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RECIP.NAME RECIPIENT AFFILIATION
JOHNSON,A.R. Project Directorate I-3

SUBJECT: Lists items raised by NRC during 900906 site insp of plant containment.Gilbert/Commonwealth ltr re design basis encl.

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ROBERT C. MECREDY
Vice President
Ginna Nuclear Production

TELEPHONE
AREA CODE 716 546-2700

September 17, 1990

U.S. Nuclear Regulatory Commission
Document Control Desk
Attn: Allen R. Johnson
Project Directorate I-3
Washington, D.C. 20555

Subject: September 6, 1990 Site Inspection
R. E. Ginna Nuclear Power Plant
Docket No. 50-244

Dear Mr. Johnson:

This letter is a follow-up to the September 6, 1990 NRC site inspection of the Ginna Station Containment. Subsequent to the inspection of the Containment Building foundation, seven items were raised by the NRC staff. These are discussed below.

<u>Item 1:</u> <u>"Discuss the Condition of the Treated and Untreated Areas of the Inspected Annular Surfaces".</u>

<u>Response</u>: At this time, sixty percent of the annular surface has been coated with an epoxy based material. The remaining area is being worked on and will be completed by November 30, 1990. The treated areas are essentially free of any standing water. A few isolated areas show some sign of moisture and are being examined periodically to determine any changes. We believe the water in these areas is the residual water that has been absorbed by the celotex and is not groundwater exiting from the containment foundation slab.

When all areas of the annular surface have been coated, our plan is to establish a formal inspection program to monitor this area.

<u>Item 2:</u> <u>"What is the Purpose and Dimension of the Space Between Containment and Adjacent Structures?"</u>

<u>Response</u>: The space between containment and adjacent structures is to permit independent movement of each structure during a seismic event. This "rattle space" is nominally three inches, which has been confirmed in the field and is filled with a compressive material. The maximum predicted seismic displacements between these buildings is less than an inch.

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Item 3: "Discuss the Potential for Cracks in the Containment Wall"

Response: The possibility of cracks in the containment wall was raised by Mr. Ashar. In the sub-basement area, the entire visible surface of the containment is coated with approximately one-half inch of bitumastic sealer. Therefore, it is impossible to detect any cracks in the concrete. RG&E believes that removal of the bitumastic coating and performance of a detailed inspection is unwarranted at this time because those areas of containment that are visible and accessable are closely examined as part of the Integrated Leak Rate Test that is performed on the structure. No detectable stress cracks have ever been observed during any of those tests.

Item 4: "Determine the Condition of Intermediate Building Column Pedestal"

Response: Upon inspection, it was determined that the column cap was constructed as designed, is in sound condition, and continues to meet all design requirements.

Item 5: "Review the Attachments to the North Wall of Intermediate Building Sub-Basement"

Response: The pipe supports and the conduit that are attached to the north wall of the basement are all safety-related. Although they are attached to what appears to be a wall, the "wall" in fact is the foundation mat of the Turbine Building. The mat is heavily reinforced and more than adequate to carry any of these support loads.

None of the attachments are in direct contact with the small amount of groundwater that enters the area from below the foundation slab.

Item 6: "Cathodic Protection"

Response: A cathodic protection system was installed during initial construction of the plant. The intent of the system was to monitor any corrosion of the embedded steel (tendons, rock anchors, rebar, etc.). The system was tested periodically, the latest being in 1981. The results of that test indicated that the rate of corrosion of the steel was less than a gram per year.

Since this amount is negligible subsequent tests were judged unnecessary. However, RG&E intends to retest in 1991.

Item 7 "Response to Item 4 of NRC Letter dated July 9, 1990, 'Request for Additional Information'"

Response: As we observed the inflow of groundwater with time, we have gained better insight into its source. Most of the water in the annular space was entering from the two areas of the sheet piling that abut the Auxiliary and Intermediate Buildings. Very little was seeping through the sheet piling itself. Another source was found to be the gap between the cable tunnel roof slab and the Containment Building at grade. The sealing material that was originally installed was ineffective in stopping rainwater from flowing down the containment wall and into the sub-basement. This joint has since been re-sealed with the result that no rainwater is now entering the area.

As part of the SEP Program, RG&E committed to determine the actual groundwater levels at the plant. Monitoring wells were installed and the water level was measured for four years. The data showed the level to be at elev. 265 ft. (grade is elev. 270 ft.). As a result, all safety-related below grade structures (except the containment foundation slab) were analyzed and found to be acceptable. When the SEP issue on groundwater was addressed and closed out, data collection was discontinued.

RG&E has recently completed an analysis of the containment foundation slab. The slab was conservatively analyzed for the uplift loads associated with groundwater at elev. 270 ft. and found to be acceptable.

Also by way of this letter, we are transmitting to you the Gilbert/Commonwealth letter #13N1-GR-T5641 which presents the original design basis for various components of the containment.

Please note also this letter serves to document the delivery of a copy of the originally docketed report "Structural Integrity Test of the Reactor Containment Structure" to Mr. Ashar of your staff by Mr. Sucheski of RG&E at their meeting in your office.

Very truly yours,

Robert C. Mecredy

Robert C. Mecredy

LAS/166
Enclosure

xc: Mr. Allen R. Johnson (Mail Stop 14D1)
Project Directorate I-3
Washington, D.C. 20555

U.S. Nuclear Regulatory Commission
Region I
475 Allendale Road
King of Prussia, PA 19406

Ginna Senior Resident Inspector

Gilbert/Commonwealth engineers and consultants
GILBERT ASSOCIATES, INC., P.O. Box 1498, Reading, PA 19603 / Tel. 215-775-2600 / Cable Gilasoc / Telex 836-431

August 15, 1990

Mr. A. Gary Goetz
Rochester Gas & Electric Corporation
49 East Avenue
Rochester, New York 14649

Attention: Mr. Leonard Sucheski

Re: R. E. Ginna Station
Evaluation of Containment
Cylinder-to-Base Joint
Ref: 13N1-GR-T5633, July 30,. 1990

Dear Mr. Goetz:

Attached for your information are the following items relative to the design of the hinged joint at the bottom of the containment cylindrical wall. This information which has been condensed resulted from our review of the Updated FSAR, all of the original design calculations and construction drawings, specification, and the seismic upgrade evaluation report.

The individual Data Sheets provide pertinent information regarding each hardware item involved in the hinged joint design as well as the items in the load transfer path below the hinge down to rock and in the cylindrical shell immediately above the hinge. The attached figures show the key elements in the hinge area.

A summary discussion regarding how the various forces are transferred across the hinged joint is also attached. Finally, a discussion on the methods used to determine the seismic loads on the containment and the magnitudes of those forces (base shear) based on a review of 1979 seismic upgrade as well as the original design.

The above information has been discussed with your Mr. L. Sucheski and Mr. D. Zebrowski during a meeting at your offices on August 7, 1990.

Should you have any questions regarding this information, please contact us at your convenience.

