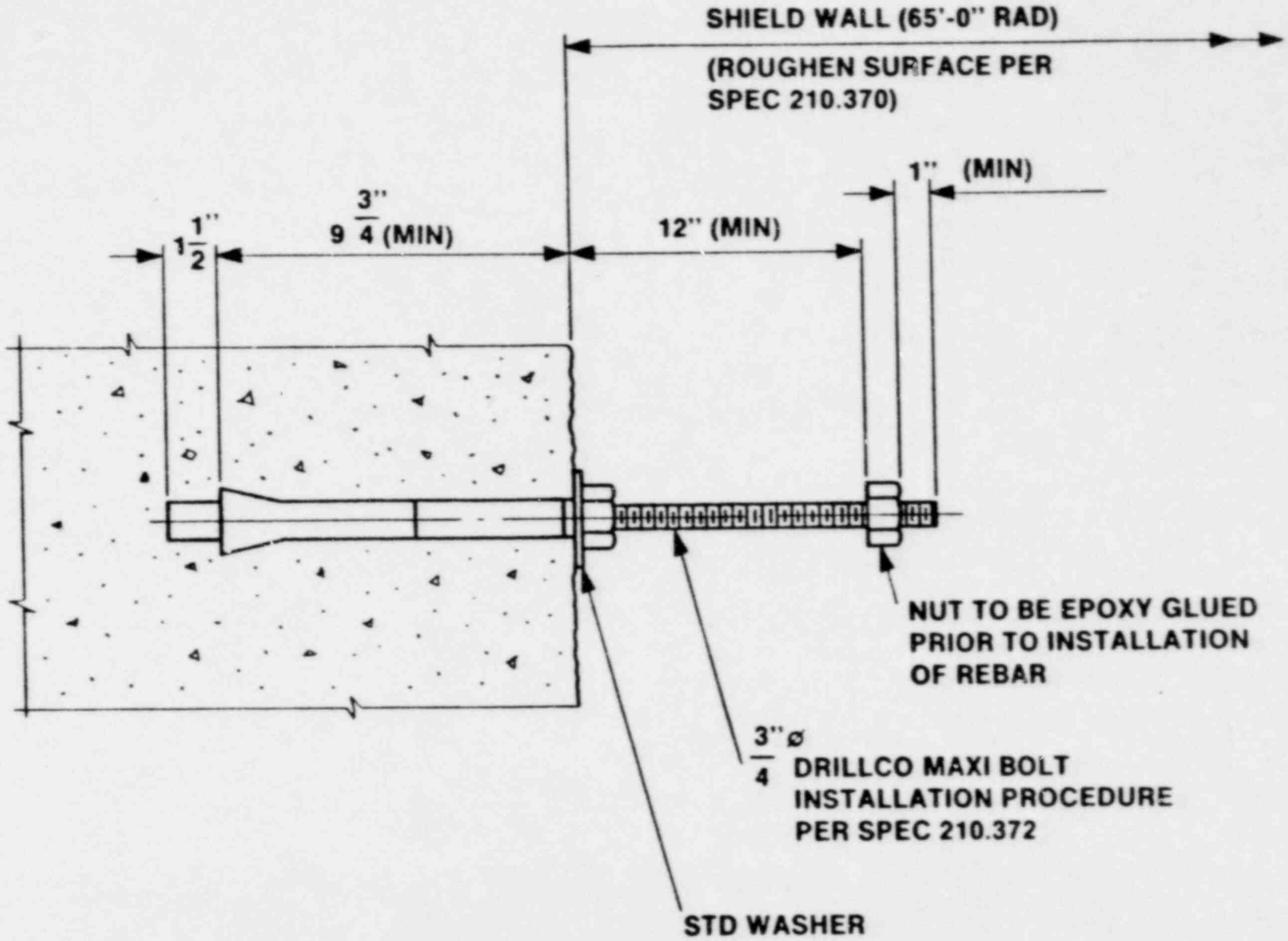


FIG. 14 DETAILS OF MAXI-BOLTS IN THE CONCRETE FILL

