NRC FORM 195 (2-76)

U.S. NUCLEAR REGULATORY COMMISSION

DOCKET NUMBER 50-220

NRC DISTRIBUTION FOR PART 50 DOCKET MATERIAL

FILE NUMBER

TO: Mr. George Lear

FROM: Niagra Monhawk Power Company, Syracuse, New York, Gerald K. Rhode

DATE OF DOCUMENT 7/27/77

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DESCRIPTION

RE our 6-20-77 HR

ENCLOSURE

Consists of requested additional information regarding the spent fuel pool modification for Unit 1.

ACKNOWLEDGED

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PLANT NAME:

Nine Mile Point Unit 1 (1-P)

VT 8/2/77

Enclosures: 17p.

1 cy Encl Rec'd

FOR ACTION/INFORMATION

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NIAGARA MOHAWK POWER CORPORATION

NIAGARA MOHAWK

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Regulatory File Cy.

July 27, 1977

RECEIVED AUG. 1, 1977 U.S. NUCLEAR REGULATORY COMMISSION Mail Section

Director of Nuclear Reactor Regulation
Attn: Mr. George Lear, Chief
Operating Reactors Branch #3
U.S. Nuclear Regulatory Commission
Washington, D.C. 20555

Re: Nine Mile Point Unit 1
Docket No. 50-220
DPR-63

Dear Mr. Lear:

Your letter of June 20, 1977, requested additional information regarding the spent fuel pool modification for Nine Mile Point Unit 1. The attached information is in response to your request.

Very truly yours,

NIAGARA MOHAWK POWER CORPORATION

Gerald K. Rhode
Vice President - Engineering

SWW/bd

Attachment

772150228

RECEIVED DOCUMENT
PROCESSING UNIT

1977 AUG 1 AM 10 22

Responses to June 30, 1977

Nuclear Regulatory Commission Questions

Nine Mile Point Unit 1
Docket No. 50-220
DPR-63

1. Provide a summary of the stresses and safety margins in the diagonal braces across the bottom of the refueling canal, the shear blocks and the snubbers, and the bearing stresses and safety margins where the snubbers contact the fuel pool walls. In addition, figures 1.3 through 1.5 of the February 11, 1977, response to NRC question 1 indicate that the shear blocks, diagonal braces, and snubbers are not physically connected to the rack base legs. This may be unacceptable since friction should not be relied upon to fasten these components under a seismic event. Also, consider the possibility of lateral instability of the diagonal braces affecting their load carrying capacity.

RESPONSE

The shear blocks, diagonal braces and snubbers are all fitted to the rack bases by means of pins and cutouts in the bases. A similar designed system was used at the Rochester Gas and Electric Ginna Plant, that has been approved by the NRC. There is no reliance on friction to fit any parts. The design of the diagonal braces was a result of a stability analysis.

The current design of the seismic bases eliminates shear blocks and relies on rack base to rack base "bumpers" that are part of these bases. The arrangement for the Group 1 (North) rack sets and cask area seismic supports are shown in Figure 1. The arrangement for the Group 2 (South) rack sets is shown in Figure 2. Seismic restraint in the southern direction for Group 1 racks is by means of bumpers that are attached to the bases and bear against the mounting brackets imbedded in the pool floor. Similar supports restrain the Group 2 racks in the northern direction. Figure 3 shows the detail of this support. The wall support is depicted in Figure 4. A summary of stresses are:

The minimum margin of safety among rack base structures occurs in the East-West members of the base in the Southeast corner of Group 1 racks. The margin of safety is 0.40 of the Operational Basis Earthquake stress-limit. The corresponding compressive stress is 14 KSI.

The minimum margin of safety among members forming the cask area restraint is in the East-West column running along the north wall. The margin of safety is 0.09 of the Safe Shutdown Earthquake buckling limit.

The minimum margin of safety for concrete bearing stress occurs at the pad on the West wall immediately south of the cask area. The margin of safety is 0.79, corresponding to a compressive stress of 0.56 KSI.

The minimum margin of safety among swing bolt floor clevises occurs along the south edge of the Group 1 racks. The margin of safety is 0.33 of the Operational Basis weld shear stress limit. The margin of safety is 0.60 of the Safe Shutdown Earthquake weld shear stress limit.