Sincerely,

R. H. Moyer
R. H. Moyer
Project Structural Engineer

Donald K. Shetter
for
J. G. Shingler
Project Manager

cc: D. Zebrowski (RG&E)
Jennifer A. McGuire (RG&E)
D. D. Krause
P. J. Rieck

13N1-GR-T5641

CONTAINMENT VESSEL FOR R. E. GINNA POWER PLANT

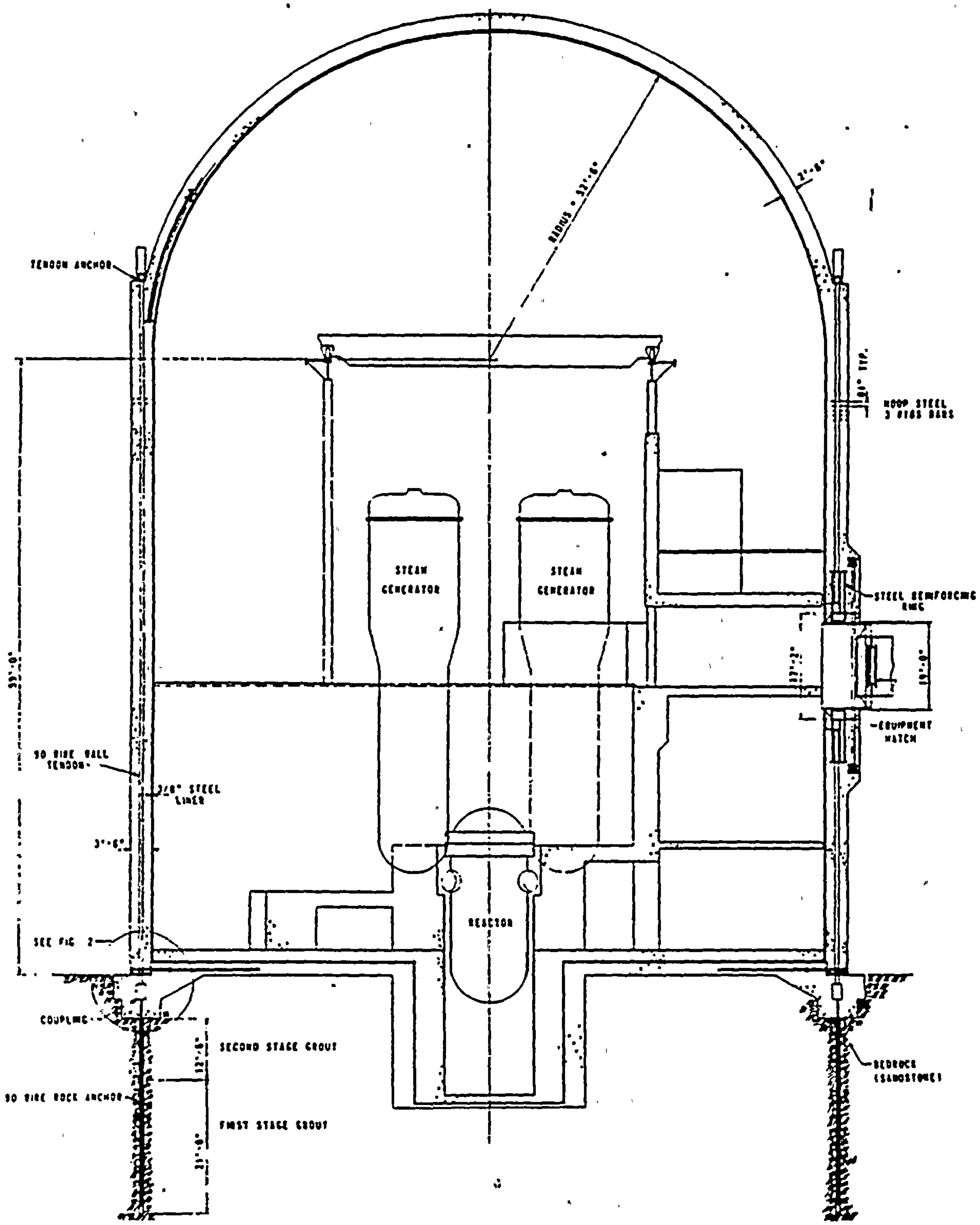
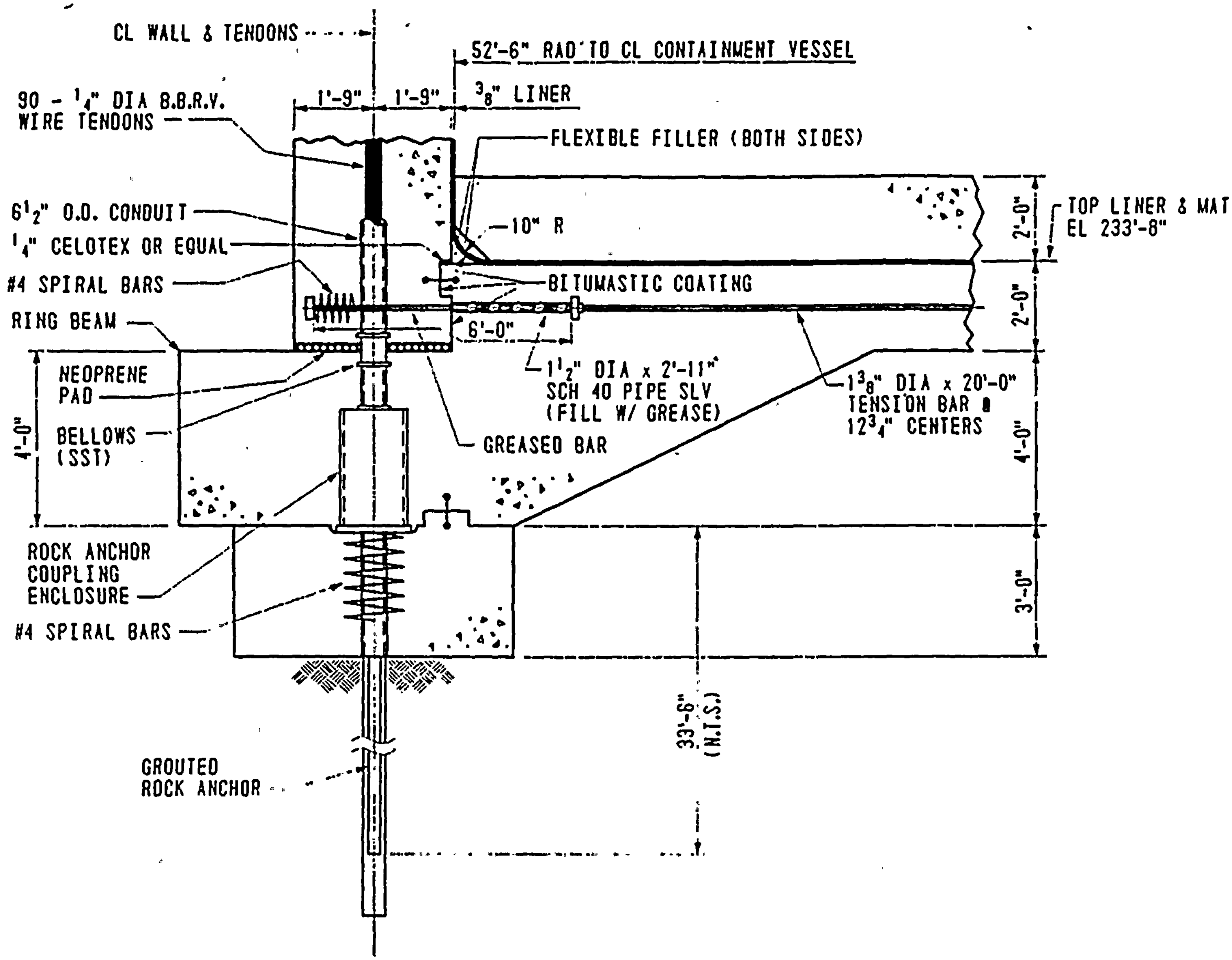
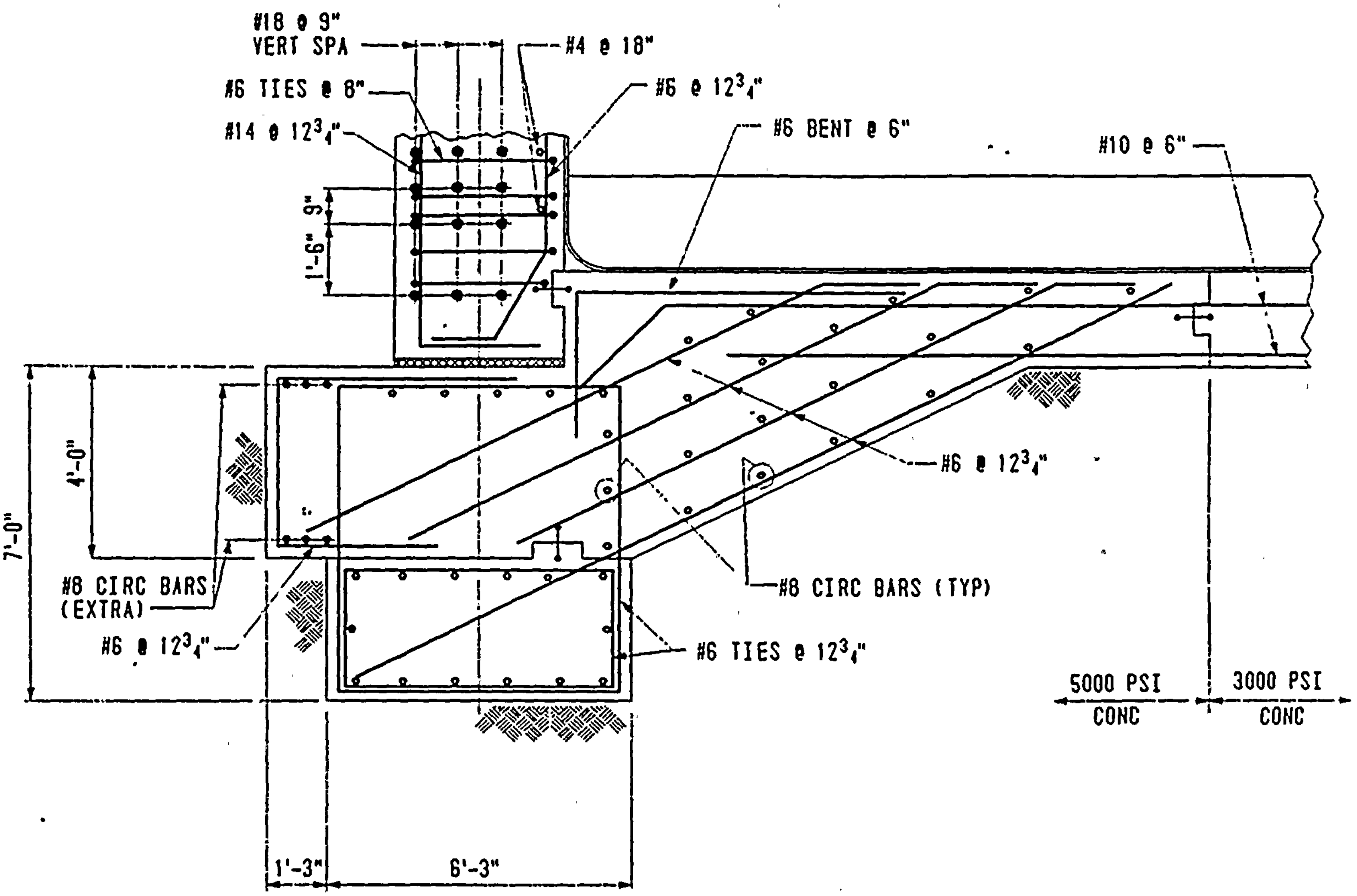
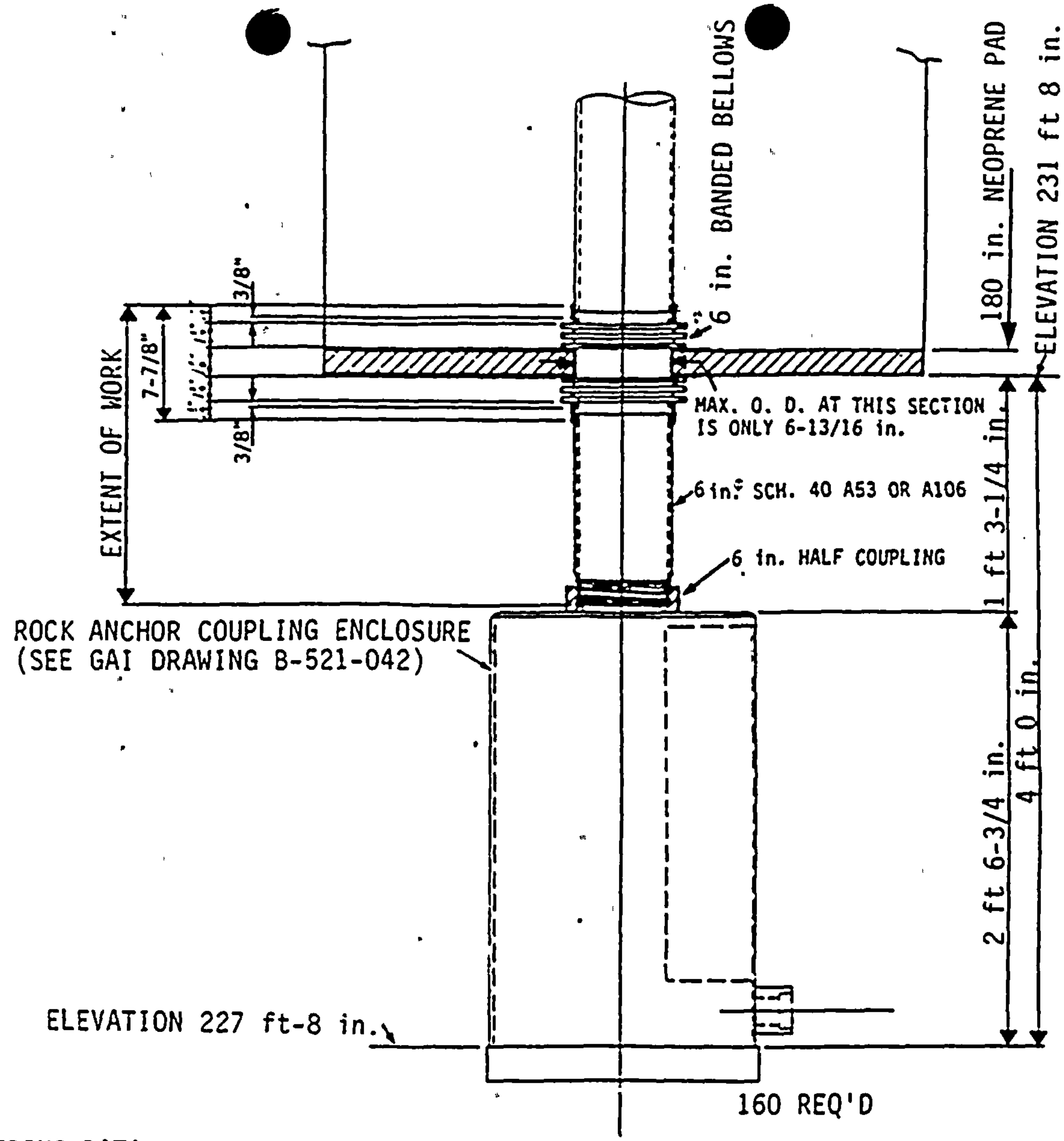


