UNITED STATES OF AMERICA

NUCLEAR REGULATORY COMMISSION

In the Matter of	)	
	)	Docket No. 50-70
GENERAL ELECTRIC COMPANY	)	
	)	Operating License
(Vallecitos Nuclear Center -	)	No. TR-1
General Electric Test Reactor	)	(Show Cause)

TESTIMONY OF GARRISON KOST, HAROLD DURLOFSKY AND DWIGHT L. GILLILAND CONCERNING ISSUE 2. SUBMITTED ON BEHALF OF THE GENERAL ELECTRIC COMPANY

May 1, 1981

DS03 S 0/1

Introduction/Summary

The United States Nuclear Regulatory Commission (NRC) issued an Order to Show Cause to General Electric Company on October 24, 1977, which suspended operations at the General Electric Test Reactor (GETR) and required resolution of the issues enumerated in the Show Cause Order. These issues are:

(1) What the proper seismic and geologic design bases for the GETR facility should be;

(2) Whether the design of GETR structures, systems and components important to safety requires modification considering the seismic design bases determined in issue (1) above; and, if so, whether any modification(s) can be made so that GETR structures, systems, and components important to safety can remain functional in light of the design bases determined in issue (1) above.

The testimony previously presented by Mr. Gilliland, Dr. Jahns, Mr. Harding, Dr. Reed and Mr. Meehan (1.0 meter offset), and Mr. Gilliland, Dr. Richter (if available) and Dr. Kovach (seismic design bases) demonstrates that the geologic and seismic design bases recommended by the NRC Staff are conservative. This testimony will start with that premise and will demonstrate that the GETR structures, systems, and components important to safety will remain functional in light of the conservative design bases.

In order to develop a firm basis for understanding the GETR and its response to the design basis seismic event, this testimony will first provide a brief functional description of the GETR and its pertinent design and operating characteristics (Part I., Facility Description). Next, the testimony will address the functional requirements which are necessary to assure a safe response of the GETR to design basis seismic conditions and the resulting structural and mechanical requirements, including modifications, for the GETR structures, systems and components which are necessary to achieve and maintain safe shutdown of the GETR under design basis seismic conditions (Part II.). Finally, the testimony will address the assumptions, methods, and results of the structural and mechanical analyses which demonstrate that each major structure, system or component which is necessary to achieve and maintain safe shutdown of the GETR will remain functional under design basis seismic conditions (Part III.).

I. FACILITY DESCRIPTION

The GETR facility consists of a high-flux, tank-type, low-pressure water reactor, reactor support auxiliary systems, and experimental facilities which has operated successfully since 1958. GETR operates at a maximum power level of 50 MW thermal. The reactor is designed to produce radioisotopes for medical and industrial use, and to test reactor fuels. A typical full power run of the reactor will last about 17 days; however, for purposes of analysis a 25 day full power run was conservatively assumed. The reactor does not produce electricity, and dissipates the heat produced through cooling towers. It operates at a stable steady state power level without any load demand changes.[1/]

The reactor, primary cooling system, irradiated fuel storage facility, experimental facilities and miscellaneous reactor auxiliary systems are housed in a reinforced concrete structure located in a steel containment building. The containment building and its surrounding support auxiliaries (Figure 1) are enclosed within a steel chain link fence.

The reactor core is contained in a 2-ft diameter cylindrical pressure vessel positioned on the bottom of a 9-ft diameter pool. The pool is flooded with demineralized water to a level 11 feet above the top of the reactor pressure vessel or 23 feet above the core. The pressure vessel and pool are enclosed in a massive concrete shielding structure.[2/] Water under low pressure in the primary system is used for cooling and moderating

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1/ In contrast, a typical large nuclear power plant is designed to produce electricity, and must be controlled to meet and accommodate changes in load demand. This requires more complex control systems which can accommodate a range of transient conditions which can be anticipated to occur for such reactors. The mode of operation for GETR would not involve or require sophisticated controls, and would not be expected to produce transient conditions.

2/ The pressure vessel and pool thus form a double barrier to loss of coolant. A typical large PWR nuclear power plant would not have this feature, and would rely upon the single barrier provided by the pressure vessel.

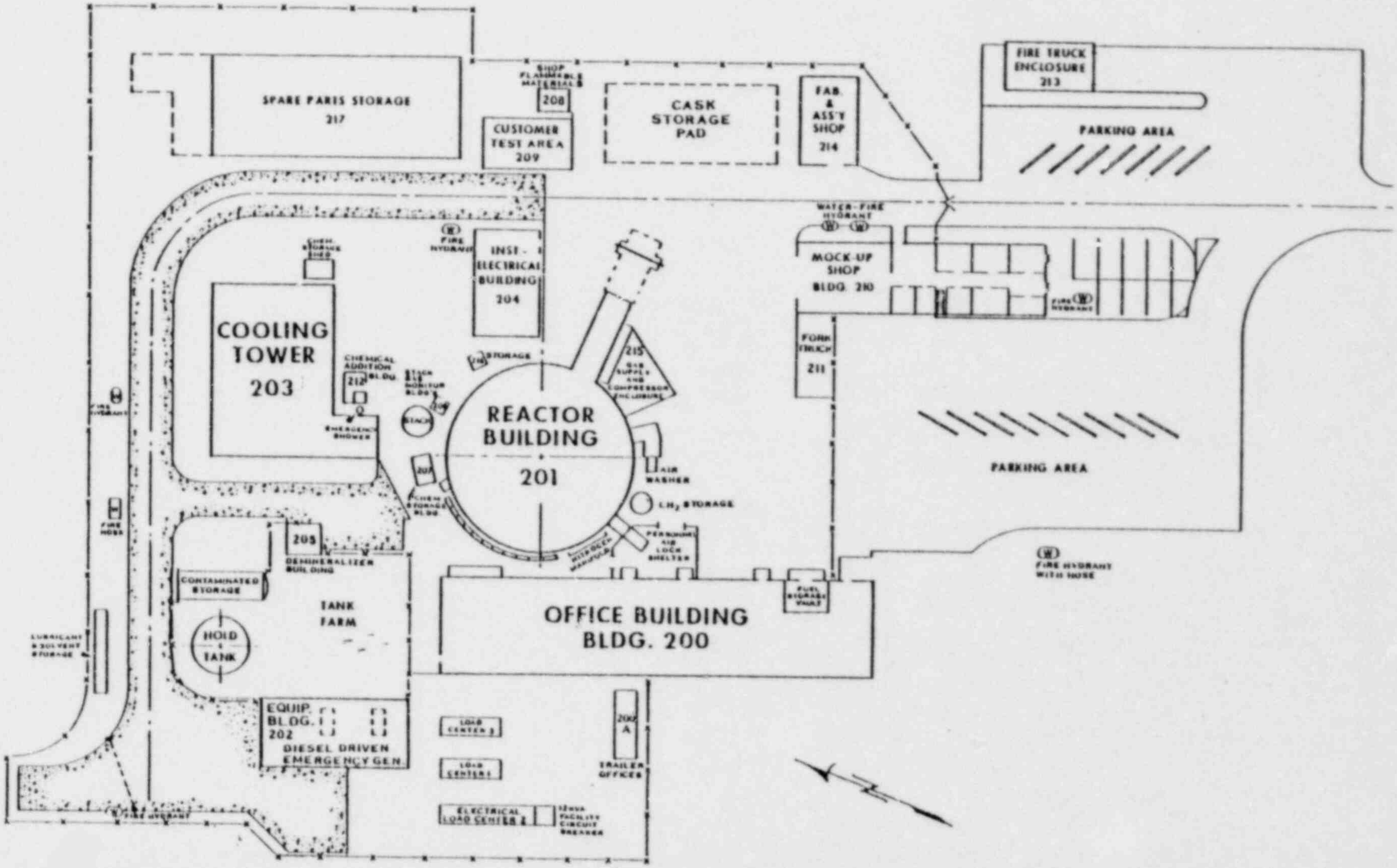

FIGURE 1. GETR AREA PLOT PLAN

the core. Unpressurized water is used in the pool for shutdown and emergency cooling and shielding. High purity demineralized water is circulated to the core and the pool by separate pumping and heat exchange systems. (See Figure 2). All of the primary cooling system piping and major components are located inside the