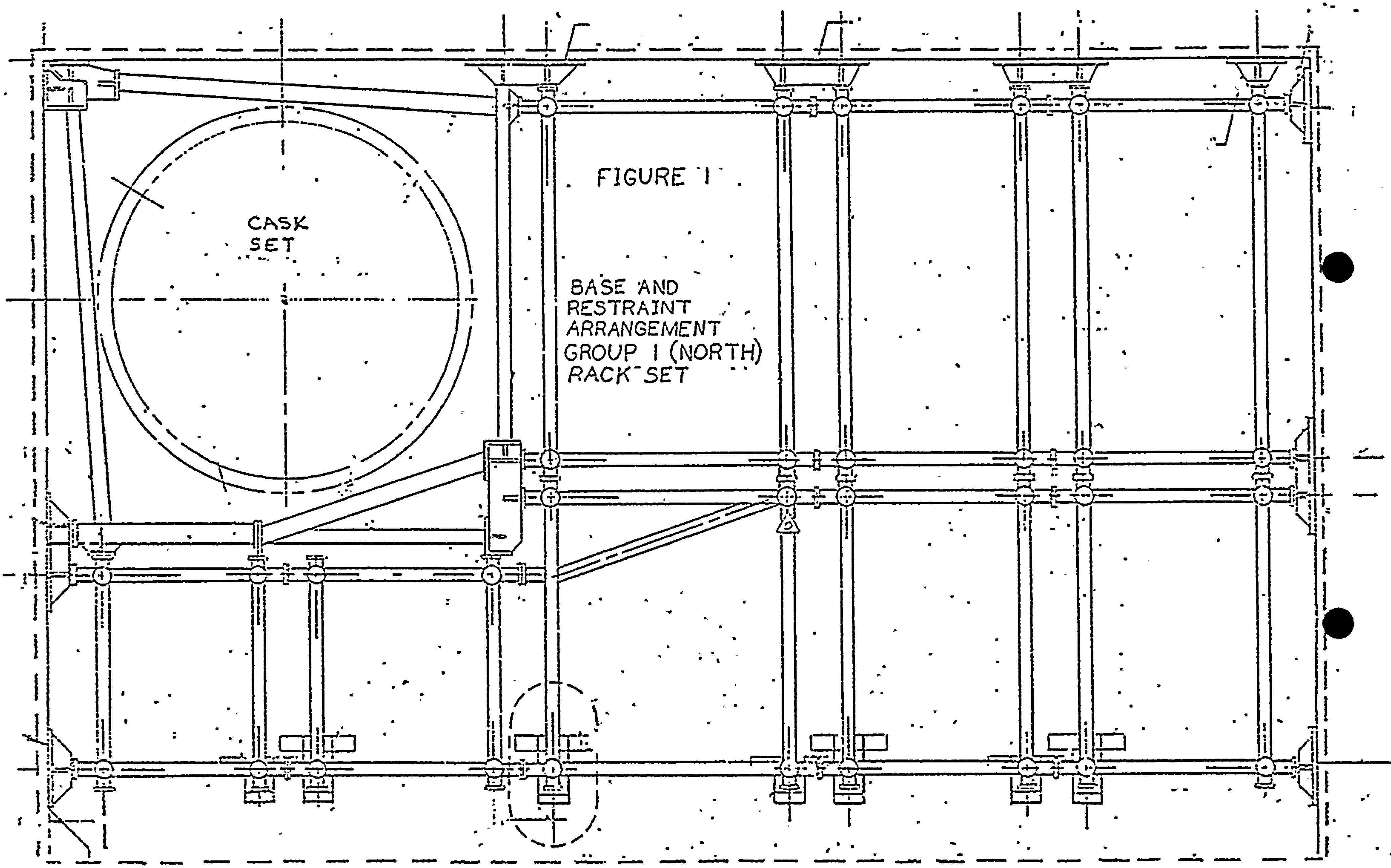
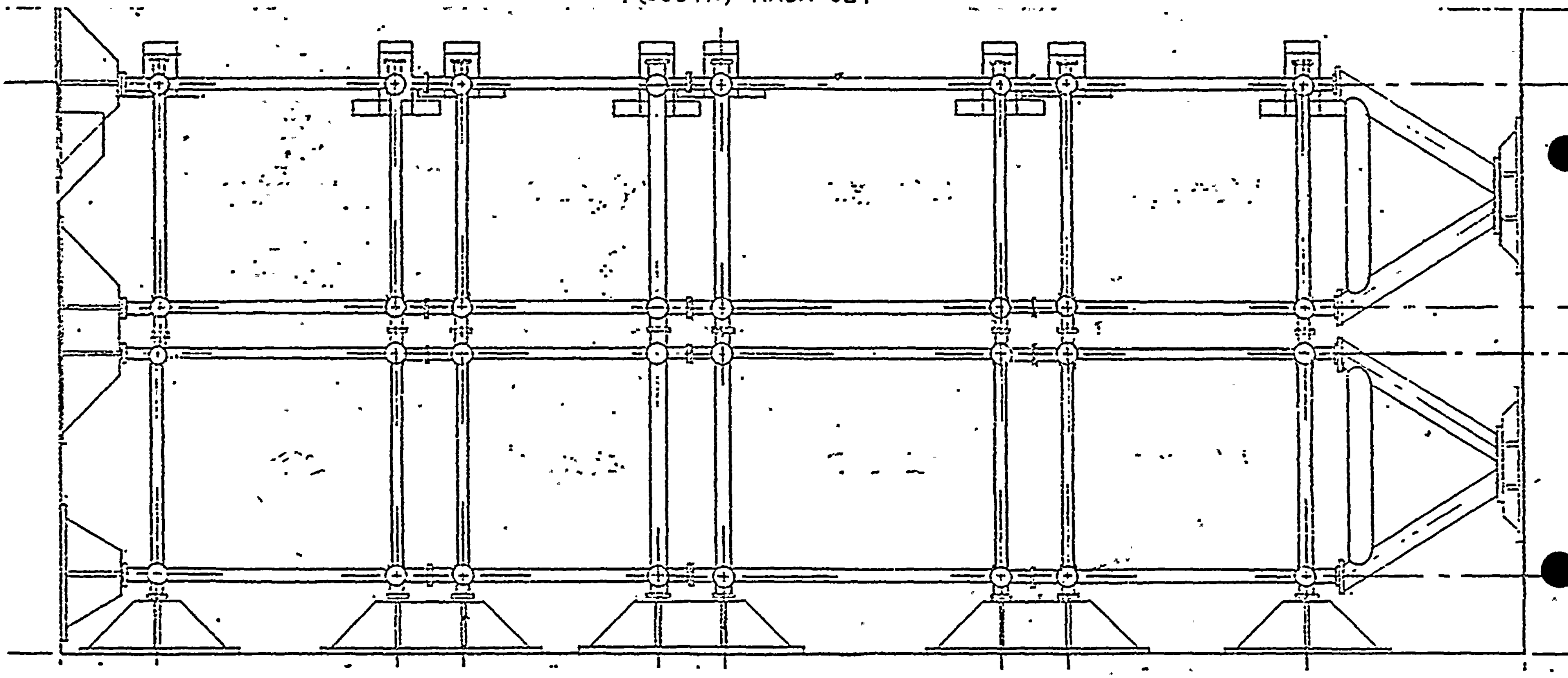
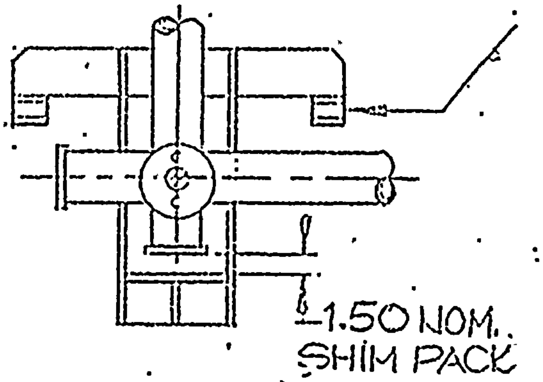