Fig. 1. Cross section of containment vessel.


CL WALL & TENDONS
52'-6" RAD TO CL CONTAINMENT VESSEL
90 - 1/4" DIA B.B.R.V. WIRE TENDONS
1'-9"
1'-9"
3/8" LINER
FLEXIBLE FILLER (BOTH SIDES)
6 1/2" O.D. CONDUIT
10" R
TOP LINER & MAT EL 233'-8"
2'-0"
1/4" CELOTEX OR EQUAL
#4 SPIRAL BARS
BITUMASTIC COATING
2'-0"
RING BEAM
6'-0"
1 1/2" DIA x 2'-11" SCH 40 PIPE SLV (FILL W/ GREASE)
1 3/8" DIA x 20'-0" TENSION BAR @ 12 3/4" CENTERS
NEOPRENE PAD
4'-0"
BELLOWS (SST)
GREASED BAR
4'-0"
ROCK ANCHOR COUPLING ENCLOSURE
3'-0"
#4 SPIRAL BARS
33'-6" (N.T.S.)
GROUTED ROCK ANCHOR



#18 @ 9"
VERT SPA
#4 @ 18"
#6 TIES @ 8"
#6 @ 12¾"
#14 @ 12¾"
#6 BENT @ 6"
#10 @ 6"
9"
9"
1'-6"
4'-0"
7'-0"
#6 @ 12¾"
#8 CIRC BARS
(EXTRA)
#8 CIRC BARS (TYP)
#6 @ 12¾"
#6 TIES @ 12¾"
5000 PSI
CONC
3000 PSI
CONC
1'-3"
6'-3"



ENGINEERING DATA

1. MOVEMENTS: CASE (1) FROM UNDEFLECTED POSITION VERTICALLY DOWNWARD 0.14 INCHES.
   CASE (2) FROM ABOVE POSITION VERTICALLY UPWARD 0.10 INCHES AND SIMULTANEOUS LATERALLY 0.16 INCHES.
2. FATIGUE: TWO CYCLES PER YEAR.
3. WORKING PRESSURE: 60 psig.
   TEST PRESSURE: HYDROSTATIC AT 150% OF WORKING PRESSURE.
   PNEUMATIC AT 125% OF WORKING PRESSURE.
4. MAXIMUM WORKING TEMPERATURE: 160°F.
5. STANDARD SPECIFICATION: ASA B31.1 CODE FOR PRESSURE PIPING.
6. TEST TWO RANDOM ASSEMBLIES FOR SPECIFIED MOVEMENTS.

ROCHESTER GAS AND ELECTRIC CORPORATION
R.E. GINNA NUCLEAR POWER PLANT
UPDATED FINAL SAFETY ANALYSIS REPORT

Figure 3.8-18

Containment Miscellaneous Steel
Tendon Conduit - Hinge Detail

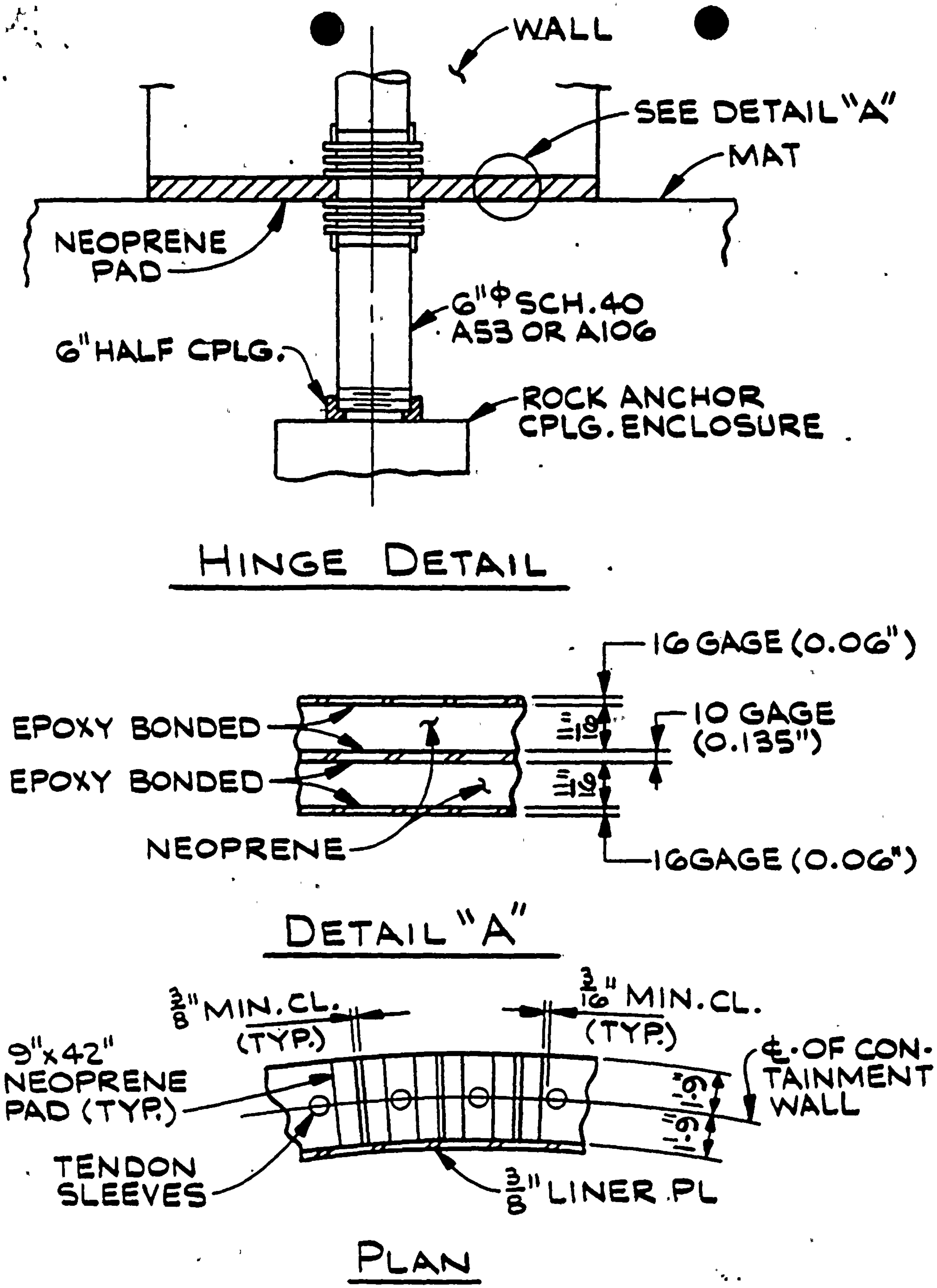


FIGURE 2.8–1
ELASTOMERIC PADS

DATA SHEET

ITEM: Bellows

PHYSICAL DESCRIPTION: 6 inch diameter, stainless steel pipe bellows complying with requirements of ASA B31.1 Code for Pressure Piping.

FUNCTION: Provides movement capability for the rigid tendon conduit at the hinged joint in order to ensure a sealed tendon enclosure which retains the grease corrosion protection around the tendon and also seals against contaminants gaining access to the tendons. The bellows also provides essentially no resistance across the hinged joint to the movements.

DESIGN REQUIREMENTS:

(1) Design Condition Movements:

Case 1: From undeflected position, vertically downward 0.14 inches.

Case 2: From above position, vertically upward 0.14 inches, and simultaneously laterally 0.16 inches.

(2) Fatigue: Two cycles per year.

(3) Working Pressure: 60 psig
Test Pressure: Hydrostatic at 1.5 Working Pressure
Pneumatic at 1.25 Working Pressure

(4) Maximum Working Temperature: -160°

MARGIN: Maximum calculated radial outward displacements at hinges are:

0.10 inches at Design Pressure (60 psig)
0.11 inches at 1.15 Design Pressure
0.15 inches at 1.5 Design Pressure
0.12 inches at 1.25 P + E (0.1g)

The margin for the bellows design is conservative since the bellows were designed for the specified movements as Design Conditions while the maximum calculated displacements listed above are for factored load conditions. The margin or factor of safety is at least 1.6.

DATA SHEET

ITEM: Concrete/Reinforcing Steel

PHYSICAL DESCRIPTION:

Concrete compressive strength for the containment shell and ring girder base is 5,000 psi at 28 days.

Reinforcing typically conformed to ASTM A15-64 or ASTM A408-62T (for special large size bars) having tensile strength of 70 ksi to 90 ksi and minimum yield of 40 ksi.

FUNCTION:

The reinforced concrete provides a structure to enclose the nuclear steam generating system and provides a biological shield during normal plant operation. It also serves as an integral part of the containment structure protective barrier which is designed to contain the energy released during maximum hypothetical accident.