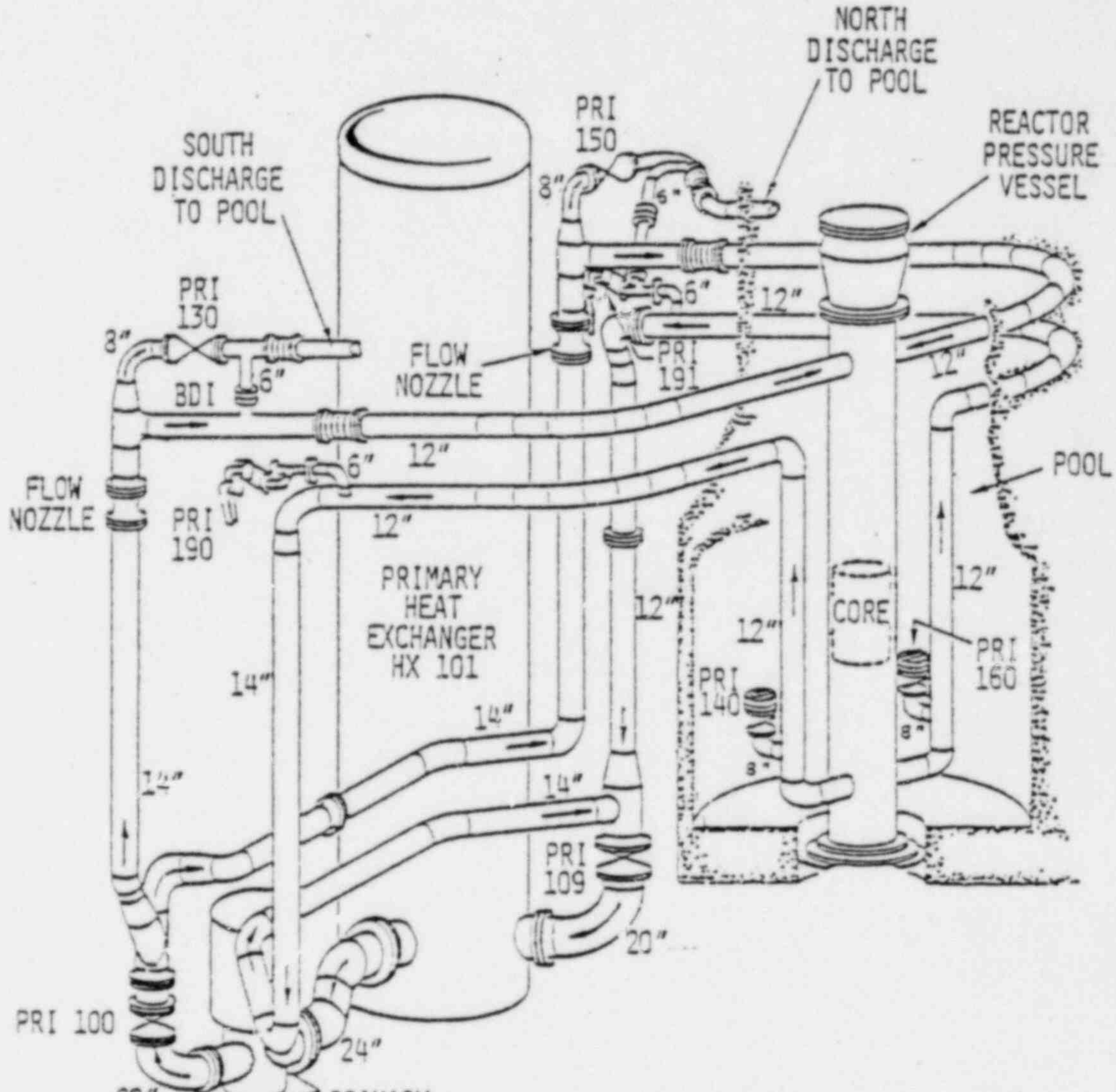

FIGURE 2. PRIMARY COOLING SYSTEM

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massive concrete structure at levels above the first floor. The reactor operates at a maximum thermal power of 50 MW,[3/] with a maximum temperature of 180°F and maximum pressure of 150 psig. The primary coolant is subcooled at atmospheric pressure, i.e., the coolant at 180°F would be below saturation - the boiling point of water (212°F) - at atmospheric pressure.[4/]

Primary coolant enters the reactor from two diametrically opposed 12-inch inlet pipes located near the top of the pressure vessel. The coolant flow is downward through the core and fuel elements, where the average water temperature is increased about 34°F for 50-MW operation. (See Figure 3). In a similar fashion, the coolant is discharged below the core through two diametrically opposed outlet pipes near the bottom of the vessel. The coolant then flows through a primary heat exchanger, and is pumped back to the reactor inlet. In the primary heat exchanger, the heat load is transferred from primary to secondary water; ultimately, this heat is transferred to the atmosphere through an induced-draft cooling tower.

3/ In contrast, a typical large PWR nuclear power plant would operate at a thermal power level of about 3,500 MW, and have a radioisotope inventory which is essentially proportional to power level.

4/ In contrast, a typical PWR operates at a temperature of 600°F and pressure of 2,100 psig. GETR thus has lesser potential for loss of coolant in that if the primary system were opened to the atmosphere, there is less expulsive force acting upon the coolant, and the primary coolant would not boil or flash to steam.