FIGURE 1
CASK SET
BASE AND RESTRAINT ARRANGEMENT GROUP 1 (NORTH) RACK SET

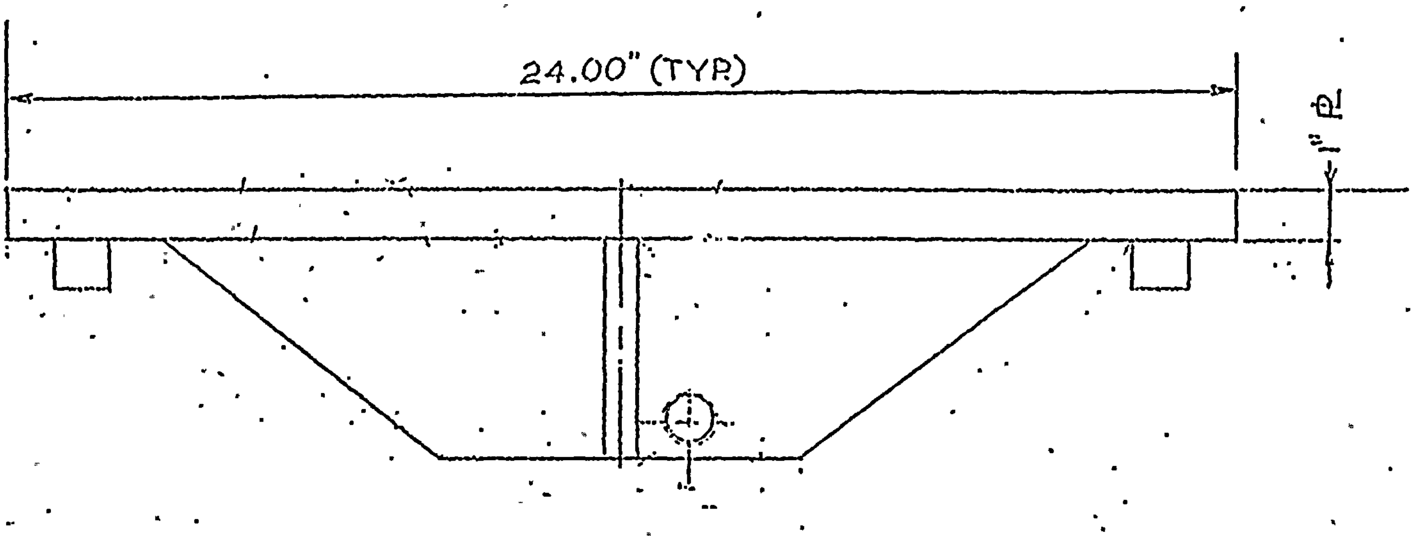
FIGURE 2
BASE AND RESTRAINT
ARRANGEMENT GROUP 2
(SOUTH) RACK SET



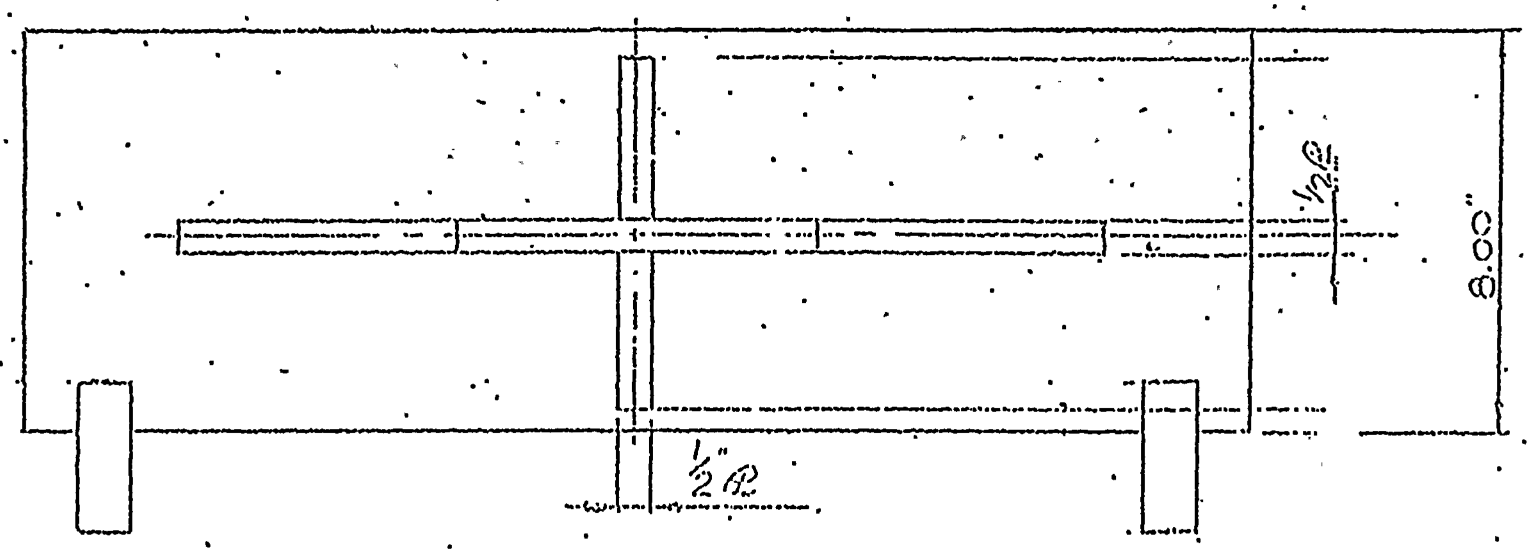


PLAN VIEW
FIGURE 3. SEISMIC RESTRAINT

FIGURE 4.
TYPICAL WALL SNUBBER



PLAN



ELEVATION

2. Your response to the February 11, 1977, NRC Question 2 is not complete. Provide the following information:

   a. Indicate the damping values utilized for the OBE and SSE analyses separately since the utilization of a structural damping value of 2.5% was specified.

   b. Your utilization of structural damping values corresponding to bolted structures is not acceptable. The connecting mechanism between two structural components is more rigid for a weld than for a bolt due to the clearances between the bolts and the bolt holes. Therefore, use damping values corresponding to those for a welded structure.

   c. Provide a summary of the natural frequencies, mode shapes and corresponding participation factors for all modes with frequencies less than 33 Hz to substantiate your statement that only the first mode of vibration contributes significantly to the response of the rack modules.

   d. Compare numerically the zero period accelerations of mass point 4 of the reactor building seismic model utilized for the seismic analysis of the fuel rack modules with the zero period accelerations of the corresponding mass point in the original seismic analysis of the reactor building.

   e. Provide the analytical model utilized for the calculation of stresses within the fuel rack modules.

   RESPONSE

   a. The damping value used for the two analyses is 2.5 percent.

   b. The honeycomb structure is made up of layers of boxes edge welded together by intermittant welds. There is over 6,000 square feet of internal bearing surfaces between boxes in the average size spent fuel rack. During deflection of the boxes the movement of boxes in the area between welds results in friction restraint. The 2 1/2 percent damping is low by at least a factor of three.

   c. The fundamental mode of all the racks is below 33 hertz. The second is above 50 hertz, as shown in Table 1. This is far removed from the 33 hertz cutoff frequency. Thus only the fundamental mode need be considered in the vibration analysis.

   d. The zero period accelerations of the ground response spectra are the same for the original seismic analysis of the reactor building as in the present analysis, i.e., 0.11 g. Since response spectra were not developed for the reactor building as a part of the present analysis, the zero period acceleration of mass point 4 of the reactor building cannot be compared to that of the original analysis. However, the

acceleration of mass point 4 of the reactor building as determined by the square-root-of-the-sums-of-the-squares method is 0.223 g in the North-South direction and 0.226 g in the East-West direction.