Membrane hoop tension forces in the cylindrical wall due to incident loads are taken by mild steel reinforcing.

Discontinuity forces in the meridional direction near the base of the cylindrical wall are taken by vertical reinforcing on the outside face.

Radial shear forces at the discontinuity due to incident loads are taken by the concrete as well as radial shear reinforcing designed in accordance with ACI 318 Articles 1701 and 1702.

DESIGN REQUIREMENTS AND MARGIN:

Reinforcing is provided to ensure that the integrity of the steel liner is not prejudiced by strains in the reinforced concrete.

Stress in mild steel reinforcement is limited to yield for all factored loads. A ϕ factor of 0.95 is also applied to reduce the permitted calculated capacity.

Initial average concrete membrane compression is 640 psi which even with a stress concentration factor of 3 yields acceptable stresses.

Bending and shear forces in the discontinuity region at the base of the cylinder are resisted by mild steel reinforcing.

Radial Shear - Critical section 3 feet above base. Shear reinforcing of #7 at 11 inch centers provided for controlling case of load condition c (60 psig plus 0.2g seismic). One

location controls. Required area of reinforcing is 0.494 square inches; 0.60 square inches was provided.

Longitudinal Shear - Assumes concrete cracks through the thickness due to horizontal membrane tension (970 psi for 60 psig pressure). The horizontal reinforcing is checked to carry the longitudinal shear due to the maximum hypothetical earthquake by dowel action. The bars are checked for combined tension and shear on the bar. For the controlling case (60 psig plus 0.2g seismic) the tensile stress is 25.4 ksi and the actual shear stress is 8.7 ksi compared to 14.1 ksi shear capability (corresponding to 25.4 ksi tension).

Horizontal Shear - Due to lateral seismic loads is transferred to the containment wall by the radial tension bars acting as spokes in a wheel. This approach assumes the wall acts as a stiff ring at the base. Minor bending and shear are produced in the stiff wall ring section.

Summary of concrete and reinforcing stresses at 3 feet above base for elastic behavior assumption. Worst case for hoop and meridional directions is for 1.5P load condition.

Meridional		Hoop
Concrete	Rebar	Rebar
-1.08 ksi	20.7 ksi	18.4 ksi

DATA SHEET

ITEM: Liner Knuckle

PHYSICAL DESCRIPTION: The liner knuckle is a 3/8 inch thick quarter round segment with 10 inch radius located at the base of the cylindrical wall liner.

FUNCTION: Provides flexibility at the joint of the cylindrical wall liner and base liner to accommodate radial and vertical movements at the hinge. A flexible filler material is placed on either side of the knuckle to act as a cushion permitting deflection of the knuckle during tensioning and for design loads.

DESIGN REQUIREMENTS AND MARGIN:

(1) Under prestress plus dead load the maximum downward vertical movement is 0.08 inches. Maximum bending stress in the knuckle is 25 ksi.

(2) For load combination with 1.5 accident pressure the upward movement is 0.08 inches, and radial outward movement is 0.08 inches. Maximum bending stress is 10 ksi and membrane stress is 1.2 ksi.

(3) For the lateral static loads associated with the maximum earthquake (0.2g) the maximum shear stress in the knuckle is 16.4 ksi using conservative fixed boundary conditions for the model.

DATA SHEET

ITEM: Liner Plate/Anchor System

PHYSICAL DESCRIPTION:

Liner Plate: 3/8 inch in dome and cylinder
1/4 inch in base
ASTM A442-60T Grade 60
F_y = 32 ksi min.

Anchors: Typical cylinder liner anchors consist of continuous channels C3X4.1 at about 4 ft. 4 inch centers both horizontally and vertically. In addition for the bottom 17 feet of the cylinder Nelson Concrete Studs 5/8 inch diameter by 6 3/8 inch long as placed at about 1 ft. 5 inch centers each way.

The base liner is anchored to a 2 foot thick base mat and is also covered by a 2 foot thick concrete mat. The cylindrical wall liner is isolated from thermal effects due to accident temperatures by a continuous cover of insulation.

FUNCTION: The liner/anchor system provides the gas tight boundary for the containment system. The system has substantial strength characteristics which are utilized by strain compatibility in the design and analysis of the containment.

DESIGN REQUIREMENTS AND MARGIN:

Liner panels and anchors were designed to ensure yielding of the plate in membrane compression prior to elastic buckling.

Under normal conditions at end of plant life the maximum liner stress is -14,000 psi in the meridional direction. For factored accident conditions the maximum stress is 28.7 ksi tension in the hoop direction. The maximum permissible stress is yield of 32 ksi.

For earthquake load the maximum liner shear stress is 8.8 ksi compared to 18 ksi allowable. Therefore, the liner can resist in-plane earthquake shear stresses without contribution of the horizontal rebar dowel action.

Liner anchors are designed to take the differential shear across the anchor resulting from the decreased stiffness of an adjacent buckled liner panel. The resulting force is 1.98 k/in. compared to 2.98K/in. permissible.

The liner anchors also are designed to transfer the total shear from the 3 ft. 6 inch thick concrete shell to the liner under 0.2g earthquake. This shear is a maximum of 0.58 K/in. compared to 2.98 K/in. capacity.

DATA SHEET

ITEM: Neoprene Bearing Pad Assembly

PHYSICAL DESCRIPTION: Each assembly consists of two 11/16 inch thick 55 durometer neoprene pads that are epoxy bonded between three carbon steel shims. Outer shims are 16 gage and center shim is 10 gage carbon steel. Each assembly is 9 inches x 42 inches in plan. Overall thickness is 1.628 inches thick. Two assemblies are placed between each tendon at the base of the cylindrical wall.

FUNCTION: These flexible assemblies permit relatively unrestrained rotation at the hinged joint as well as provide negligible shear resistance to minor radial movement.

DESIGN REQUIREMENTS AND MARGIN:

(1) Shear stress due to 0.16 inch lateral movement is 15 psi which is much less than 100 psi shear allowable for the neoprene. Shear modulus was based on physical testing.

(2) Compressive strain is limited to providing a maximum displacement of 15% of the elastomer thickness. For the maximum load of 894 kips due to dead plus initial prestress on a pair of 9 inch by 42 inch pads, this load produces a 1181 psi bearing stress and 0.12 inch displacement. The permitted displacement is 15% x 2 x 11/16 inch = 0.21 inches. The factor of safety on allowable compressive strain is about 2.

(3) Tests were performed on specimens of the neoprene material to ensure compliance with specifications for: (1) original physical properties including tear resistance, hardness, tensile strength, and ultimate elongation, (2) change in physical properties due to overaging, (3) extreme temperature characteristics, (4) ozone cracking resistance, (5) oil swell, (6) shear modulus.

Two full size pads were tested, one for creep and one for ultimate load. At load of 2000 kips corresponding to 5.3 times the design load the test was terminated without failure. Rebound of material was essentially complete.

DATA SHEET

ITEM: Tension Bar Assemblies

PHYSICAL DESCRIPTION: Each bar assembly is approximately 20 feet long and consists of a 1 3/8 inch diameter bar with two anchor plates. The bar conforms to ASTM A322-64a and ASTM A29-64 with the following physical properties:

Ultimate Strength	145 ksi
Yield Stress (2% Offset)	130 ksi
Elongation at Rupture in 20 Dia.	4%
Reduction of Area at Rupture	15%

The anchorage plates conform to AISI C-1040 and develop 100% of the bars ultimate strength. The assemblies are oriented radially at about 1 foot 1 inch centers tying the base of the cylindrical wall to the outer portion of the mat adjacent to the ring beam.

FUNCTION: The bars were designed to serve two functions as part of the hinge design. First, transfer horizontal forces due to incident pressures and seismic inertia forces from the cylindrical wall into the foundation. The tensile forces is carried by the bar. The force transfer is accomplished through the two anchor plates.