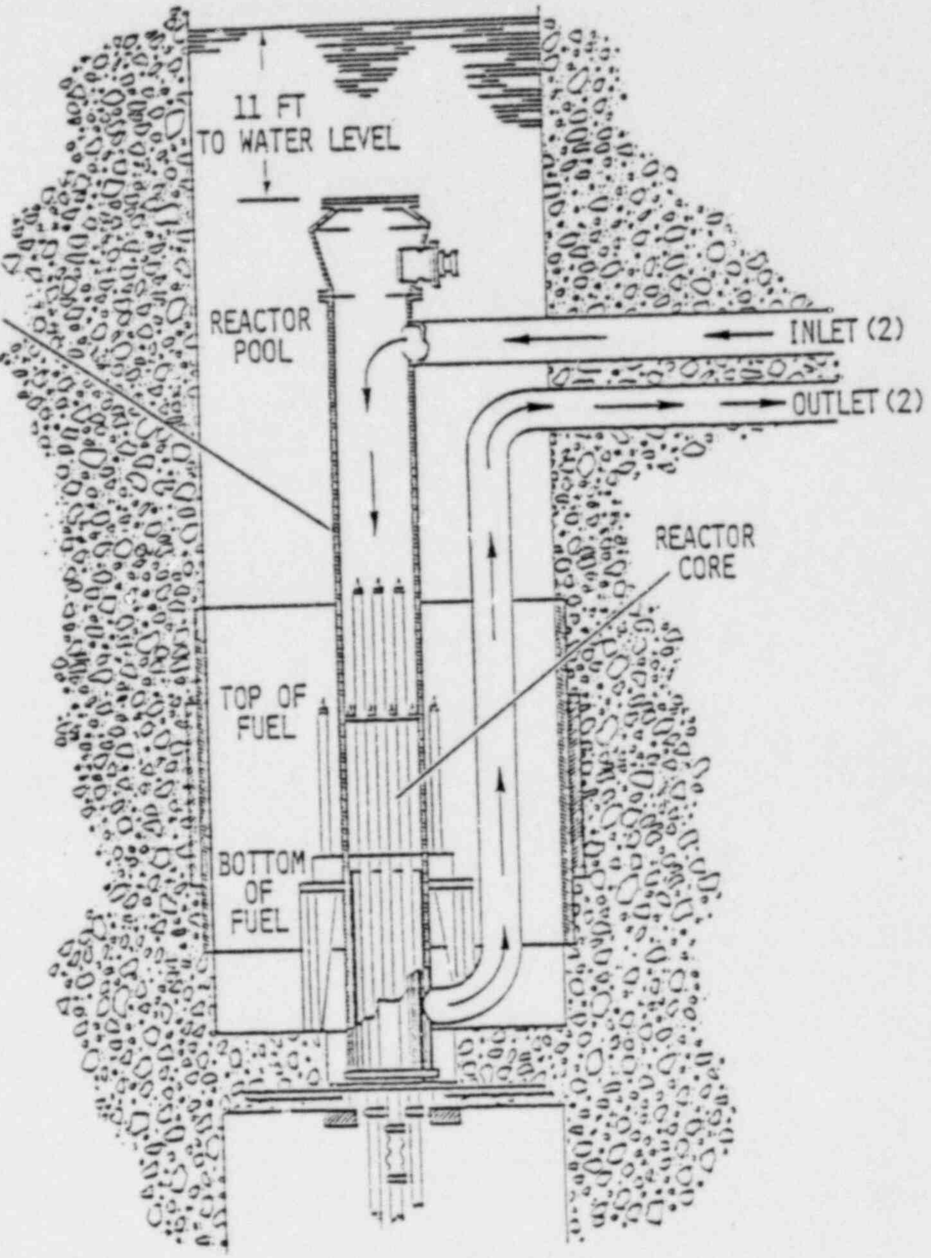

FIGURE 3. REACTOR COOLANT FLOW

The reactor core contains square cross-section fuel elements, filler pieces, and six bottom-mounted, top-entry control rods arranged in a close-packed square array. Experiment capsules may be positioned in the filler pieces to utilize the high core neutron flux. The number and position of fuel and filler pieces is adjusted as necessary to achieve the appropriate reactivity balance and flux distribution. Surrounding the square array, appropriately shaped beryllium and aluminum peripheral pieces round the core into a 2-ft diameter, 3-ft-high cylinder. (See Figure 4).

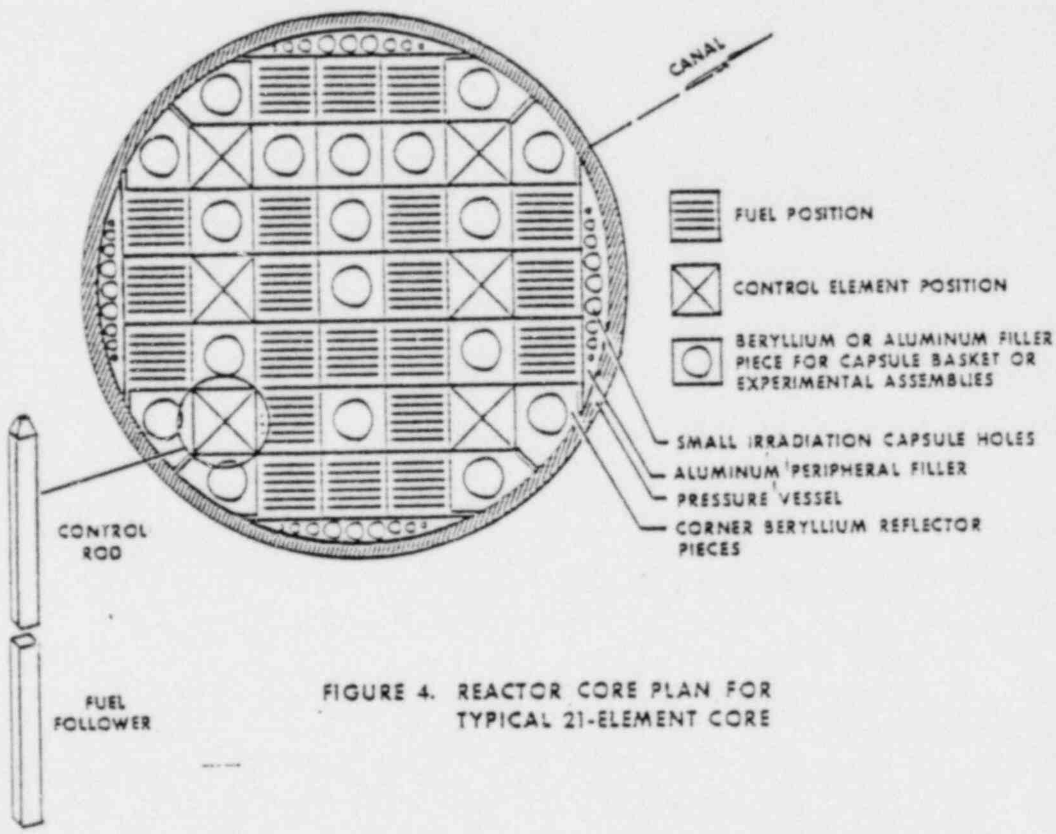

FIGURE 4. REACTOR CORE PLAN FOR TYPICAL 21-ELEMENT CORE

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The six individually actuated combination control rod and fuel follower assemblies are each separated from the other by at least one lattice unit. Shutdown or scram action permits the simultaneous drop of all control rods by gravity with primary coolant assist. (See Figure 5). The fuel follower section drops out of the core and the poison section enters the core. Any combination of five control rods provides a minimum shutdown margin of at least 1.0% $\Delta k/k$ under all reactor loading or operating conditions. For the normal core,[5/] which contains an equilibrium xenon concentration and partly burned fuel, either center rod or any combination of three or more rods is sufficient to ensure lasting subcriticality, while any single rod entering the core provides a significant shutdown margin long enough to permit unloading of the core.

An irradiated fuel storage facility (canal) is located adjacent to the pool and is also within the massive concrete shielding structure. The canal is filled with high purity demineralized water. (See Figure 6). Canal gates, which normally separate the pool and canal, are removed during shutdown to facilitate refueling. The irradiated fuel is stored in leak-tight fuel storage tanks located in the bottom of the canal. The canal water is circulated through a separate heat exchanger system to remove residual heat from the stored fuel.

5/ The GETR core contains 26 pounds of U^{235} in 21 fuel elements in a volume of approximately 9 $feet^3$. In contrast, a typical nuclear power reactor (BWR) contains 7,000 pounds of U^{235} in a core volume of about 2,500 ft^3.