TABLE 1

SUMMARY OF SPENT FUEL RACKS FUNDAMENTAL NATURAL FREQUENCIES, MODESHAPES, AND PARTICIPATION FACTORS

	96		108		120		128		160		200	
	N-S	E-W	N-S	E-W	N-S	E-W	N-S	E-W	N-S	E-W	N-S	E-W
f_1 Hz	19.0	18.6	20.1	19.9	18.4	21.1	21.6	17.6	23.3	16.9	22.3	19.1
p.f.$_1$	14.21	14.20	15.29	15.25	16.09	16.24	16.88	16.66	19.16	18.71	21.73	21.41
y_1	.005	.004	.006	.005	.005	.006	.007	.005	.009	.005	.010	.007
y_2	.011	.010	.011	.011	.010	.012	.012	.010	.014	.010	.014	.011
y_3	.018	.018	.019	.018	.017	.019	.019	.017	.020	.016	.019	.016
y_4	.027	.027	.028	.027	.025	.027	.026	.024	.027	.022	.025	.022
y_5	.037	.037	.038	.040	.034	.035	.035	.033	.033	.030	.030	.029
y_6	.048	.048	.046	.046	.043	.044	.043	.042	.040	.038	.036	.035
y_7	.059	.059	.059	.056	.053	.053	.051	.051	.047	.046	.042	.042
y_8	.070	.070	.066	.066	.063	.062	.060	.060	.053	.054	.047	.048
y_9	.081	.081	.075	.076	.072	.071	.068	.070	.059	.062	.053	.054
y_{10}	.092	.092	.084	.085	.082	.079	.076	.079	.065	.070	.058	.061
y_{11}	.103	.103	.093	.094	.091	.087	.083	.088	.070	.078	.062	.066
f_2, Hz	65.7	65.5	65.4	65.2	63.7	64.6	63.7	64.3	61.0	60.6	59.2	59.4
p.f.$_2$	7.94	7.96	8.10	8.31	9.00	8.55	8.67	9.35	8.91	10.49	10.04	11.17

f_1 - First F_n, p.f.$_1$ - participation factor of f_1, y_i - relative displacements of f_1.

f_2 - Second f_r, p.f.$_2$ - participation factor of f_2.

e. The analytical model used for calculation of stresses within the rack modules is shown in Figure 5. This is a three-dimensional statically indeterminant structure which was solved elastically for each of the three directions for all loads. The results were combined using the square root sum of the squares method.

3. Your responses to the February 11, 1977, questions concerning the impacting of the fuel racks with the bottom of the fuel pool and the impacting of the fuel assemblies within the cans are inadequate. Therefore:

   a. Quantify the kinetic energy of a dropped rack module, and the energy absorption capacities of the rack bases and the fuel pool floor for the case of a rack module impacting on either of these structures with its corner or edge. State the effects on the structural integrity of the rack base, and fuel pool liner and floor.

   b. Discuss and quantify the local and gross effects on the rack modules, and fuel pool liner and floor for the following three cases of a dropped fuel assembly:

      1. a straight drop on the top of a rack module

      2. an inclined drop on the top of a rack module

      3. a straight drop through a can with the fuel assembly impacting the bottom of the can.

   Include the kinetic energies and the height of drop considered for each of the three cases. In addition, consider the effects of the loading which will result from a fuel assembly sticking inside a can. (This loading is defined in ANSI Standard N210-197). The upward loading should be the maximum force the crane is allowed to exert on a fuel assembly.

<u>RESPONSE</u>

a. The kinetic energies and the energy absorption characteristics of the spent fuel racks, bases and fuel pool floor have been examined.

   Each new spent fuel rack is assumed to be dropped from three feet above the pool water surface. Due to its shape and construction, the rack's attitude will tend to stabilize due to the effects of the hydraulic forces as it sinks toward the pool floor. As the rack sinks, the water it displaces will be forced through the fuel plate holes causing a pressure difference between the top and bottom of the rack structure. A terminal velocity will be reached which limits the drop energy to less than that calculated for a free-fall through the equivalent height in air. The energy at impact ranges from 3,800 ft.-lbs. for the smallest rack to approximately 10,900 ft.-lbs. for the large 200 fuel assembly rack.