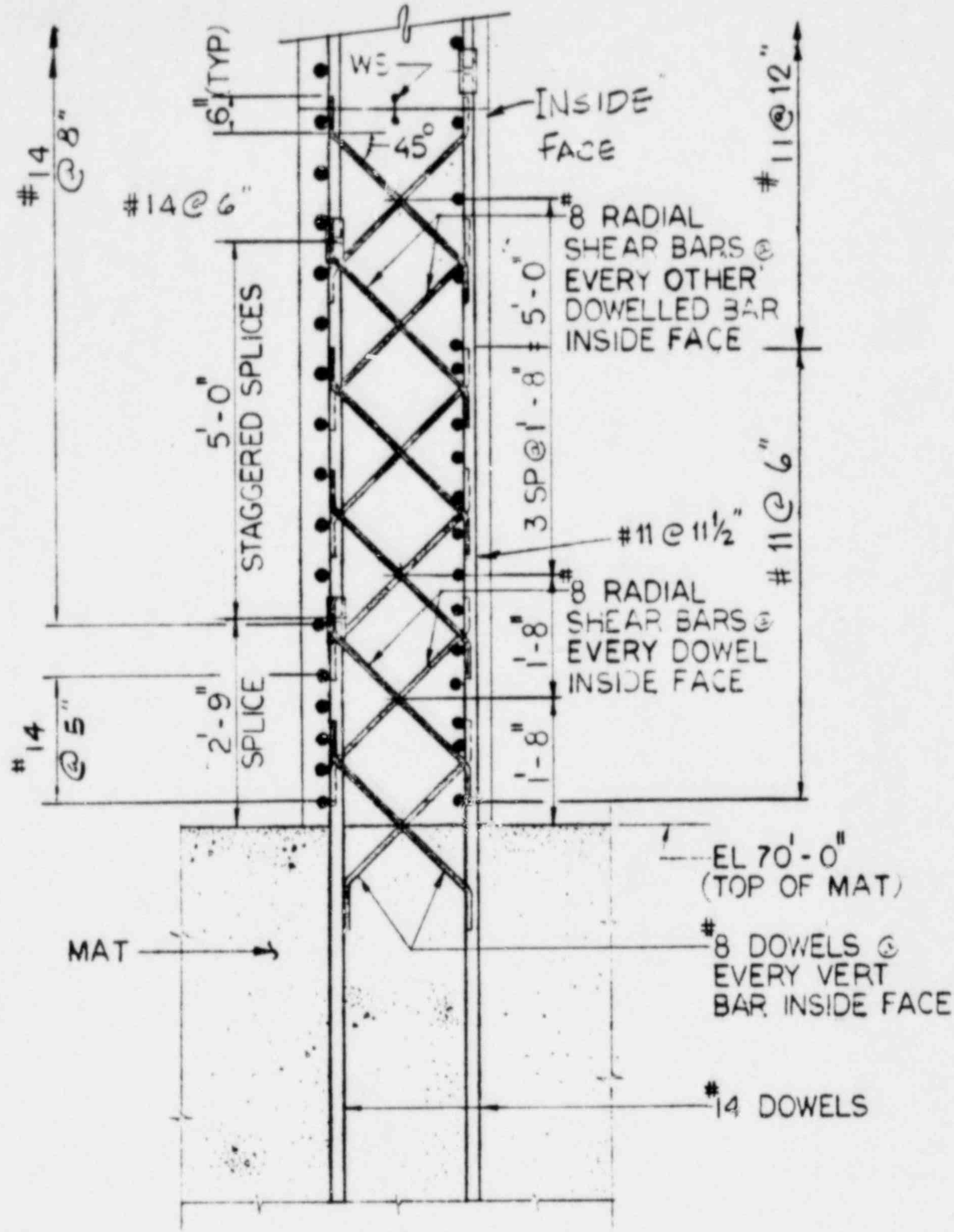


FIG. 15 SHIELD BUILDING DETAILS

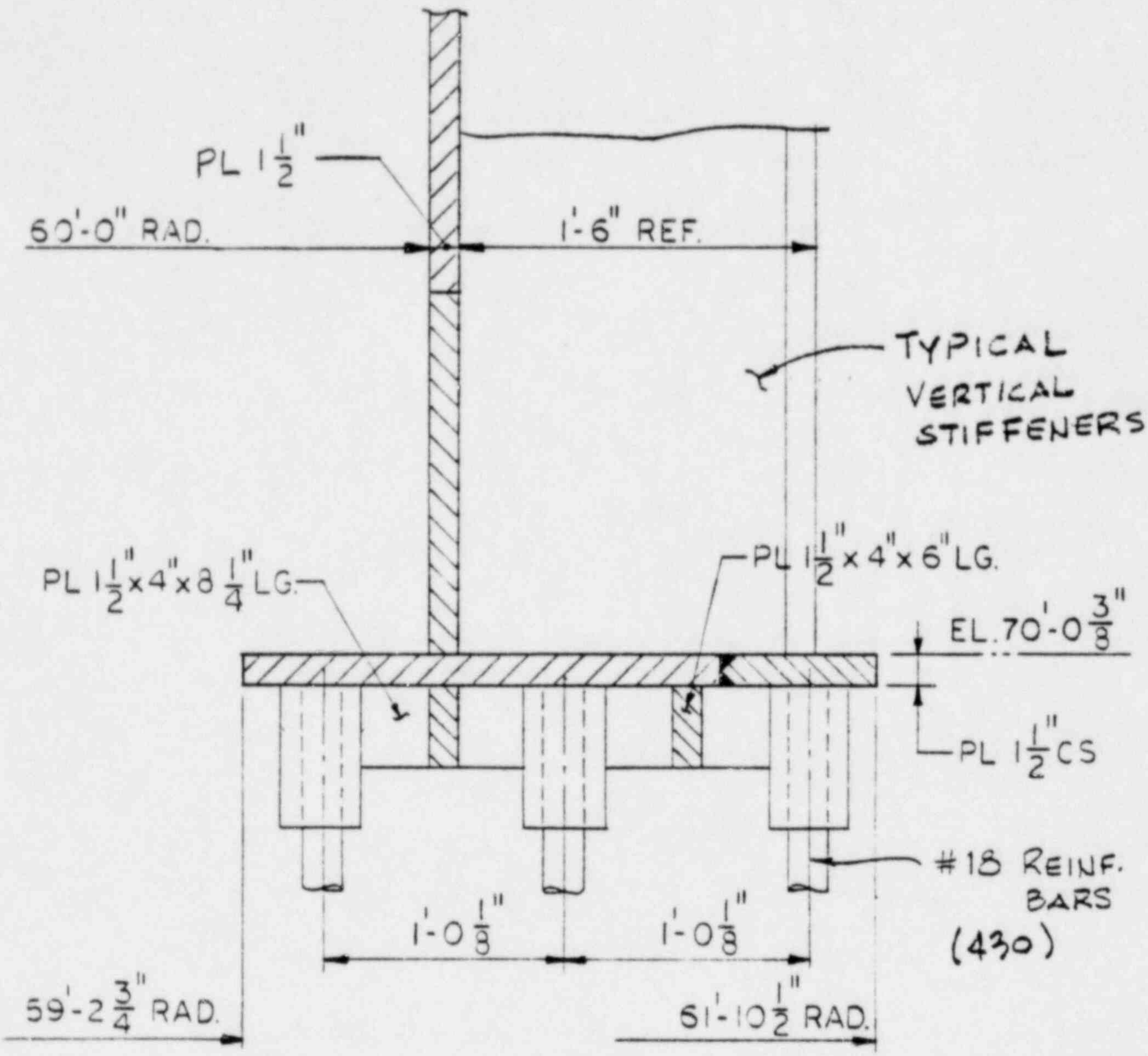