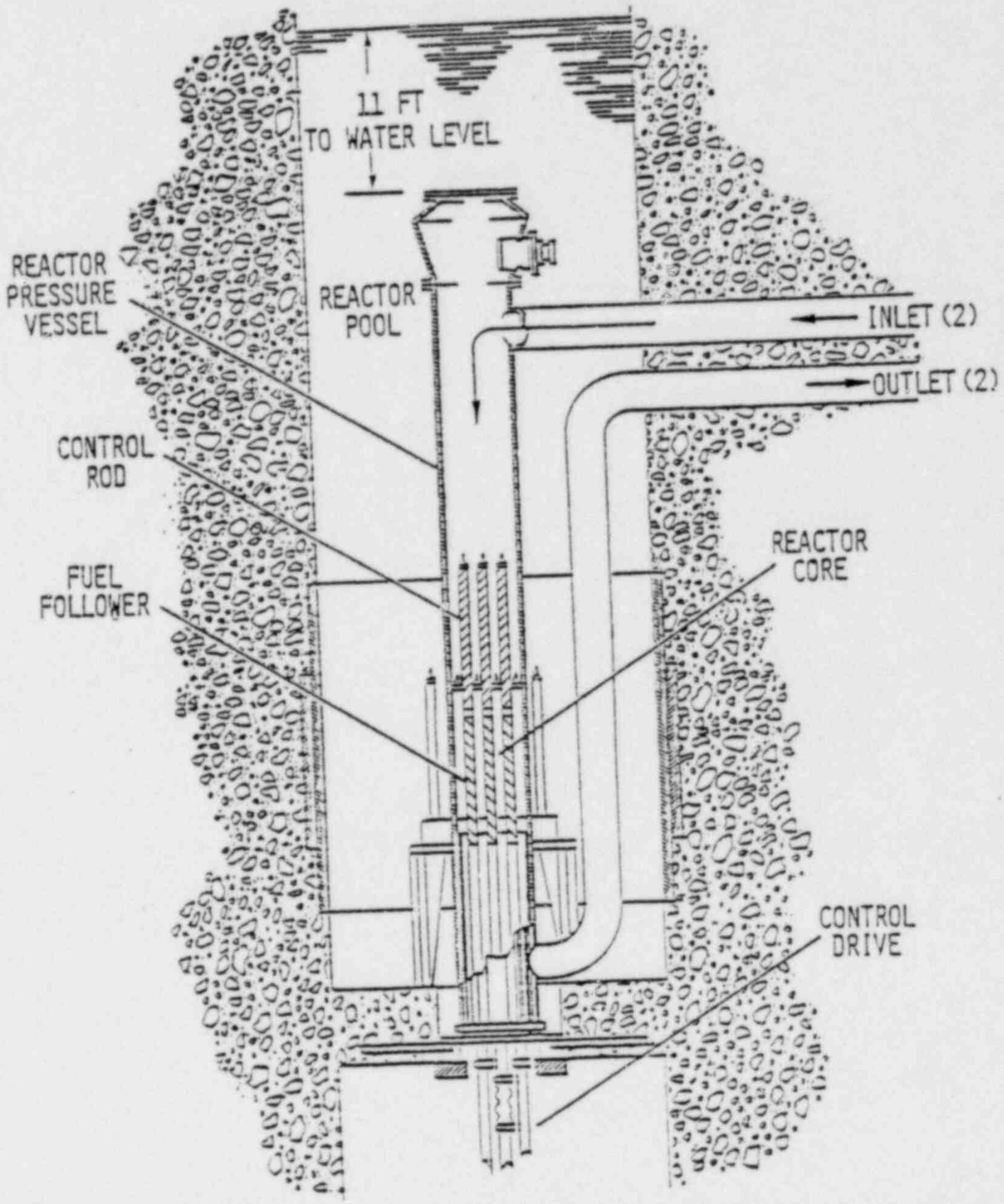

FIGURE 5. REACTOR AND POOL COMPONENTS

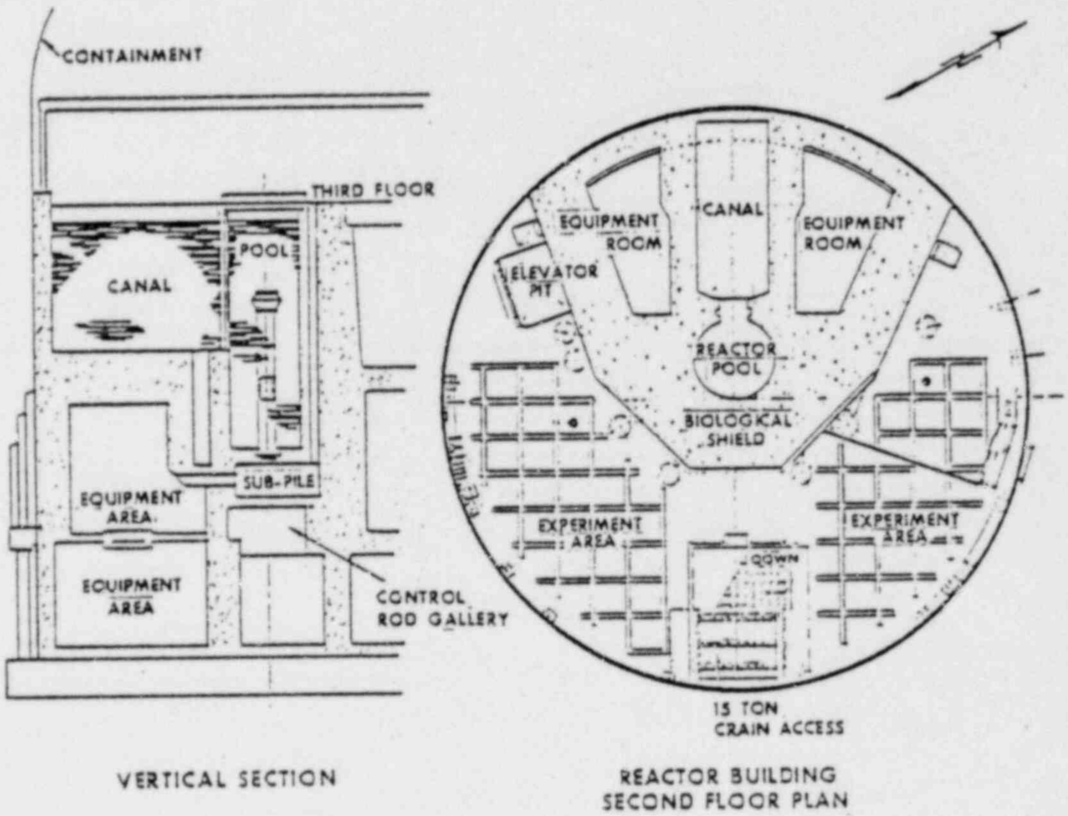

FIGURE 6.

A domed, cylindrical steel containment building (Figure 7) encloses the reactor, pool, adjacent fuel storage canal, shielding, heat exchangers, primary pump, and reactor servicing and experiment areas. The containment building extends approximately 90 feet above ground and 20 feet below ground