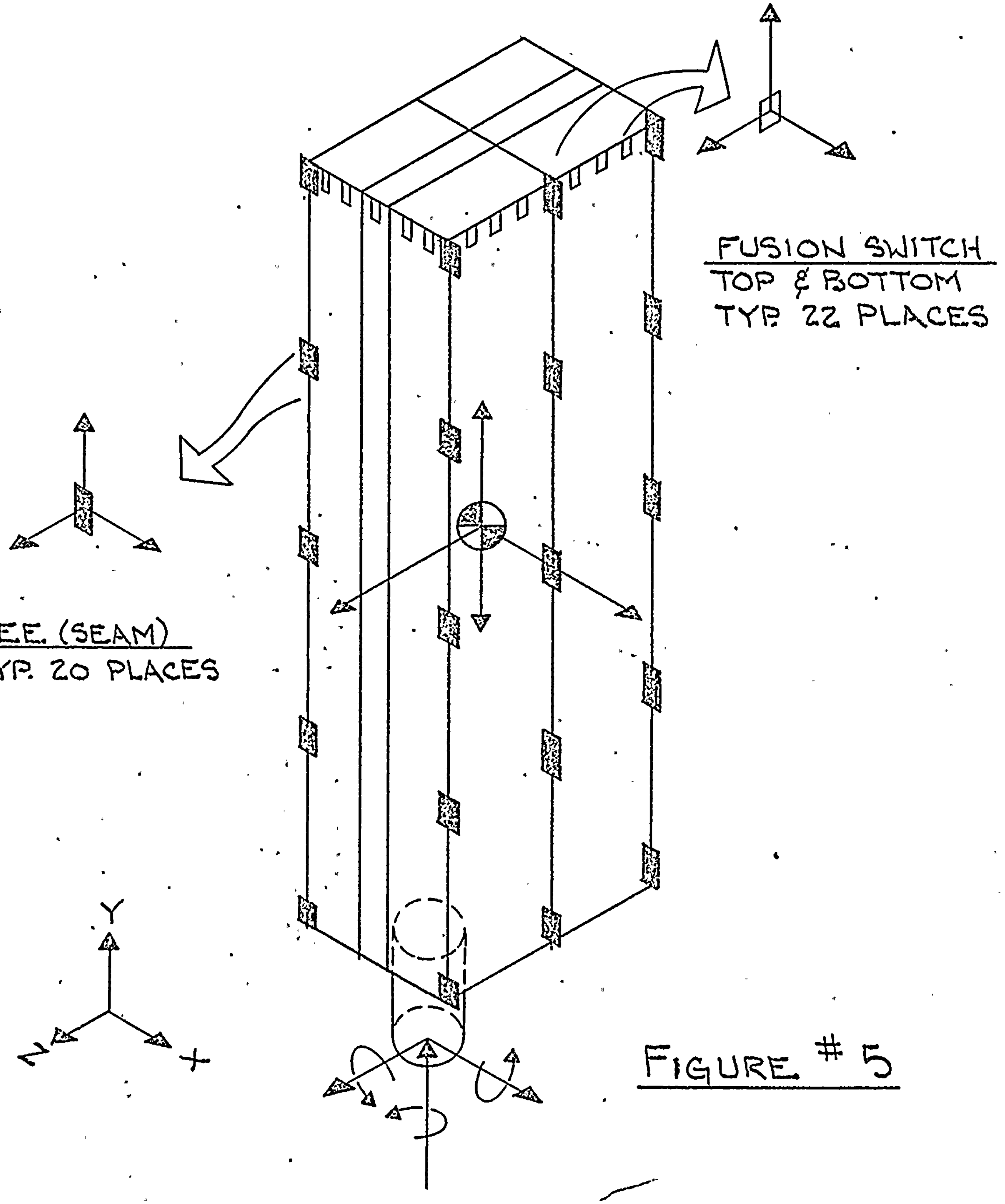

FUSION SWITCH
TOP & BOTTOM
TYP. 22 PLACES
VEE. (SEAM)
TYP. 20 PLACES
Y
Z
X

FIGURE # 5

If any empty rack module should be dropped onto the pool floor, the dynamic loading caused by a flat-bottom-to-pool-floor loading will be on the order of 750 PSF over the rack projected area. The spent fuel pool floor loading is designed for 2,000 PSF which suggests that no damage will result from such a drop to the pool floor.

The energy absorption of the rack base structure has been calculated as 19,830 ft.-lbs. The energy absorption of the rack module itself ranges from 1,100,000 ft.-lbs. for the smallest rack to 3,150,000 ft.-lbs. for the large 200 rack.

In considering a rack drop onto a base structure, the absorption characteristics and deformation of both structures must be taken into account. If a rack module is dropped onto a previously installed base structure, the corner pedestals will transmit the dynamic loads through heavy bearing plates which serve to reinforce the local pool liner and concrete during the design seismic loading. The dynamic forces are expected to be about 21 percent of the design loads for a fully-loaded 200 fuel assembly rack undergoing a worst-case vertical seismic event. The rack drop on top of a base does not pose a threat to pool floor integrity; however, the rack will quite likely be deformed so as to preclude further use for fuel storage.

If the dropped rack contacts either the pool floor or a base structure with an edge or a corner, the duration of impact will be longer, with a corresponding lower force loading. However, on-site observation of the lowering of nine similarly sized new spent fuel racks into the Rochester Gas and Electric Ginna spent fuel pool strongly suggests that an edge or corner impact configuration during rack descent is extremely remote. Nevertheless, with either an edge or corner impact, the rack and base structure is damaged, (i.e., pipes bent, fuel and water boxes buckled) but the pool liner and concrete floor sustain little, if any, damage. This type of rack drop will not threaten the safety of the spent fuel pool structure.

b. The dynamic loading from a dropped fuel assembly and the resulting effects on the rack modules and fuel pool structure have been calculated for the following cases:

1. Straight drop on top of a four box intersection.

2. Straight drop on top of a box wall for a two box intersection.

3. Inclined drop on top of a box wall for a two box intersection.

4. Straight drop through a box with the fuel assembly impacting the bottom fuel box plate.

The drop height, which is governed by the fuel crane limit mechanisms, is such that the distance between the bottom fuel assembly nozzle and the peak of the lead-in guide on the fuel rack module is 38 3/4 inches. The energy of the fuel assembly was calculated to be 2,585 ft.-lbs. (Case 1 & 2) and 7,385 ft.-lbs. (Case 3).