FIG. 16 CONTAINMENT MAT JUNCTION DETAIL

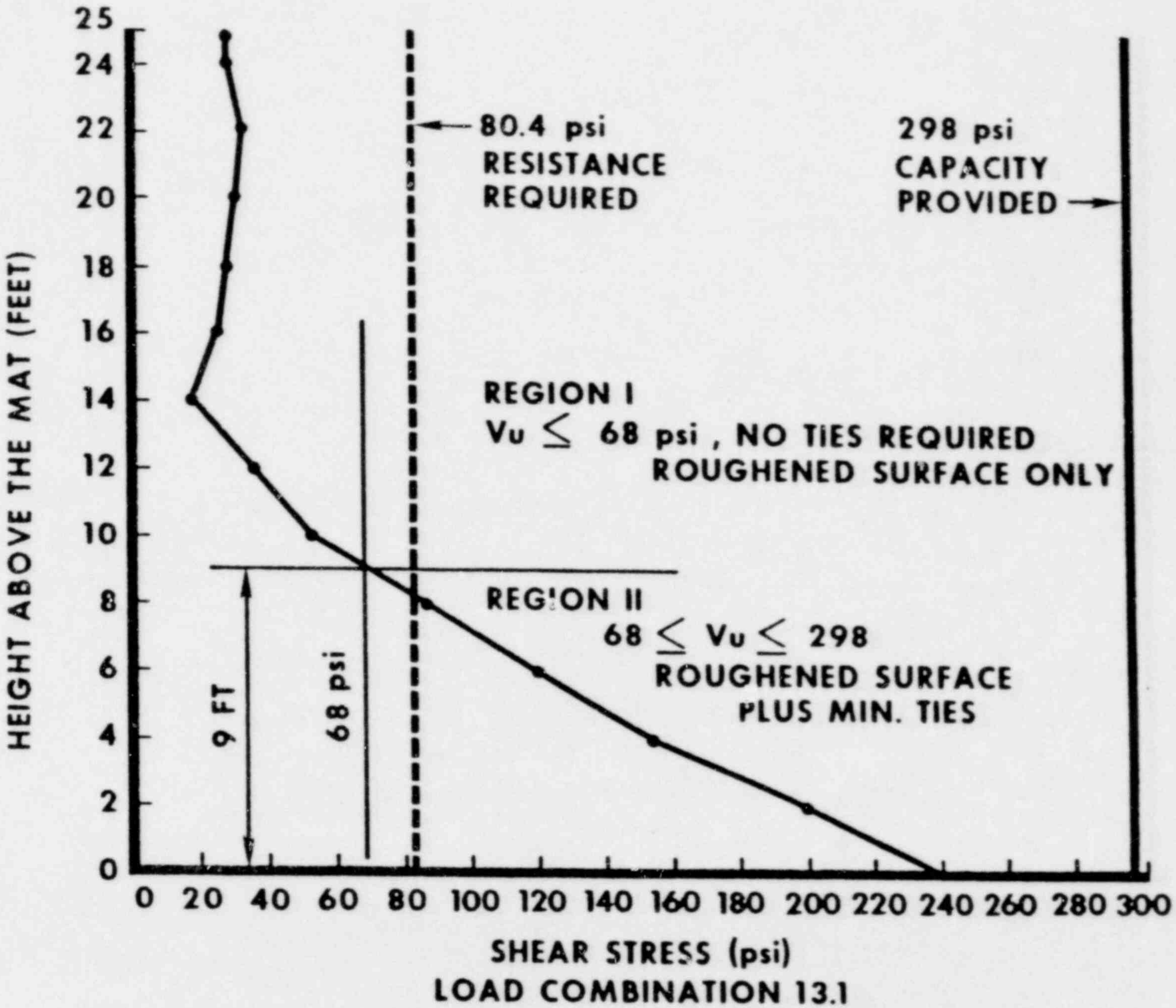
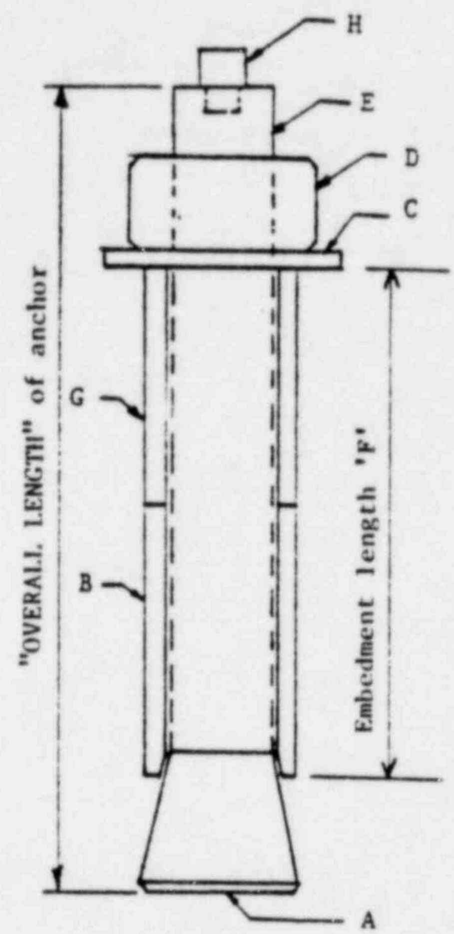