Secondly, the assembly is greased (unbonded) for the 6 foot length between the two anchor plates to permit radial outward movement of the wall corresponding to the elongation of the bars. This deformation is less than the permitted shear deformation of the elastomeric bearing pads. The radial movement along with the rotation permitted by the hinge design eliminates negative restraining moments in the lower wall as well as reduces radial shears.

For the original design concept, the horizontal displacement due to seismic excitation produces a base shear which is transferred via the base mat to the side walls by the radial bars in tension. As the earthquake shear is imposed the bars react similar to a wheel with prestressed spokes with a load applied to the hub and the rim is restrained from moving. The load transfer from the radial bars to the wall occurs by varying circumferential membrane forces in the lower portion of the wall.

DESIGN REQUIREMENTS AND MARGIN:

L.C.a = Operating plus incident loads
L.C.b = Operating plus incident plus design seismic
L.C.c = Operating plus incident plus max. potential seismic

STATIC SEISMIC

Load Combination	Bar Force kips	Bar Stress ksi	% Yield Strength	Seismic Hor. Accel.	% Bar Stress Due to Seismic
a	130	87.5	67	-	-
b	149	100.0	77	0.23g	25%
c	170	114.3	88	0.46g	44%

Factor of Safety (vs. Yield) = 1.14

DYNAMIC MODAL ANALYSIS SEISMIC

Load Combination	Bar Force kips	Bar Stress ksi	% Yield Strength	Seismic Hor. Accel.	% Bar Stress Due to Seismic
c	137.5	92.4	71	0.26g	25%

Factor of Safety (vs. Yield) = 1.4

STRUCTURAL EVALUATION PROGRAM SEISMIC
(Ref. UFSAR 3.8.2.2.7.3)

Maximum tensile stress in rod for combined load case (incident pressure and temperature) is about 54 ksi providing a factor of safety of 2.6 versus yield.

The development of the tension bars by the concrete provides 320 kips which represents a substantial margin of safety.

DATA SHEET

ITEM:	Rock
PHYSICAL DESCRIPTION:	Sandstone - fine grained unconfined compressive strength greater than 8,000 psi.
FUNCTION:	Provides the bond strength to anchor the prestressed rock anchors. Rock mass provides the resistance to uplift caused by external accident pressure and earthquake forces. Provides vertical bearing resistance under the ring beam at the base of the cylindrical wall. Provides lateral bearing resistance at the ring beam.
DESIGN REQUIREMENTS AND MARGIN:	For bond strength and uplift, see rock anchor description. For bearing under the ring beam, allowable pressure is 35 tons per square foot. For lateral resistance at side of ring beam the allowable is 25 ksf, versus 24 ksf for the factored load combination of operating plus incident.

DATA SHEET

ITEM:	Rock Anchors
PHYSICAL DESCRIPTION:	160 vertical rock anchors at about 2 ft. 1 1/2 inch centers are provided at about 54 ft. 3 inch radius. Each consists of 90-1/4 inch diameter wires. Wires conform to ASTM A421-65BA with F_y = 190 ksi, minimum (1% elong.) and F_{ULT} = 240 ksi, minimum.
FUNCTION:	The side walls of the containment are anchored to the foundation rock with the prestressed rock anchors which place a preload in the foundation rock and ring beam at the base of the cylindrical wall. The post-tensioned vertical tendons in the wall are coupled to the rock anchors. Vertical forces in the containment due to incident pressure and earthquake are transferred directly to the foundation rock by the anchors.
DESIGN REQUIREMENTS AND MARGIN:	Tests were performed to establish the permissible 170 psi bond stress between the grouted tendon and rock which was used to establish the embedment length. A factor of safety of at least 2 against slippage is provided. The rock anchors were prestressed to 0.8 F_{ULT} and locked off at 0.7 F_{ULT}. Maximum force in the tendons occurs at initial prestress. The coupling between the rock anchor and tendon is designed to remain elastic for the force corresponding to the ultimate strength of the tendon wire. The rock anchors are sized based on the design assumption that the anchor pull is resisted only by the submerged weight of rock. The weight of rock is calculated assuming breakout at 45° to the center of the first stage grouting. Rock tensile strength is assumed to be zero. The factor of safety against "overturning" is calculated to assure no net uplift force. The approach using earthquake forces obtained from single lumped mass model and statically applied is conservative. Also, checking stability in this manner using factored load conditions is conservative.

STATIC SEISMIC

Load Condition	Factor of Safety
a	1.26
b	2.38
c	1.96
Seismic (0.2g)/ no pressure	2.31
Structural Proof Test	1.47

DYNAMIC MODAL ANALYSIS

b	3.04
c	1.64
0.2g seismic/ no pressure	3.06

DATA SHEET

ITEM: Prestressing Tendons

PHYSICAL DESCRIPTION: 160 vertical tendons are provided in the cylindrical wall. Each consists of 90 1/4 inch diameter high strength steel wires conforming to ASTM A421-59T, Type BA with F_{ULT} = 240 ksi minimum and F_y = 192 ksi minimum. The tendons are positioned at the center of the 3 foot 6 inch thick cylindrical wall and are anchored at the top, 12 feet 6 inches above the spring line of the dome and coupled to the prestressed rock anchors at the base.

FUNCTION: The tendon system is designed to provide sufficient vertical prestress force so there are no net meridional membrane tensile forces in the concrete shell under application of design load combinations.

The prestress serves to balance the vertical tensile membrane forces thus allowing the use of the provisions of ACI 318 Section 1701 and 1702 for shear reinforcement design.

The coupling of vertical tendons to the rock anchor provides a direct path for transfer of net vertical forces to the rock foundation without introducing bending moments into the ring beam under the cylindrical wall.

DESIGN REQUIREMENTS AND MARGIN: Each tendon is tensioned initially to 0.8 F_{ULT} and locked off at 0.7 F_{ULT}. The maximum force experienced by the tendon occurs during initial installation.

Effective prestress is always less than 0.6 F_{ULT} or 0.8 F_y whichever is smaller.

Protype tendons were tested for fatigue (cyclic) and dynamic loads.

LOAD TRANSFER AT HINGED JOINT

The primary load conditions addressed in the design of the containment were:

A. $C = 0.95D + 1.5P + 1.0T$

B $C = 0.95D + 1.25P + 1.0T' + 1.25E$

C $C = 0.95D + 1.0P + 1.0\underline{T} + 1.0E'$

The hinged joint design was used to eliminate negative restraining moments at the base of the cylindrical wall and to reduce radial shears substantially.

Load transfer at the hinge as described in the Ginna UFSAR is as follows:

1. Vertical Forces due to dead, prestress, pressure, and seismic are transferred by either compression, bearing on the elastomeric bearing pads and thence to the ring beam and rock foundation, or by tension through the prestressed tendons located at the center of the cylindrical wall and directly to the attached prestressed rock anchors and into the rock.

2. Vertical or Meridional Moments - Due to the rotational capability provided by the elastomeric bearing pad, the hinged joint is free to rotate about a horizontal axis thus eliminating development of restraining moments at the hinge. The moment caused by the stiffness of the bearing pad corresponding to the rotation calculated for 1.5P condition is approximately 45 FT-K/FT. From UFSAR Figure 3.18-13 maximum moment is 412 FT-K/FT at about 10 ft. above the base. Therefore the moment from the pad which is accounted for in the design, is not significant compared to moments from other loads.

3. Radial Forces - The neoprene pad is relatively flexible in shear and will deform as required for compatibility without providing significant shear resistance. Horizontal shear forces at the hinge level are generated primarily by two basic loads - Incident Pressure and Earthquake.

   For design of the concrete cylinder (UFSAR 3.8.1.4.1) seismic shear distribution was assumed to flow perpendicular to the containment radius (;i.e. tangential) with a maximum equal to twice the average (and presumably occurring at 90° to direction of earthquake).