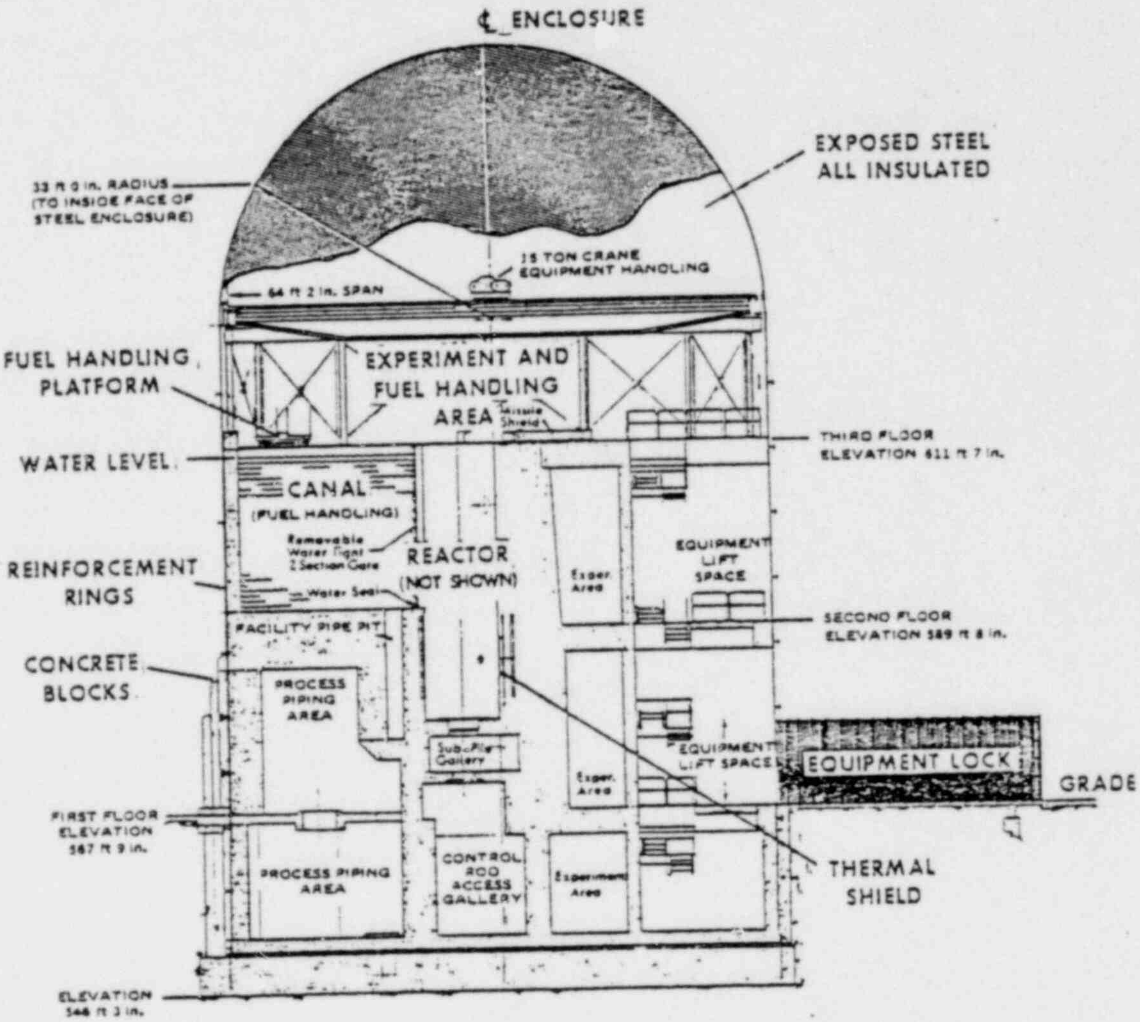

FIGURE 7. REACTOR BUILDING CENTERLINE SECTION

surface; the diameter is 66 feet.[6/] Containment building penetrations permit secondary coolant water to be pumped from the primary, pool and canal system heat exchangers to the cooling tower. Control and instrument penetrations permit reactor control and experiment instrumentation to be monitored in the adjacent reactor control room. Other penetrations are provided for process pipes and utilities.

A natural convection cooling system provides backup cooling for the reactor under certain emergency conditions (not necessarily design basis seismic conditions, for which an additional, separate fuel flooding system is available) and also during normal shutdown periods. A pneumatically reset, solenoid-tripped, spring-to-open, emergency cooling valve is provided on each leg of the two primary inlet cooling lines. (See PRI-130 and PRI-150 in Figure 8). In each of the primary coolant outlet lines in the reactor pool, check valves (installed vertically)

6/ Most operating test and research reactors of comparable size to GETR do not have leak tested containment structures enclosing and extending beneath the reactor, but are enclosed in substantial concrete buildings (reinforced or block) with filtered exhaust. These are best characterized as confinement structures, and include the High Flux Reactor (100 MW), Oak Ridge Research Reactor (30 MW), the Brookhaven Reactor (60 MW), the Engineering Test Reactor (175 MW), the Advanced Test Reactor (175 MW), the National Bureau of Standards Reactor (10-20 MW) and the University of Missouri Research Reactor (10 MW). The analysis for GETR does not take credit for the containment during design bases seismic conditions. Although the GETR containment has not been analyzed in detail under design bases seismic conditions, it will withstand vibratory ground motions in the order of 0.6g.

open due to gravity when the primary system is depressurized (PRI-140 and PRI-160, Figure 8). In the event of high reactor inlet temperature, low reactor differential pressure, low primary cooling flow or seismic switch trip, the reactor scrams and an emergency cooling trip signal causes the four valves (PRI 130, 150, 140, 160, Figure 8) to open the primary system to the reactor pool. If the primary pump continues to run, approximately 33% of the primary flow is bypassed to and from the pool with the cooler water from the pool mixing with the primary system. If the primary pump stops, the flow through the reactor reverses in a short interval; and natural convection cooling circulates from the pool through the open check valves up through the core and back to the pool (via the power-operated emergency cooling valves). The residual heat from the relatively small mass of the core and structure can easily be removed following shutdown or scram so long as makeup water is available (normally supplied from the pool via the vertical check valves into the bottom of the core).[7/] No electrical energy is required to maintain a safe shutdown status for extended periods. In the subsequent discussion, the additional backup cooling system (fuel

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7/ GETR has a decay heat load which is less than 2% of that for a large nuclear power plant. At about one minute after shutdown, the power level in GETR is 4% of full power, or about 2 MW. Within 40 hours, the decay heat load would decrease to 0.1 MW. This is equivalent to the heat transferred by the radiator of a large tractor-trailer truck.

flooding system) which is available to mitigate a design basis seismic event will be described.

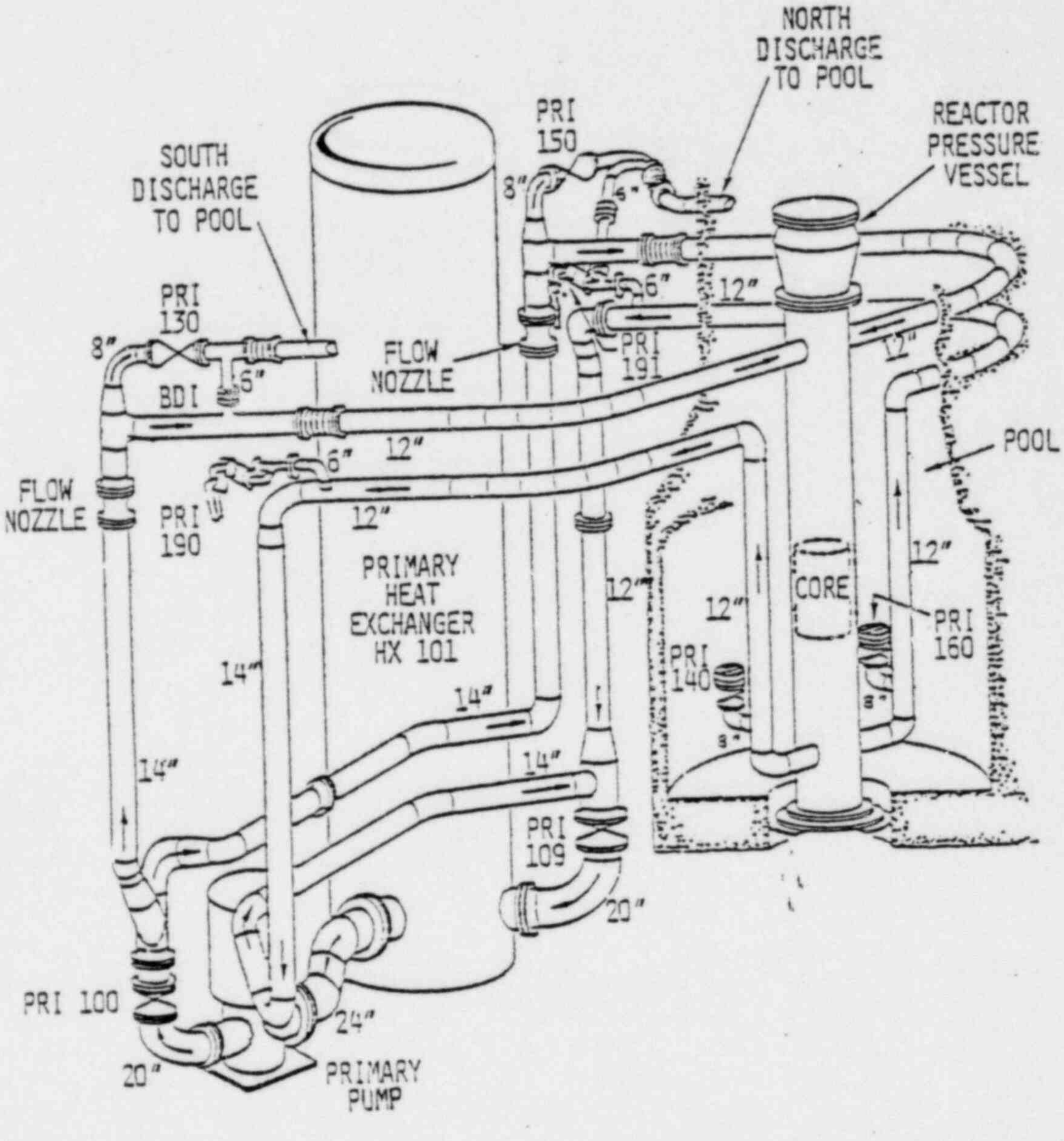

FIGURE 8. PRIMARY COOLING SYSTEM

II. FUNCTIONAL AND STRUCTURAL/MECHANICAL REQUIREMENTS AND MODIFICATIONS

There are three basic functional requirements which the GETR must satisfy for design basis seismic conditions:

(1) Reactor scram at the onset of the seismic event to terminate the fission heat source.