FIG. 17 INTERFACE SHEAR STRESS - COMPARISON OF THE CAPACITY AND REQUIREMENTS



TYPICAL DRILLCO MAXI-BOLTS

A: Conical Nut- ASTM A193 Gr. B7

B: Expansion Sleeve- ASTM A513 Type 5

C: Washer- ASTM A325 Hard Round Washer

D: Heavy Hex. Nut- ASTM A194 Gr. 2H

E: Threaded Stud Bolt- ASTM A193 Gr. B7

G: Filler Sleeve- ASTM A513, Type 5

H: Hex or Square Head for non standard length anchor, indicating length code

FIG. 18 DETAILS OF MAXI-BOLT

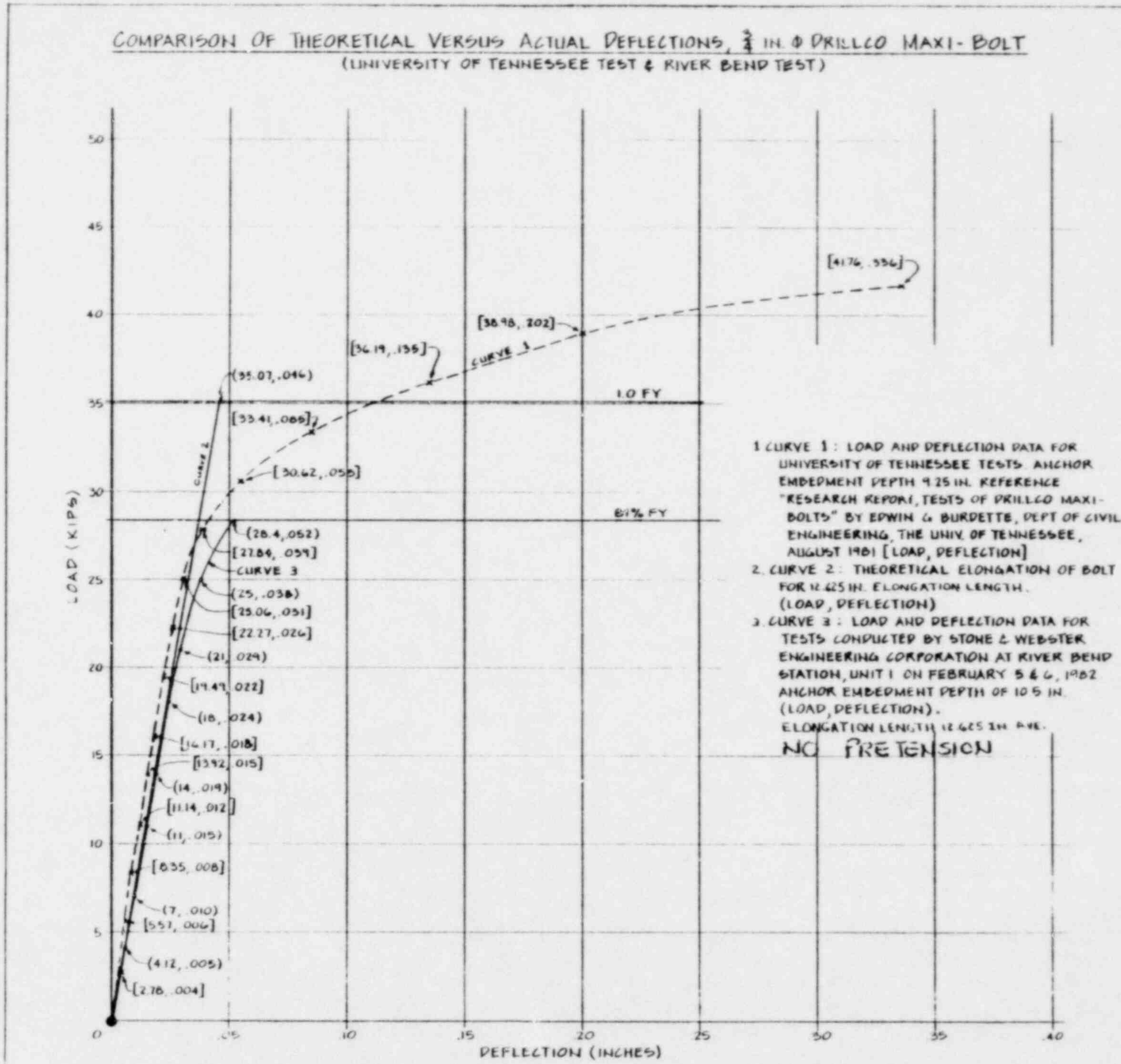


FIG. 19 LOAD DEFLECTION DATA FOR MAXI-BOLT TESTS AT RIVER BEND STATION

APPENDIX

SUMMARY OF THE
TESTS ON MAXI-BOLTS

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SUMMARY OF TESTING

The following pages present in tabular form the most pertinent information relative to documented testing performed on the Drillco Maxi-Bolt. Original test reports upon which this summarization is based are available upon request from Drillco Devices, Ltd.

In addition to the test results presented, other testing has been completed for which documentation is not available at this time. This testing includes site demonstrations at the Catawba, Riverbend, and Satsop Nuclear Stations. Static tension tests of the 1 inch Maxi-Bolt have also been performed at the University of Tennessee by TVA. Results of these tests were consistent with the material recorded here.