   On UFSAR page 3.8-25 it states that the base shear is transferred via the base mat to the side walls by the radial reinforcement, i.e. tension bars. The bars will react similar to a wheel with prestressed spokes with a load applied at the hub and the rim restrained from moving. The bars are assumed to have no shear resistance. "The load transfer from the radial bars will occur by means of varying circumferential membrane forces in the lower portion of the wall". The wall is assumed to be a stiff ring. This assumption gives maximum forces in the bars. The bending and shear in the assumed ring section of the wall due to differential radial tension bar forces are minimal.

For axisymmetric radial forces resulting from pressure load the tension bars are designed to accept the radial, horizontal reaction at the base of the cylinder wall while providing an acceptable elongation of each bar over the 6 ft. unbonded length between the cylindrical wall and ring beam. This movement also has the beneficial effect of reducing radial shear in the concrete at the discontinuity.

On page 3.8-45 of the FSAR, it states that the base slab serves only as the anchorage point for the radial tension bars and is not an integral part of the containment shell for this design. The horizontal base reaction is transferred by tension into the ring beam and then laterally by bearing to the rock abutting the outside face of the ring beam.

The 3/8 inch plate liner knuckle offers negligible restraint to radial movement of the cylinder wall. However, the knuckle is relatively stiff for lateral earthquake loads. For the 0.2g earthquake the maximum shear stress in the knuckle is conservatively calculated at 16.4 ksi while bending stresses are low (UFSAR page 3.8-45). This is understood to mean that the knuckle has been checked for the worst case where it is assumed to function in shear transfer in accordance with an assumed model and stiffness.

For the SEP seismic reanalysis of the containment it was determined that the circumferential stiffness of the shell was much higher than the radial stiffness and, therefore, the lateral seismic loads were represented by tangential loads.

The SEP reanalysis determined that the maximum force in the radial tension bars due to temperature and pressure loads was 54 ksi compared to 130 ksi yield stress (page 3.8-142 UFSAR). The reanalysis stated that radial <u>tension</u> loads are resisted by the radial tension rods but radial <u>compression</u> loads in the inward direction are resisted by the concrete base slab in baring. Presumably, the only loading that results in radial compression are seismic and therefore the SER evaluation is stating that concrete bearing on the base slab is a load path for seismic force across the hinged joint. Further, no shear stress occurs in the rods due to the design clearance between the rod and sleeve. The rod is 1 3/8 inch O.D. The 1 1/2 inch diameter schedule 40 pipe has an I.D. of 1.610 inches.

Summarizing, the radial tension bars are the only means available to take the radial outward forces at the base of the containment wall resulting from pressure and temperature loads. The compressed neoprene pads are very flexible in shear and cannot be relied upon to transfer horizontal forces. However, if the assumption is made that the tension bars are unavailable, the primary restraint to radial growth becomes the horizontal #18S reinforcing through the hoop membrane forces. The radial movement at the base hinge could approximate the maximum horizontal deflections higher up on the cylindrical wall which are:

Load Condition	Radial Deflection Wall
A. C = .95D + 1.5P + 1.0T	0.86 inches
B. C = .95D + 1.25P + 1.0T′ + 1.25E	0.72 inches
C. C = .95D + 1.0P + 1.0$\underline{T}$ + 1.0E′	0.59 inches

With deflections of this magnitude compared with design deflections at the base on the order of 0.1 inches, inelastic behavior of the liner anchor system would require evaluation to assure leak tightness. Secondly, the question of how the lateral seismic loads would be transferred across the joint in this condition must also be addressed. If the transfer is by compression between the inside face at the base of the shell and the base mat, then it is assumed that the containment base is sliding across a gap to make contact with each oscillation of the earthquake. In this case the shear strain in the neoprene base pad could be excessive resulting in a loss of integrity of the base pad.

If the transfer is by shear through the liner knuckle into the base liner-anchor system, the inelastic behavior of the liner-anchor system must be evaluated and determined whether adequate capacity is available to transfer the loads to the base mat.

If the base of the cylinder is unrestrained radially, i.e. no tension bars, and a larger gap occurs between the inside face of the base of the shell and the base mat, the shell will be oscillating laterally on the neoprene pads with the beneficial result of isolating the containment shell from the seismic input motion at the base. The resulting inertia forces in this case will be significantly lower than the design forces.

RG&E GINNA STATION
CONTAINMENT BUILDING SEISMIC ANALYSES

ORIGINAL STATIC ANALYSIS

THIS ANALYSIS CONSISTED OF MODELING THE CONTAINMENT AS A SINGLE DEGREE OF FREEDOM SPRING/MASS MODEL. TWO MODELS, ONE FOR HORIZONTAL AND ONE FOR VERTICAL, ARE USED.

1. BASED UPON HOUSNER SPECTRA FOR 2% DAMPING.
2. GROUND ACCELERATIONS: OBE - 0.08 G
   SSE - 0.20 G
3. VERTICAL ACCELERATIONS SAME AS HORIZONTAL.
4. FREQUENCIES: HORIZONTAL - 4.54 HZ. Sa = 0.44 G (PEAK OF HOUSNER SPECTRA = 0.46 G IS USED FOR DESIGN)

   VERTICAL - 14.0 HZ. Sa = 0.29 G
5. DESIGN FORCES: CONTAINMENT WEIGHT = 24,170 K.
   BASE SHEAR = 11,100 K
   = 32.5 K/FT (AVERAGE)
   OVERTURNING= 1,110,000 FT-K
6. OVERTURNING MARGIN = 1.05
7. MERIDIONAL FORCES:

D.L.	-		71 K/FT
VERT. EQ.	-	(+/-)	20.6 K/FT
HORIZ. EQ.	-	(+/-)	120 K/FT

REFERENCES:

1. CALCULATION "ORIGINAL STATION DESIGN - CONTAINMENT VESSEL", BOOK 2 OF 3.

PAGE I-A-1 (Items above 5,7)
I-B-1 (1,4,7)
I-B-2 (1,4,5)
I-B-4 (7)
I-K-2 (6)
I-E-27,34 (7)

2.FSAR

3.7.1.2 (1,2,3)
3.7.2.6 (3)
3.8.1.3 (1)
FIG. 3.8-9 (7)
TABLE 3.8-5 (7) (THIS TABLE IS SCALED TO 1.0 G)

RG&E GINNA STATION
CONTAINMENT BUILDING SEISMIC ANALYSES

ORIGINAL MODAL ANALYSIS (1969)

THIS CONSISTS OF A 12 MASS STICK MODEL OF THE CONTAINMENT. THE PURPOSE OF THE ANALYSIS WAS TO IMPROVE UPON THE OVERTURNING FACTOR OF SAFETY. SOIL SPRINGS WHEN ATTACHED DID NOT AFFECT TOTAL MOMENT OR SHEAR BY MORE THAN 5% COMPARED TO FIXED BASE.

THE COMPUTER PROGRAM "SAND" FROM JPL IS USED TO CALCULATE FREQUENCIES. THE COMPUTER PROGRAM SPECTA IS USED TO COMBINE MODAL RESPONSES USING SRSS.

SHEAR RESULTS FROM THIS ANALYSIS ARE USED TO EVALUATE LINER (REF.1, PAGE II-A-1).

1. BASED UPON HOUSNER SPECTRA FOR 2% DAMPING.

2. GROUND ACCELERATIONS: OBE - 0.08 G
   SSE - 0.20 G

3. VERTICAL (0.20 G) SEISMIC NOT ANALYZED DYNAMICALLY, MAY HAVE USED STATIC APPROACH SOMEWHERE.

4. FREQUENCIES: HORIZONTAL, F1 = 6.95 HZ. (FIXED BASE)
   = 6.28 HZ. (SOIL SPRNG)

5. DESIGN FORCES: SSE: SHEAR = 6,700 K.
   = 19.7 K/FT.
   OVERTURNING = 850,000 FT-K.

   OBE: OBE = (0.08/0.20) X'S SSE

NOTE: EFFECTIVE HORIZONTAL G = 0.28 G IS BASED ON RATIO WITH 1,100 K BASE SHEAR USING 0.46 G. FSAR 3.8.1.3 STATES 0.26 G AS EFFECTIVE FOR THIS ANALYSIS.