(2) Initial removal of decay heat by boiling/evaporation of the water inventory existing in the reactor pool and fuel storage canal at the onset of the seismic event.

(3) Long-term cooling/decay heat removal by providing sufficient makeup water flow to the reactor vessel and fuel storage containers.

These functional requirements will assure adequate cooling of the fuel under design basis seismic conditions. In simpler terms, these three functional requirements can be reduced to two: a) the reactor must be promptly scrammed, and b) the fuel must be kept covered with water.

From the standpoint of cooling the reactor core itself, the rupture of the primary coolant piping is the most limiting accident which can be postulated to follow from the design basis seismic event. The accident scenario is:

(1) The design basis seismic event occurs and reactor trip is initiated by the seismic scram system;

(2) The primary system piping is assumed to rupture simultaneously and nonmechanistically;

(3) Heat transfer and decay heat rates are those associated with a 25-day, full-power (50 MW thermal) run of the GETR.

Under this scenario, water will drain from the reactor vessel and pool through the primary return lines until the water reaches the level of the return line outlet from the reactor vessel (5.5 feet above the fuel). Further drainage is prevented by anti-siphon valves, and the system configuration reduces to the following (see Figure 9).

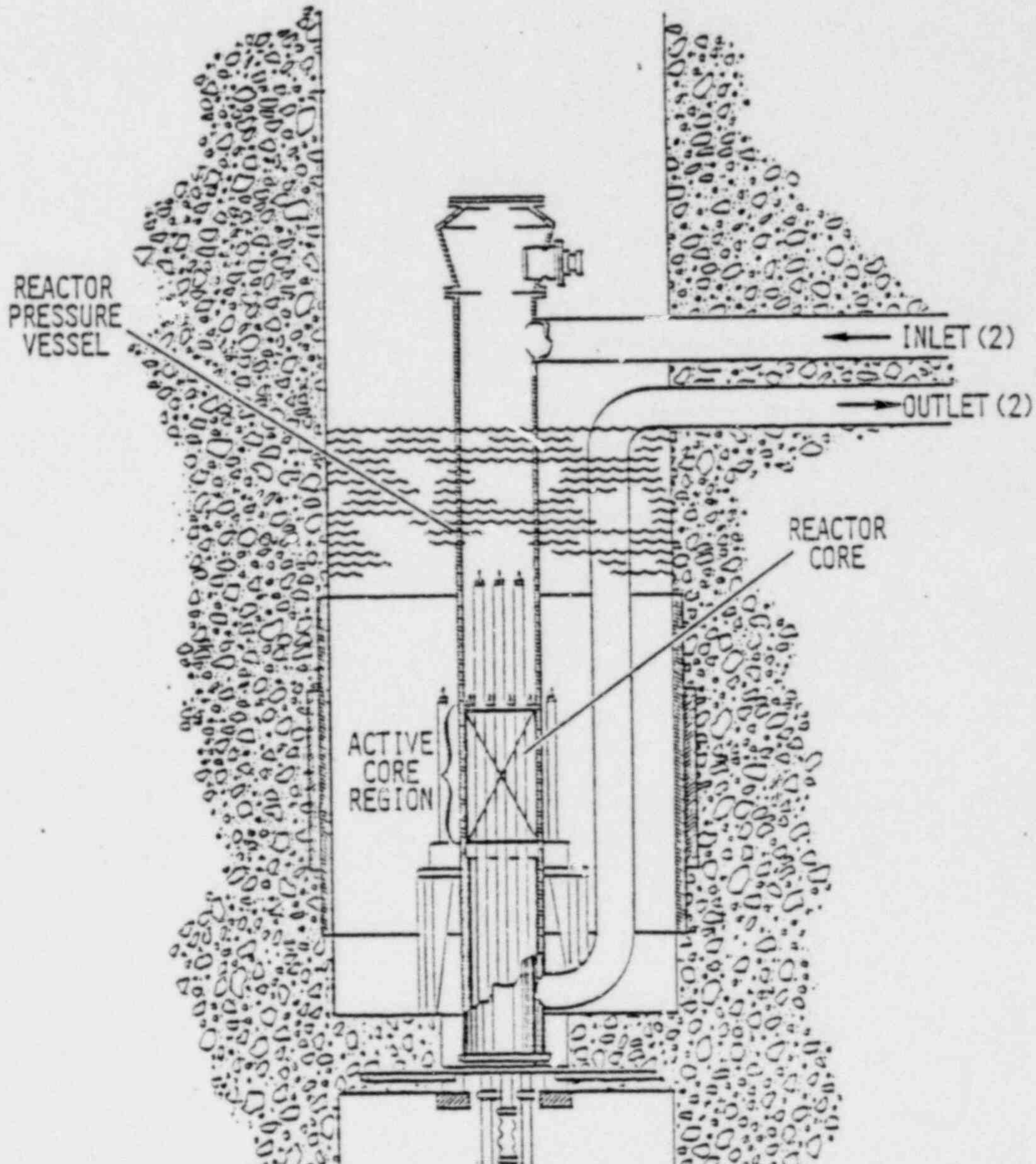

FIGURE 9. REACTOR AND POOL

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Under these conditions the initial 5.5-foot water level will drop to the top of the core at 45 hours after the event (assuming no makeup flow). Boil-off from decay heat would then require makeup water at a rate of .8 gpm. Heat transfer is sufficient to maintain the fuel cladding temperature low enough to preclude core damage. Thus, the problem of maintaining reactor core cooling simplifies to assuring: 1) that the reactor is scrammed, and 2) that the fuel remains covered with water.

From the standpoint of makeup flow requirements, the most limiting scenario involves the case in which a freshly discharged core has been taken from the reactor and placed in the fuel storage canal following reactor shutdown under the following assumptions:

(1) The design basis seismic event occurs 6 hours after reactor shutdown from the maximum 25-day run at full power (50 MW thermal).

(2) Fuel storage canal temperature is at a maximum of 130°F.

(3) The primary pipe rupture occurs simultaneously and nonmechanistically with the design basis seismic event.

Under these conditions, the water in the fuel storage canal will drop to the top of the fuel (assuming no makeup flow) about 34 hours after the event. Boil-off from decay heat would then require makeup water at a rate of 1.64 gpm. This makeup flow requirement would decrease with time, and there is no need for reactor pool makeup in this case. However, the fuel flooding system has been conservatively designed to provide a capacity and flow rate adequate to supply make up water to both the canal storage tanks and the reactor pressure vessel from each of two redundant systems. On this basis, the functional requirements for the GETR for this scenario reduce to assuring that the fuel in the storage canal remains covered with water.

Thus, the functional requirements for the limiting cases of reactor core cooling and makeup flow (pipe rupture and fresh fuel discharge to storage canal, respectively) are to: 1) promptly shut down the reactor, and 2) keep all fuel elements covered with water.

As to the first functional requirement, shutdown of the reactor is effected by inserting the control rod assemblies before consequential accelerations occur. The control rods are dropped from an initial 36-inch withdrawn position. This is initiated by action of either or both of two triaxial seismic switches that are set to trip at 0.01g.[8/] This, in turn, actuates the reactor scram system which causes the control rod assemblies to disengage and begin rod drop within 0.18 seconds of the triaxial switch having tripped. Once the triaxial seismic

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8/ This represents a few percent of the design basis seismic acceleration (0.75g). Although typical large nuclear power plants do not generally have seismic scram systems, those that do, such as Diablo Canyon, are set at about 60% of the safe shutdown earthquake acceleration.

switch has tripped and the scram system caused the control rod magnet (latches) to disengage, the seismic switch and scram system are no longer required to remain functional. The control rods and fuel follower assemblies drop by gravity and force of coolant flow, and are fully inserted within 0.5 seconds after initial control rod disengagement. Within 0.3 seconds from the disengagement (or 0.48 seconds from seismic switch trip), the control rods will be at or below the 12.2-inch withdrawn position, at which point the reactor is shutdown.