TESTING PER [illegible]

1/2" Maxi-Bolt Static Tension   6" [illegible] [illegible] TVA

	Deflection" @ .50 Fy*	Deflection" @ .81 Fy*	Peak Load (KIPS)	Peak Stress (KSI)	Failure Mode	Location
	.007	.050	19.2	135.2	Stud	TVA
	.004	.065	20.5	144.4	Stud	TVA
	.005	.022	20.1	141.5	Stud	TVA
	.012	.050	20.1	141.5	Concrete (1)	TVA
Average	.007	.047	20.0	140.7		
	.032	.039	16.7	117.6	**	UT
	.012	.029	20.9	147.2	Stud	UT
	.014	.030	20.2	142.3	Stud	UT
	.016	.035	20.9	147.2	Stud	UT
	.015	.035	20.9	147.2	Stud	UT
	.014	.034	22.1	155.6	Stud	UT
	.005	.018	20.5	144.4	Stud	UT
	.009	.026	19.5	137.3	**	UT
Average	.015	.031	20.9	147.2		
	.014	.116	20.6	145.1	Stud	P
	.046	.145	20.5	144.4	Stud	P
	.013	.042	21.5	151.4	Stud	P
age	.024	.101	20.9	147.0		
	.057	.085	21.2	149.3	Stud	R
	.047	.065	22.7	159.9	Stud	R
	.038	.060	23.0	162.0	Stud	R
	.034	.076	23.0	162.0	Stud	R
	.063	.092	22.4	157.7	Stud	R
Average	.048	.076	22.5	158.2		

TVA- Tennessee Valley Authority, Average f'c 4227 PSI
UT- University of Tennessee, Average f'c 3000 PSI
P- Pittsburgh Testing Lab, Average f'c 3775 PSI
R- Rockport Generating Station, Average f'c 7000 PSI

* .50 Fy = 7.5 KIPS
.81 Fy = 12.1 KIPS

** Block split. Bolts set 7 in. from free edge. Results omitted from average.

(1) 125,000 psi minimum specified tensile capacity for A 193 B7 bolting material. Ductility of material fully developed.

5/8" Maxi-Bolt Static Tension

	Deflection" @ .50 Fy*	Deflection" @ .81 Fy*	Peak Load (KIPS)	Peak Stress (KSI)	Failure Mode	Location
	.010	.075	30.5	135.0	Stud	TVA
	.010	.035	29.3	130.0	Stud	TVA
	.010	.025	29.8	131.9	Stud	TVA
	.020	.025	30.5	135.0	Stud	TVA
Average	.013	.040	30.0	133.0		
	.010	.019	30.6	135.4	Stud	UT
	.011	.025	31.0	137.2	Stud	UT
	.011	.022	30.3	134.1	Stud	UT
	.013	.029	30.6	135.4	Stud	UT
	.013	.025	29.6	131.0	Stud	UT
	.012	.025	30.3	134.1	Stud	UT
Average	.012	.024	30.4	134.5		
	.074	.213	30.1	133.2	Stud	P
	.114	.413	29.9	132.3	Stud	P
	.121	.293	28.9	127.9	Stud	P
	.019	.157	29.4	130.1	Concrete(1)	P
	.015	.213	30.9	136.7	Concrete(1)	P
Average	.069	.258	29.8	132.0		
	.046	.073	29.5	130.5	Stud	R
	.052	.102	31.5	139.4	Stud	R
	.084	.120	31.2	138.1	Stud	R
	.069	.099	29.9	132.3	Stud	R
	.058	.075	31.5	139.4	Stud	R
Average	.062	.094	30.7	135.9		

TVA- Tennessee Valley Authority, Average f'c 4227 PSI
UT- University of Tennessee, Average f'c 3000 PSI
P- Pittsburgh Testing Lab, Average f'c 3775 PSI
R- Rockport Generating Station, Average F'c 7000 PSI

* .50 Fy = 11.9 KIPS
.81 Fy = 19.2 KIPS

(1) 125,000 psi minimum specified tensile capacity for A 193 B7 bolting material. Ductility of material fully developed.

3/4" Maxi-Bolt Static Tension

	Deflection" @ .50 Fy*	Deflection" @ .81 Fy*	Peak Load (KIPS)	Peak Stress (KSI)	Failure Mode	Location
	.020	.070	44.0	131.7	Stud	TVA
	.060	.125	45.3	135.6	Stud	TVA
	.050	.090	46.6	139.5	Stud	TVA
	.040	.150	44.0	131.7	Stud	TVA
Average	.043	.109	45.0	134.6		
	.021	.043	46.6	139.5	Stud	UT
	.015	.043	46.3	138.6	Stud	UT
	.025	.050	44.5	133.2	Stud	UT
	.024	.046	47.7	142.8	Stud	UT
	.011	.028	46.6	139.5	Stud	UT
	.020	.042	45.9	137.4	Stud	UT
Average	.019	.042	46.3	138.6		
	.065	.161	47.4	141.9	Stud	P
	.033	.185	46.6	139.5	Stud	P
	.039	.153	46.2	138.3	Stud	P
	.033	.188	46.6	139.5	Stud	P
	.016	.092	48.6	145.5	Stud	P
Average	.037	.156	47.1	140.9		
	.070	.113	47.3	141.6	Stud	R
	.091	.168	48.9	146.4	Stud	R
	.120	.248	48.6	145.5	Stud	R
	.073	.149	46.7	139.8	Stud	R
	.069	.118	47.3	141.6	Stud	R
Average	.085	.159	47.8	143.0		

TVA- Tennessee Valley Authority, Average f'c 4227 PSI
UT- University of Tennessee, Average f'c 3000 PSI
P- Pittsburgh Testing Lab, Average f'c 3775 PSI
R- Rockport Generating Station, Average f'c 7000 PSI

* .50 Fy = 17.5 KIPS
.81 Fy = 28.4 KIPS

1-1/4" Maxi-Bolt Static Tension

Tests conducted January 8 and 9, 1981.