Case 1 drop results in local rack structure deformation. The lead in guides and some of the box walls are buckled from the impact; however, the dynamic loading is not sufficient to cause general rack structural failure. The extent of the local damage has been estimated to be no more than 11 inches deep into the rack structure. For this case, the box walls will buckle but the attachment welds will not fail.

Case 2 is quite similar to Case 1 except there is a smaller area of contact with the fuel assembly and box wall. Calculations, which ignored the beneficial energy absorption characteristics of the lead-in guides, showed that the rack structure possessed adequate strength margin to resist the fuel drop loads without gross failure. The estimated depth of local deformation is on the order of 30 inches and the damage is a bit more extensive than Case 1. Plastic deformation of the box walls is expected from this type of accident. The calculations have assumed that the fusion welds on the top of the box walls directly underneath the fuel assembly impact were failed, but none of the other attachment welds were stressed beyond their ultimate load capability.

Case 3 results in a lesser penetration of the local rack structure but does have the effect of deforming many more lead-in guides and spreading the drop energy more uniformly throughout the top of the rack. Many of the same conclusions of Case 2 will apply to Case 3: local damage, local plastic material and weld behavior, but no gross loss of rack structural strength.

Case 4 is different from the previous three. For this accident, it is assumed that the fuel assembly is dropped so that it is perfectly aligned with the fuel box centerline touching neither box wall nor adjacent fuel assembly as it falls. Given this special case, the water resistance which the fuel assembly encounters was modeled as the free-fall of a leaky hydraulic piston in a cylinder. The calculations revealed that a terminal velocity was achieved which served to limit the kinetic energy of the fuel assembly. The velocity of impact has been calculated to be about 6 ft./sec. as compared to an equivalent free-fall velocity of 32 ft./sec. The impact energy is therefore about 450 ft.-lbs. The bottom fuel plate was modeled as a spring to generate a peak force of deceleration; the weld was then checked and found to be stressed nearly to shear yield. The shear stress was estimated to be 13,500 psi for a material which (assuming maximum shear stress theory of failure) had a shear yield stress of 12,500 psi and an ultimate shear stress of 37,500 psi. The conclusion to be drawn is that no damage to the pool liner will result since the fuel plate weld does not rupture.

In every case, the rack was considered to be fully-loaded with its maximum load of fuel assemblies. The dynamic forces which were calculated for all four cases were all less than the seismic forces to which the rack and base structure were designed.

The maximum uplift force which can be exerted by the fuel crane is 2,000 lbs. While it is exceedingly remote that a fuel assembly could be stuck in the smooth-surfaced interior of a fuel box, if the two kip load is transferred uniformly to the box walls, the seam welds will be stressed to 4.4 percent of their design strength. If this fuel assembly should somehow catch the bottom fuel plate, the outside bottom fillet weld will be stressed to 9.7 percent of design. It is concluded that the uplift accident poses no threat to either the rack structure integrity or to the spent fuel pool.

4. In your February 11, 1977 response to NRC question number 7, no temperature loading was considered for the case of a temperature gradient across a rack module (e.g., consider the case of an empty module with a full rack module on one side and an empty rack module on the opposite side).

RESPONSE

During the design of the spent fuel racks, thermal loadings from temperature gradients have been considered.

The only case of gross thermal expansion to be considered would be any pair of adjacent spent fuel racks in the pool. There is at least 2.91 inches of clearance between any rack and its adjacent, east-west neighbor. Wall-to-rack clearances range from 4.25 inches to 13.46 inches. The clearance between the control rod rack structures (located east-west in the central part of the pool) and the rack structure is on the order of 1.5 inches. The only portion of any fuel rack which physically touches any other metal structure is as follows: the south face of the northernmost row of racks which touches the north face of the row of racks immediately to the south; and the north face of the southernmost row of racks which touches the south face of the row of racks immediately to the north. Note that all spent fuel racks have no constraint in the vertical direction of east-west direction. Thus the only case to be considered is a north-south rack pair.

Each north-south pair is fastened to bases which have been shimmed to a solid fit. If, in the case of one rack completely full of fuel and the other empty, the average metal temperature is assumed to be 200 F and if, at installation, the rack temperature was 60 F, then the maximum unrestrained expansion would be no more than 0.158 inches over a maximum dimension of 120.2 inches. Previous responses indicate that the seismic bumpers are installed

and are shimmed to a nominal 0.25 inch clearance. If the pool floor is considered frictionless, the largest rack cannot expand enough to close the gap. If both racks are at 200 F, the total expansion would be no more than 0.316 inches which means that no forces due to thermal expansion will be transmitted to the spent fuel pool walls.