An evaluation of 94 earthquake records, including those from the Imperial Valley earthquake (1979), showed that consequential accelerations (the values were <0.08g horizontal and <0.2g vertical) were not reached within 0.5 seconds of a trip at 0.01g (reference 64, 67, 69).[9/] Thus, one can conclude that reactor scram is initiated within 0.18 seconds after acceleration reaches 0.01g, and reactor shutdown will occur within 0.3 seconds after seismic trip, while more than 0.5 seconds will elapse before accelerations in the range of 0.08g horizontal and 0.2g vertical are reached.

9/ References identified throughout this testimony are listed in Attachment A to the Feb. 25, 1981 Licensee's Supplemental Response to Intervenor's Discovery.

Since GETR commenced operation in 1958, a total of nine events have caused the present seismic triggers to operate. Four of these events have occurred since October 1977, including a Richter magnitude 4.1 event on March 3, 1981 with an epicenter on the Hayward Fault nine miles southwest of the GETR. In all cases, the seismic trigger functioned reliably. It should be noted that new seismic triggers and power supplies, which are seismically qualified to 0.5g, are being installed.

Additional analyses of the electrical systems were performed which show there is no credible way of inducing control rod withdrawal once scram occurred (reference 13). Finally, analyses were performed to assure that the control rod assemblies will not be forced out of the core by seismic motions.

On the foregoing bases, the following conclusions can be drawn in regard to reactor shutdown: 1) trip of the control rods and reactor shutdown will occur before consequential accelerations occur; 2) the control rods, once scrammed, cannot be withdrawn except by direct operator action; 3) the control rods will not be forced out of the core by vibratory motion; and 4) the system has operated reliably.

The second functional requirement (maintaining fuel covered with water) leads logically to a set of structural and mechanical requirements for the design basis seismic event.

Given the small size and simplicity of the GETR relative to a commercial nuclear power plant, safe response is assured without resorting to a large number of complex active electrical and mechanical systems.[10/] The following basic mechanical and structural requirements were developed to satisfy the second functional requirement:

1. The fuel element containers must be kept intact. In particular, the structural integrity of the reactor pressure vessel and canal fuel storage tanks must be assured against: a) unacceptable stresses by seismic-induced motion of those components, or by motion of attached piping or structure, or b) by potential missiles from other portions of the plant.
2. A water supply for boil-off and evaporation must be available (assuming that the normal fuel cooling system

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10/ As a result of its higher power level and power density, a typical large pressurized water reactor (PWR) nuclear power plant will have a high pressure injection system, core flooding tank system, and low pressure injection system. These systems are comprised of a large number of pipes, valves, tanks, and pumps, power supplies, and associated controls, all of which are redundant and diverse. In contrast, GETR has a simple passive emergency cooling system which circulates reactor pool water for cooling, and a gravity-fed fuel flooding system for makeup water under design basis seismic conditions. Neither system requires electrical power for its function. Typical PWR high and low pressure injection systems have makeup flow requirements in excess of 500 gallons per minute. GETR would require no makeup flow for about 40 hours after a design basis event, and a maximum of 1.6 gallons per minute thereafter (this requirement would further diminish with time after the event).

has failed). A sufficient source of water, including the associated piping system, must be available after the seismic event to provide water to the reactor vessel and spent fuel storage tanks to replenish that lost through boil-off and evaporation.

3. The concrete structure which encloses the canal and fuel storage tanks, and encloses and supports the reactor pressure vessel and fuel storage tanks must be kept intact.

Modifications have been made or designs are in place to meet the first two mechanical and structural requirements described above. None were necessary to meet the third requirement. The primary modifications are:

A. Modifications to provide additional assurance of reactor vessel integrity

The reactor pressure vessel is centered in the pool five feet below the top of the vessel with three restraints. The restraints attach to the side of the pool. Evaluation showed that one of the pins was of inadequate strength, and it was replaced.

There are four different kinds of restraints that are or will be installed on the primary piping system to eliminate stresses on the reactor vessel, thus assuring its integrity. The first kind strengthens the gusset below the 20-inch elbow connected to the primary pump discharge. A second restraint is a saddle and U-bolt arrangement that provides a vertical restraint for the 14-inch reactor vessel discharge pipe. The third type provides vertical restraint of the right pump discharge pipe and the left heat exchanger inlet pipe where the two run in parallel. It had been initially planned to attach this restraint to the underside of the canal floor. It is now planned to mount it on the floor of the equipment room. The fourth category of pipe restraints are collars that attach the pipes to the walls. There are 16 of them, and they consist of a clamp around the pipe with an interconnecting strut to a wall bracket.

In addition to the large pipe restraints described above, restraints were added to the small diameter piping that is connected to the bottom of the pool and the vessel.

Restraints were also added to the primary heat exchanger. Collars were placed around the heat exchanger near its top and center. Struts were installed between the collar and attachment points on the walls. In addition, a restraint is attached to the bolt circle on the bottom of the heat exchanger with struts connecting the restraint with attachment points on the walls.

Restraints were placed around the pool heat exchanger so it would not fall into the primary system piping. Standpipes were installed above the emergency cooling check valves so that in the unlikely event of loss of water from the pool, water would stay over the core.

B. Modifications to provide additional assurance of canal storage tank integrity

The canal storage tanks are located in the storage canal on the bottom at the end farthest from the pool. A new canal storage tank has been constructed that consists of three leak-tight inner tanks placed in a leak-tight outer tank. There are, thus, two leak-tight containers to assure water will remain over the stored fuel elements in the unlikely event that water is drained from the canal.

The inner tanks are constructed of one-quarter-inch 304 stainless steel, and the outer tanks are of one-half-inch 304 stainless steel. The thick-walled outer container also provides physical protection for the inner tanks.

Modifications have also been made to prevent equipment on the third floor from dropping on the canal storage tank or reactor pressure vessel. These are enumerated below.

(1) Structural bents were installed to catch or cause the polar crane to fall away from the canal or pool should it derail.

(2) Restraints were installed on the bridge trolley to assure it would stay in place on the bridge beams of the polar crane.

(3) Restraints are planned for the missile shield, and restraints are also planned to be added to the refueling bridge.

(4) Also planned but not constructed is a canal impact system which will preclude damage to the fuel storage tank should a cask fall and tip toward it.

C. <u>Modification to provide a water supply for boil-off and evaporation (Fuel Flooding System)</u>

A water supply system is planned that will assure a reliable source of water to the canal storage tank and reactor vessel to make up for water lost by boil-off and evaporation. An evaluation was made (reference 3) of the thermal-hydraulic effects of nonmechanistic pipe failures (resulting from a seismic event) upon the reactor fuel located in both the reactor pressure vessel and the canal storage tanks. This analysis demonstrated that long time periods (in the order of 40 hours) are available following a design basis seismic event before makeup water to the reactor pressure vessel or canal storage tanks is required and that the makeup requirements are very small (2 gallons/minute).

There will be two reservoirs (Figure 10) located on the hills above GETR, each with a capacity of 100,000 gallons. This is sufficient for a seven-day supply to both the canal storage tank and reactor vessel (reference 22) under the worst case (nonmechanistic) conditions attendant to the design basis seismic event after discharge of fuel from a full-power run (the thermal-hydraulic analyses are reported in detail in references 3, 13, 22). Each reservoir consists of two pillow or bladder tanks that are connected through inch-and-a-half piping that is routed down the hills to roughly opposite sides of the containment. Piping is routed inside the containment so that water is supplied to the canal storage tanks and to the reactor vessel. The water system is completely redundant. The Fuel Flooding System would begin to supply water when the triaxial scram switch trips. The water supply system is gravity fed, and no power is required for operation.