Results of Three Static Tension Tests Performed at Singleton Materials Laboratory, Tennessee Valley Authority.

Anchor: MB-1250 1¼ inch x 41 inch overall x 16 inch embedment Drillco Maxi-Bolt

Concrete: Three blocks 36 inch x 36 inch x 36 inch, approx. compressive strength - 5,000 psi.

Test Procedure: After pretensioning anchors to .80 F_y of stud bolt material, ASTM A 193 GR B7, the anchors were pulled to failure with ultimate capacity and failure mechanism being recorded.

Test Results:

Test #1: Ultimate capacity - 149,100 lbs.
Failure mechanism - Stud bolt broke.

Test #2: Ultimate capacity - 135,000 lbs.
Failure mechanism - Concrete block split, stud bolt material in yield.

Test #3: Ultimate capacity - 127,000 lbs.
Failure mechanism - Stud bolt broke.

Results tabulated by Drillco representative present at testing.

Installations performed by TVA personnel per Drillco's Suggested Installation Procedure.

1/2" Maxi-Bolt Static Shear

	Torque Ft. Lb.	Peak Load (KIPS)	Peak Stress (KSI)	Total Deflection"	Failure Mode
(1)	150	12.8	90.1	.350	Stud
	150	13.7	96.5	.280	Stud
	150	15.7	110.6	.305	Stud
	150	12.6	88.7	.260	Stud
	150	14.7	103.5	.357	Stud
	150	14.4	101.4	.220	Stud
	150	13.0	91.5	.251	Stud
	150	13.3	93.7	.178	Stud
	150	13.3	93.7	.168	Stud
Average		13.7	96.6	.263	
(2)	150	18.4	129.6	.570	Stud
	150	16.6	116.9	.490	Stud
	150	16.6	116.9	.580	Stud
	150	14.4	101.4	.720	Stud
	125	13.6	95.8	.380	Stud
	150	18.5	130.3	NR *	Stud
	150	18.8	132.4	NR *	Stud
	150	16.0	112.7	NR *	Stud
	150	18.4	129.6	NR *	Stud
Average		16.8	118.4	.548	

(1) Conducted at University of Tennessee December 15,16,17-1980
f'c between 5700 and 6000 PSI

(2) Conducted at University of Tennessee August 4,5,7-1981
f'c between 3000 and 4100 PSI

* Conducted at University of Tennessee April 7, 1981
f'c = 3786 @ 25 Days. Deflections not recorded.

5/8" Maxi-Bolt Static Shear

	Torque Ft. lb.	Peak Load (KIPS)	Peak Stress (KSI)	Total Deflection"	Failur Mode
(1)	175	18.6	82.3	.265	Stud
	200	19.2	85.0	.216	Stud
	250	21.0	92.9	.383	Stud
	250	18.8	83.2	.267	Stud
	250	20.4	90.3	.240	Stud
	250	19.7	87.2	.309	Stud
	250	20.2	89.4	.199	Stud
	250	20.7	91.6	.286	Stud
	250	19.1	84.5	.371	Stud
Average		19.7	87.4	.282	
(2)	250	22.8	100.9	.421	Stud
	250	23.2	102.7	.555	Stud
	250	19.2	85.0	.450	Stud
	275	20.0	88.5	.392	Stud
	250	20.8	92.0	.696	Stud
	250	20.6	91.2	.507	Stud
	250	19.4	85.8	.360	Stud
	250	20.0	88.5	.348	Stud
	250	19.4	85.8	.258	Stud
Average		20.6	91.2	.443	

(1) Conducted at University of Tennessee December 17,18-1980. f'c between 5700 and 6000 PSI

(2) Conducted at University of Tennessee July 30,31 and August 3,6,7-1981. f'c between 3000 and 4100 PSI

3/4" Maxi-Bolt Static Shear

	Torque Ft. lb.	Peak Load (KIPS)	Peak Stress (KSI)	Total Deflection"	Failure Mode
(1)	350	29.1	87.1	.223	Stud
	350	30.0	89.8	.245	Stud
	350	30.2	90.4	.246	Stud
	350	31.2	93.4	.224	Stud
	350	29.3	87.7	.192	Stud
	350	31.2	93.4	.272	Stud
Average		30.2	90.3	.234	
(2)	350	34.8	104.2	.563	Stud
	350	37.0	110.8	.570	Stud
	350	31.6	94.6	.390	Stud
	350	28.8	86.2	.539	Stud
	350	31.2	93.4	.604	Stud
	350	30.8	92.2	.536	Stud
	350	32.0	95.8	.680	Stud
	350	32.0	95.8	.620	Stud
	350	32.4	97.0	.434	Stud
rage		32.3	96.7	.548	

(1) Conducted at University of Tennessee December 19, 1981
f'c between 5700 and 6000 PSI

(2) Conducted at University of Tennessee August 3,4,6,7,-1981 &
f'c between 3000 and 4100 PSI July 31, 1981

LOAD INCREMENTS FOR DYNAMIC TESTS

--f_y = 105 ksi--

A. Tensile Tests

No.	Stress	No. of Cycles	Maximum Loading (kips) 1/2" Dia.	5/8" Dia.	3/4" Dia.
1	$0.50f_y$	7,000	7.46	11.87	17.54
2	$0.60f_y$	2,000	8.95	14.24	21.04
3	$0.70f_y$	2,000	10.44	16.61	24.55
4	$0.80f_y$	2,000	11.93	18.98	28.06
5	$0.90f_y$	2,000	13.42	21.36	31.56
6	$1.00f_y$	2,000	14.91	23.73	35.07