6. OVERTURNING MARGIN = 1.64

7. MERIDIONAL FORCES: VERT.EQ. = (+/-) 14.1 K/FT
   HORIZ.EQ. = (+/-) 76 K/FT
   NOTE:VERT.= (0.20)(24,170)/341

REFERENCES:

1. CALCULATION "ORIGINAL STATION DESIGN - CONTAINMENT VESSEL", BOOK 2 OF 3.

PAGE I-K-5,6 (5,6)
II-A-1 (5) (FOR SHEAR IN LINER DESIGN)

2.FSAR
3.8.1.3 (1,2,3,4,5)
FIG. 3.8-10,11,12 (5)

3. NUREG/CR-1821 LLNL SEP EVALUATION, DEC., 1980.
PAGE 55, FIGURE 24 (7)

RG&E GINNA STATION
CONTAINMENT BUILDING SEISMIC ANALYSES

LLNL SEP ANALYSIS (NOV., 1980)

THIS SEISMIC ANALYSIS IS DONE BY LLNL FOR THE NRC AS PART OF THE NRC SYSTEMATIC EVALUATION PROGRAM.

THIS ANALYSIS USES THE SAME MODEL AS THE ORIGINAL GINNA 12 MASS MODAL ANLYSIS USED.

1. BASED ON USNRC RG 1.60 SPECTRA FOR 7% DAMPING
2. GROUND ACCELERATIONS: 0.20 G (HORIZONTAL, SSE ONLY)
3. VERTICAL SEISMIC = 2/3 HORIZONTAL = 0.13 G (REF. 3, PG 53). ASSUMED RIGID IN VERTICAL DIRECTION.
4. FREQUENCIES: F1 = 6.97 HZ
5. DESIGN FORCES: SSE: SHEAR = 9,505 K
   = 27.9 K/FT
   OVERTURNING = 923,500 FT-K
6. OVERTURNING MARGIN: NOT FOUND CALCULATED, = 1.51 USING PREVIOUS APPROACH.
7. MERIDIONAL FORCES: VERT. EQ. = (+/-) 12 K/FT
   HORIZ. EQ. = (+/-) 100 K/FT
   NOTE: VERT = 0.17 G, NOT 0.13 G

REFERENCES.

1. NUREG/CR-1821 LLNL SEP EVALUATION, DEC., 1980.

PAGE 53 (1,2)
PAGE 21 (3)
PAGE 55 (4), FIGURE 24 (7)

2. FSAR

3.8.2.2.4,5 (1,2,3)
TABLE 3.8-17 (4)
TABLE 3.8-18 (5) (SCALED UP BY .20/.17)

RG&E GINNA STATION
CONTAINMENT BUILDING SEISMIC ANALYSES

SITE SPECIFIC EARTHQUAKE

THE SITE SPECIFIC SEISMIC ANALYSIS IS USED FOR THE SEP STRUCTURAL REVIEW. THIS ANALYSIS USES THE LLNL ANALYSIS RESULTS AND SCALES THEM TO 0.17 G SSE HORIZONTAL AND 0.11 G VERTICALLY.

1. BASED ON USNRC RG 1.60 SPECTRA FOR 7% DAMPING
2. GROUND ACCELERATIONS: 0.17 G HORIZONTAL
3. VERTICAL SEISMIC = 2/3 HORIZONTAL = 0.11 G
4. FREQUENCIES: F1 = 6.97 HZ
5. DESIGN FORCES: SSE: SHEAR = 8,080 K
= 23.7 K/FT
OVERTURNING = 785,000 FT-K
6. OVERTURNING MARGIN: NOT FOUND CALCULATED, = 1.78 USING PREVIOUS APPROACH.
7. MERIDIONAL FORCES: NOT FOUND

REFERENCES:

1. FSAR

SECTION 3.8.2.2.4
TABLE 3.8-18

Gilbert/Commonwealth
ENGINEERS/CONSULTANTS
CALCULATION

PROJECT 29 TE GINNA STATION CONTAINMENT SEISMIC ANALYSES		IDENTIFIER	PAGE OF PAGES
REV. 0	1	2	3
MICROFILMED			
ORIGINATOR			
DATE /			

SITE SPECIFIC
1984 PIPING UPGRADE ANALYSIS (SSE)

WT (K) (1)	NP	ELEV.	HEIGHT	FORCE (2)	MOMENT
2017	299	374'	142	2017	286,414
1963	300	363'	131	1963	257,153
2634	301	343'	111	2634	292,374
2070	302	331'	99	2070	204,930
1710	303	320'	88	1710	150,480
3600	304	312'	80	3600	288,000
5040	305	280'	48	5040	241,920
4320	306	256'	24	4320	103,680
Σ 23,354		232'	0	Σ 23,354	1,824,951

SITE SPECIFIC (0.17G)
ANALYSIS: 1984 PIPING UPGRADE
MODE: 2 0.07 DAMPING
FREQUENCY: 7.26 Hz
DIRECTION: HORIZ. X

DAMPING: (3)
CONC. - 0.10
STEEL - 0.07
PRE-CONC - 0.07

BASE SHEAR = 0.17G x 23,354 = 3970 K
= 12 K/FT

OVERTURNING = 0.17G x 1,824,951 = 310,200 FT-K

REACTOR BLDNG SITE SPECIFIC PIPING UPGRADE
(1) BOOK DI/DV/DC-1 PG 5+
(2) " PAGE 12
(3) CO: 1:38.6 BOOK 1 OF 1 VOL 48C
SSE ANALYSIS.

GAI-446 7-84

Gilbert/Commonwealth ENGINEERS/CONSULTANTS CALCULATION	PROJECT RG&E GINNA STATION CONTAINMENT SEISMIC ANALYSES			IDENTIFIER	PAGE ___ OF ___ PAGES
	REV. 0	1	2	3	
	MICROFILMED				
	ORIGINATOR				
	DATE /				

1980 PIPING UPGRADE ANALYSIS

ACCUMULATED LOAD VECTOR DIRECTION 1 (SSE)

WT(K)	NP (1)	ELEV.	HEIGHT	FORCE (K) (2)	MOMENT
2625	300	363'	131	3.09×10^3	404,790
3705	301	343'	111	4.36	483,960
2070	302	331'	99	2.43	240,570
1710	303	320'	88	2.01	176,880
3600	304	312'	80	4.24	339,200
5040	305	280'	48	5.94	285,120
4320	306	256'	24	5.09	122 160
23,070		232'	0	–	
				$\Sigma = 27,160^K$	$\Sigma = 2,052,680$

ANALYSIS: PIPING UPGRADE 1979 → 0.21 G INPUT (PEAK ACCEL)

(3) MODE: 2

(2) FREQUENCY: 7.53 Hz 0.05% DAMPING 0.118 G FOUND. ACCEL.

DIRECTION: HORIZ 1

DAMPING: (3) CONC - 0.007
STEEL - 0.004
PRE-CONC - 0.005

BASE SHEAR $= 27,160 \times 0.20g = 5432^K$
$= 15.9\ ^K/FT$

BASE OVERTURNING: $2,052,700 \times 0.20 =$ 410,540 FT-K.

EFFECTIVE G = 0.20G. (2)

1. PIPING SEISMIC UPGRADE PROGRAM (1980)
   (1) BOOK 1 PAGE 2
   (2) BOOK 4 DYNRE 1 SSE
   (3) BOOK 3 LANCZOS & GUYAN REDUCTIONS.