Local box wall thermally-induced bending stresses have been evaluated for normal and severe service conditions. Employing conservative assumptions concerning temperature distribution and box wall edge restraints, the maximum, peak bending stress of a box wall located near the hottest fuel assembly in the pool has been estimated to be less than 4,000 psi, compressive on the hot side and tensile on the cold. These stresses have only a local effect due to the method of box-to-box fastening which permits slight relative elastic wall displacement.

5. Provide the details of the procedures and the methods utilized in the testing of the welds to establish the ultimate weld stresses referenced in your February 11, 1977, response to NRC questions 8 and 9.

RESPONSE

Tensile tests of butt welded specimens for qualification of weld procedures, welding fabricators and welder operators were performed in accordance with Section IX of the ASME Boiler and Pressure Vessel Code, Part QW. The Procedure Qualification Specimens documented in Table QW-451 were adhered to and a test plate of 3/8 inches (0.375 inches) thickness was selected, since this served to qualify welds in the range of 1/16 inches (0.0625 inches) to 3/4 inches (0.750 inches). The tension test specimens were prepared from test plates shown graphically in Figure QW-463.1 (a), with tension specimens prepared in accordance with Figure QW-462.1 (a). Face and root bend specimens were prepared in accordance with Figure QW-462.3 (b). Tensile testing was performed by Industrial Test Laboratory Services Corporation. The ultimate tensile stresses encountered ranged from 87,400 psi to 93,600 psi. In accordance with the maximum shear theory of failure for ductile steel, the ultimate shear stresses range from 43,700 psi to 46,800 psi.

In accordance with Section QW-202.2, qualification for butt-welding shall qualify the procedure, the fabricator and the welder for all types of fillet welds in all thicknesses of metal.

In addition to the above tests, tensile-shear testing of fusion spot welds was performed using 0.090 inches test specimens randomly chosen from welding development pre-production tests utilizing production welding parameters documented on welding procedures.

These test specimens demonstrated ultimate shear strengths ranging from 3,120 to 4,150 pounds force to rupture. The fusion spot weld was conservatively modeled as a fillet weld around the inside perimeter of a spot - equivalent - sized hole. [The model is similar to a plug weld except the hole diameter is taken as the mean spot weld diameter at the interface of the joined sheets, determined by sectioning weld specimens and measuring the weld penetration. For further conservatism, the diameter was reduced by twice the theoretical fillet weld size (2 x 0.090 = 0.180)]. The amount of shear stress calculated in this manner is on the order of 15.4 psi per pound of shear load. Thus the ultimate shear stress of the weld model ranged from 47,900 psi to 69,800 psi according to these tests.

The test value of the Tungsten Inert Gas arc fusion weld shear ultimate strength is undoubtedly lower than that for a pure shear test since no shear supports were employed to offset the imposed tensile forces of the testing machine. Thus, as the weld failed in shear by plastic deformation, the pulling forces tended to superimpose a tensile load, thereby producing a combination shear and tensile loading of the welded diametral sections. The expected service loading of the spot welds approaches pure shear since the box walls are much more restrained than the weld sample was during the test.

6. There is a conflict between Figures 3 and 5 of your original submittal and Figure 9.1 of your February 11, 1977, response to NRC question 9. Figure 9.1 indicates that a plate was considered separating two adjacent fuel assemblies within a can, whereas, no plate is illustrated in Figure 3 and 5. If no plate is provided between two assemblies within a can, perform the impact analysis without the plate indicated in Figure 9.1 for an earthquake in the direction of the axis between the centers of the assemblies within a can. Also, perform the impact in the opposite direction. Discuss the local as well as gross effects on the rack modules and fuel assemblies themselves.

RESPONSE

A divider plate exists at the top of the fuel rack that serves to support the top of the fuel assembly as installed in the racks. Analyses have previously been performed to show that the fuel assembly with channel does not deflect enough to touch an adjacent fuel assembly under the worst load condition.

7. Provide a summary of the uplift forces and overturning moments, along with safety margins against overturning of the critical rack module for the load combinations considered.

RESPONSE

The following is a table of safety factors considering the overturning moments imposed on the racks during an Safe Shutdown Earthquake. This table includes both North-South (N-S) and East-West (E-W) directions.

Rack Capacity	Factor of Safety	
	E-W	N-S
96	1.193	1.264
108	1.389	1.262
120	1.587	1.262
128	1.191	1.766
160	1.189	2.268
200	1.584	2.266

8. Provide the water chemistry which will be maintained in the spent fuel pool. Include the boron concentration, pH, chloride, floride and any heavy metal concentrations.

RESPONSE

A summary of the water chemistry expected to be maintained in the spent fuel pool at Nine Mile Point Unit 1 is shown below:

Component	Concentration	Remarks
boron	0.0 ppm	Boron is not added to the Spent Fuel Pool water and is not analyzed for.
pH	neutral	The spent fuel pool water is air saturated.
chloride	<0.1 ppm	
fluoride		Not analyzed for.
heavy metal		Not analyzed for.
conductivity	~ 1 μ mho/cm	

[illegible]