29.5

B. Shear Tests

No.	Stress	No. of Cycles	Maximum Loading (kips) 1/2" Dia.	5/8" Dia.	3/4" Dia.
1	$0.35f_y$	7,000	5.22	8.31	12.27
2	$0.42f_y$	2,000	6.26	9.97	14.73
3	$0.49f_y$	2,000	7.31	11.63	17.18
4	$0.56f_y$	2,000	8.35	13.29	19.64
5	$0.63f_y$	2,000	9.39	14.95	22.09
6	$0.70f_y$	2,000	10.44	16.61	24.55
7	$0.77f_y$	2,000	11.48	18.27	27.00
8	$0.84f_y$	2,000	12.50	19.93	29.46
9	$0.91f_y$	2,000	13.54	21.59	31.91
10	$0.98f_y$	2,000	14.61	23.26	34.37

TEST RESULTS FOR DYNAMIC TENSION TESTS

--f'_c BETWEEN 3,300 AND 3,800 psi EXCEPT

FOR TESTS WITH * WHICH HAD f'_c BETWEEN 5,500 AND 6,000--

CONDUCTED AT UNIVERSITY OF TENNESSEE

Test No.	Bolt Dia. (In.)	Peak Load (KIPS)	Peak Stress (KSI)	No. Cycles at Peak Load
1	1/2	14.91	105	2,000
5	1/2	22.2 (1)	156	0
6	1/2	21.5 (1)	151	0
7	1/2	14.91	105	110
1*	1/2	17.0	120	110
2	5/8	23.73	105	1,350
3	5/8	23.73	105	750
4	5/8	29.4 (2)	130	0
2*	5/8	23.0 (3)	102	1,000
3*	5/8	22.0 (4)	97	1,200
8	3/4	48.0 (5)	105	0
9	3/4	42.3 (5)	105	0
10	3/4	35.07	105	880
11	3/4	35.07	105	1,820
12	3/4	31.56	94	1,480
4*	3/4	47.0 (6)	141	0
5*	3/4	38.5 (7)	115	0

Notes:

(1) After 2,000 cycles at 14.91 kips, bolt failed under a static load of 22.2 kips in Test 5 and 21.5 kips in Test 6.

(2) After 2,000 cycles at 23.73 kips, bolt failed under a static load of 29.4 kips.

(3) Bolt was cycled 7,000 times at 13.0 kips, 2,000 at 18.0, 2,000 at 22.0, and 1,000 at 23.0 kips when failure occurred

(4) Bolt was cycled 7,000 times at 13.0 kips, 2,000 at 18.0, and 1,200 at 22.0 kips when failure occurred.

(5) After 2,000 cycles at 35.07 kips, bolt failed under a static load of 48.0 kips in Test 8 and 42.3 kips in Test 9.

(6) Bolt was cycled 7,000 times at 19.3 kips and failed under a static load of 47.0 kips.

(7) Bolt was cycled 7,000 times at 19.3 kips, 2,000 at 26.0, 1,000 at 32.0, and failed under a static load of 38.5 kips.

TEST RESULTS FOR DYNAMIC SHEAR TESTS

--f'_c Between 3,000 psi AND 3,800 psi--

CONDUCTED AT UNIVERSITY OF TENNESSEE

Test No.	Bolt Dia. (in.)	Peak Load (KIPS)	Peak Stress (KSI)	No. Cycles at Peak Load
1	1/2	12.50	88.0	1,650
2	1/2	13.54	95.4	1,275
3	1/2	13.54	95.4	900
4	1/2	13.54	95.4	0
5	1/2	13.54	95.4	0
6	5/8	18.27	80.8	600
7	5/8	16.61	73.5	400
8	5/8	16.61	73.5	975
9	5/8	19.93	88.2	40
10	5/8	21.59	95.5	20
11	3/4	29.46	88.2	800
12	3/4	27.00	80.8	0
13	3/4	29.46	88.2	0
14	3/4	27.00	80.8	190
15	3/4	24.55	73.5	675

1/2" Maxi-Bolt Cracked Beam Dynamic Tests

Tests conducted at University Of Tennessee By TVA.

Purpose of Test:

Evaluate expansion anchor performance under maximum stress allowables in highly stressed cracked concrete.

Test Beam:

Test beam measured 2 feet wide by 1 foot thick by 17 feet long. The concrete had a 28 day compressive strength of 5800 psi. Three No. 6 and three No. 8 bars were placed in each face of the beam. Beam span for test was 12 feet.

Description of Test:

Load was transmitted from a hydraulic jack through a load cell to a pin connected rigid attachment which was fastened to the test beam with expansion anchors. Loading sequence was controlled by a function generator which can input and readout both load and deflection. Load was measured by a load cell connected to the loading ram and displacement was measured by an LVDT. A sine-wave function was utilized for input control throughout these tests. The input signal controlled beam deflection.

Test Results:

1. All four anchors were cycled in tension from 3 KIPS to 29 KIPS (7.25 KIPS per bolt or 49% Fy) for a total of 305 cycles. Plate deflection measured .019 inch. At this point flexure cracking in the beam had propagated to the anchors.

2. The beam was cycled in compression from 2 KIPS to 16 KIPS for 317 cycles. This loading affected the beam only, not the anchors.

3. Because of test apparatus load limits, tension loading at more than 29 KIPS was not possible. To attain a higher load per bolt it was necessary to remove the nut and washer from two anchors diagonal to one another. The remaining two anchors were then cycled in tension from 2 KIPS to 23 KIPS (11.5 KIPS per bolt or 77% Fy) for 110 cycles. Plate deflection measured .093 inch.

4. The beam was cycled in compression from 2 KIPS to 27 KIPS for 91 cycles.

1/2" Maxi-Bolt Cracked Beam Dynamic Tests, continued

5. The same two bolts were once again cycled in tension from 1 KIP to 24 KIPS (12 KIPS per bolt or 80% Fy) for 298 cycles. Plate deflection measured .164 inch.

6. The nut and washers were next removed from the two anchors tested in paragraphs 1., 3. and 5. above, and the nuts and washers were replaced on the other two anchors which had been tested only in paragraph 1. above. The nuts on these anchors were not retorqued but were instead run down finger tight on the anchor studs. A compression tension cycle was applied to these anchors from 17 KIPS compression of the beam through zero to 22 KIPS (11 KIPS per bolt or 74% Fy) tension of the anchors. 105 cycles were run in this fashion. Plate deflection measured .133 inch.

ATTACHMENT

ASME CODE CASE N-258

CASES OF ASME BOILER AND PRESSURE VESSEL CODE

Meeting of January 11, 1980
Approved by Council, March 17, 1980
Approved by ACI, March 5, 1980

This Case shall expire on March 17, 1983
unless previously annulled or reaffirmed.

Case N-258
Design of Interaction Zones for Concrete Containments, Section III, Division 2

Inquiry: What rules apply for the design of an interaction zone between a steel shell portion of a containment and the Section III, Division 2 concrete?

Reply: It is the opinion of the Committee that for Section III, Division 2 containments, the interaction zone may be designed using the following rules:

(1) Interaction zone is that portion of the concrete containment where concrete is used in conjunction with the steel shell for load resisting purposes.

(2) The steel shell portion of the interaction zone shall meet the requirements of Section III, Division 1, except testing shall be in accordance with CC-6000.

(3) The concrete containment in the interaction zone shall meet the requirements of Section III, Division 2.

(4) The design and analysis of the interaction zone shall be made considering the interaction of the steel shell and concrete. In the interaction analysis and design, anchorage shall be provided. Between anchor points, full bonding and absence of bonding between the Division 1 shell and the Division 2 concrete shall be considered. Justification shall be provided that intermediate bonding is not a limiting case.

See Fig. 1 on the next page for limitation of the interaction zone.

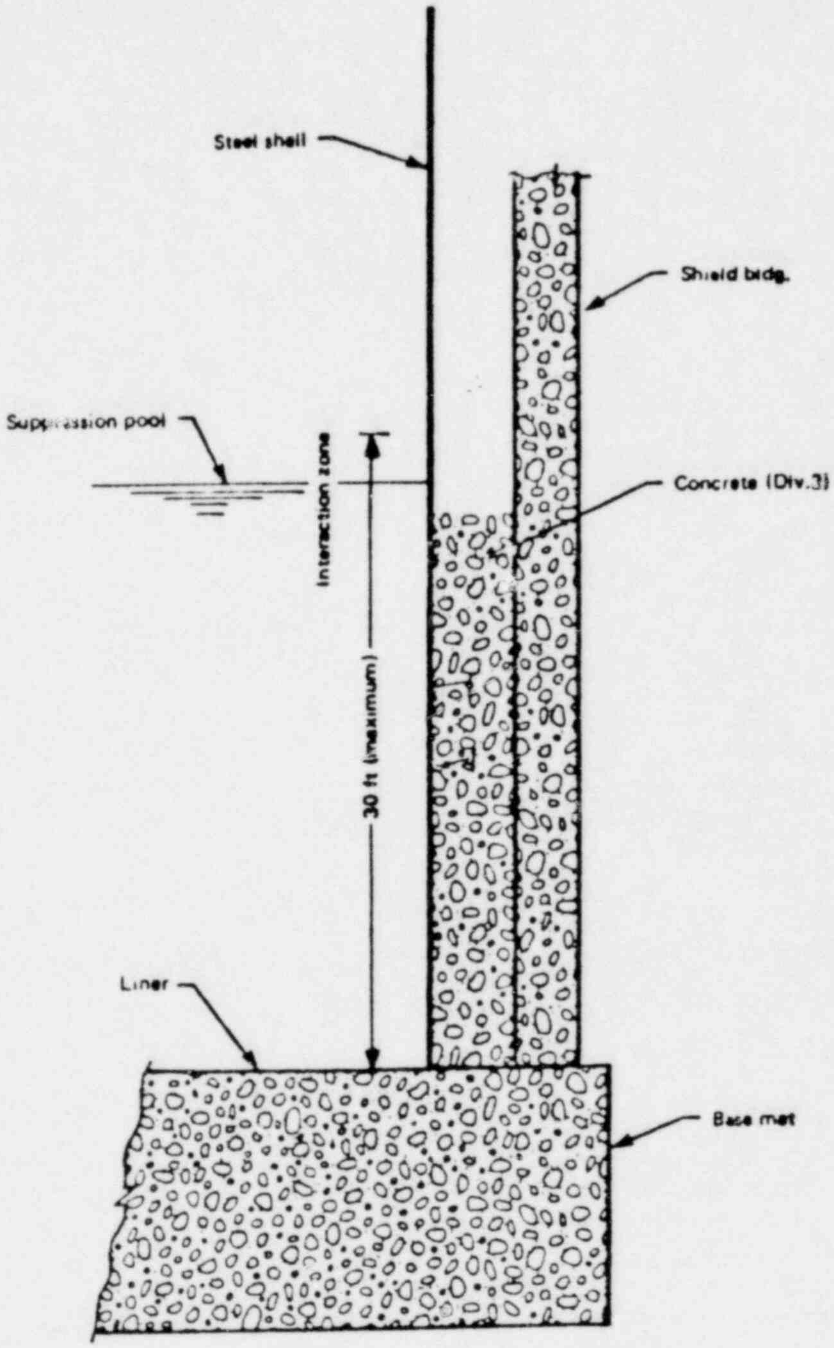

FIG. 1. LIMITATION OF THE INTERACTION ZONE