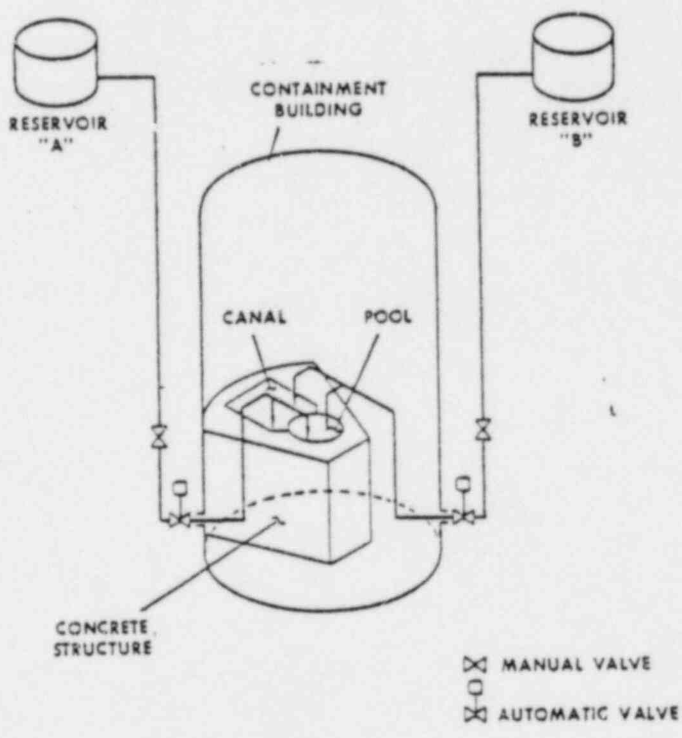

FIGURE 10. FUEL FLOODING SYSTEM

D. Summary

The General Electric Test Reactor is a facility of relatively modest size and complexity. Its reactor pool and canal and piping are located in the upper portion of a massive concrete structure. The reactor power level is 50 megawatts thermal. The reactor can be promptly and reliably shut down (scrammed) and maintained shut down in the design basis seismic event. The remaining seismic design basis system requirement is to maintain water over the fuel elements in the canal storage tank and reactor vessel, a requirement which will be met readily and reliably by the installed or planned modifications.

It remains, then, to address the structural and mechanical responses of the GETR structures, systems, and components necessary to achieve and maintain shutdown of the GETR in light of the three structural and mechanical requirements enumerated herein and the NRC Staff's design basis seismic criteria.

III. STRUCTURAL AND MECHANICAL ANALYSES

The structural and mechanical analyses described in this part of the testimony were performed to show that the GETR safety-related structures and equipment meet the following requirements when subjected to the design criteria loading:

A. Assure integrity of Reactor Building concrete core structure which supports other systems and components important to safety

B. Assure integrity of reactor pressure vessel

C. Assure integrity of canal fuel storage tanks

D. Assure capability of providing make-up water to spent fuel storage tanks and reactor vessel

The above requirements are met as follows:

A. Integrity of Reactor Building Concrete Core Structure - The investigations described in Section A of this testimony show that integrity of the concrete core structure which supports other systems and components important to safety is assured.

B. Integrity of Reactor Pressure Vessel - The integrity of the reactor pressure vessel is assured by demonstrating the adequacy of the concrete core structure, and by the investigations and modifications performed for the reactor pressure vessel and related piping and components. These investigations are described in Section B of this testimony. Restraints have been evaluated, modified, or added to meet the seismic design basis event for the:

- Reactor Pressure Vessel and Primary Cooling System
- Primary Heat Exchanger
- Reactor Pressure Vessel and Pool Drain Lines and Poison Injection Line
- Safety-Related Valves

- Lateral Restraints on Pool Heat Exchanger
- Control Rod and Incore Shuttle Assemblies

C. Integrity of Canal Fuel Storage Tanks - The integrity of the canal fuel storage tanks is assured by adequacy of the concrete core structure and the following investigations and modifications:

- New, structurally stronger tanks were constructed
- Structures were added and equipment was modified to prevent potential missiles from being generated or causing damage by installing:
    - Impact structure for the polar crane
    - Restraints on the polar crane trolley, missile shield, and refueling bridge
    - A canal impact pad to prevent damage due to cask tipping

The investigations performed for the canal fuel storage tanks are described in Section C of this testimony.

D. Provide Make-Up Water - Water in the canal fuel element storage containers and the reactor pressure vessel is replenished by a new Fuel Flooding System (FFS). This system begins to supply water when the scram switches activate. A redundant seven-day gravity flow (no power required) supply is designed and has been partially constructed. The investigations performed for the FFS are described in Section D of this testimony.

As summarized in Section E, the testimony demonstrates: 1) that the integrity of the Reactor Building concrete core structure, reactor pressure vessel, and canal fuel storage tanks is assured; and 2) the fuel flooding system will be available to provide adequate make-up water.

A. Integrity of Reactor Building Concrete Core Structure

The investigations described in this section of testimony demonstrate that the concrete core structure of the GETR Reactor Building, which supports other systems and components important to safety, is assured. The Reactor Building concrete core structure is shown schematically in Figure A-1.

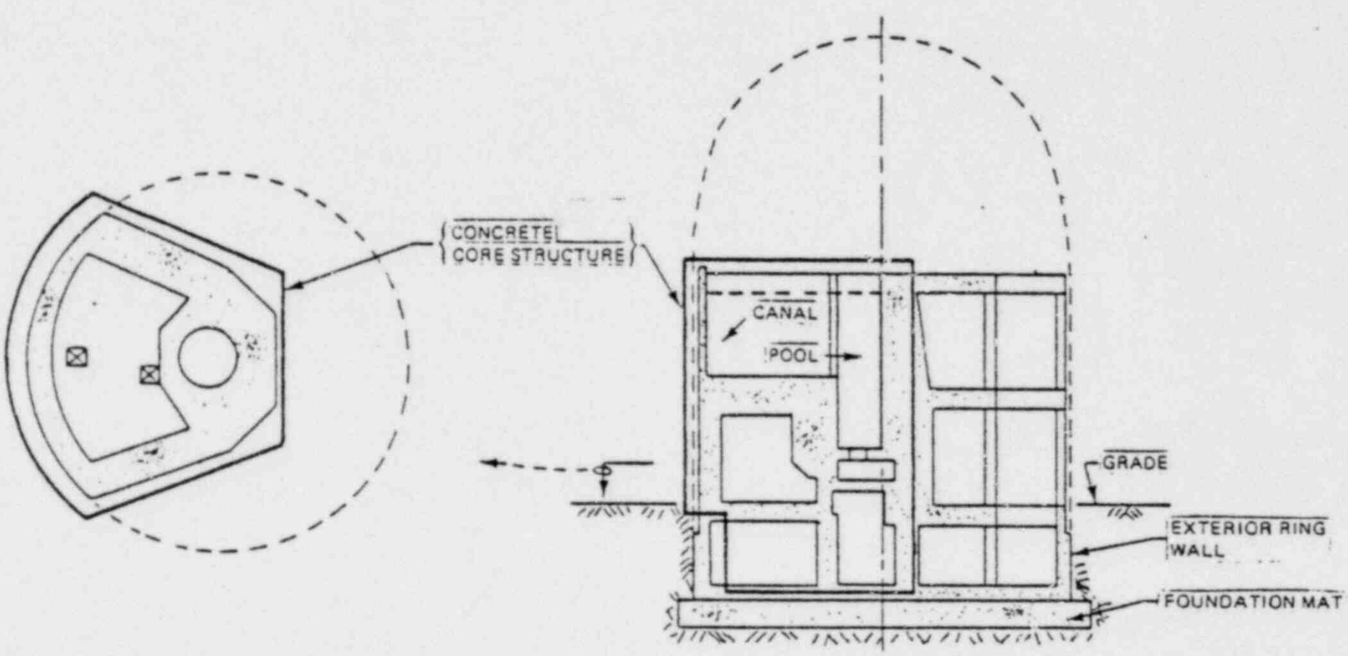

FIGURE A-1 RE ACTOR BUILDING CONCRETE CORE STRUCTURE

It can be seen from this figure that the concrete core structure is of heavy massive construction. Figure A-2 shows the Reactor Building floor plans at the basement, first-floor, second-floor, and third-floor levels, and further illustrates the massiveness of the concrete core structure.

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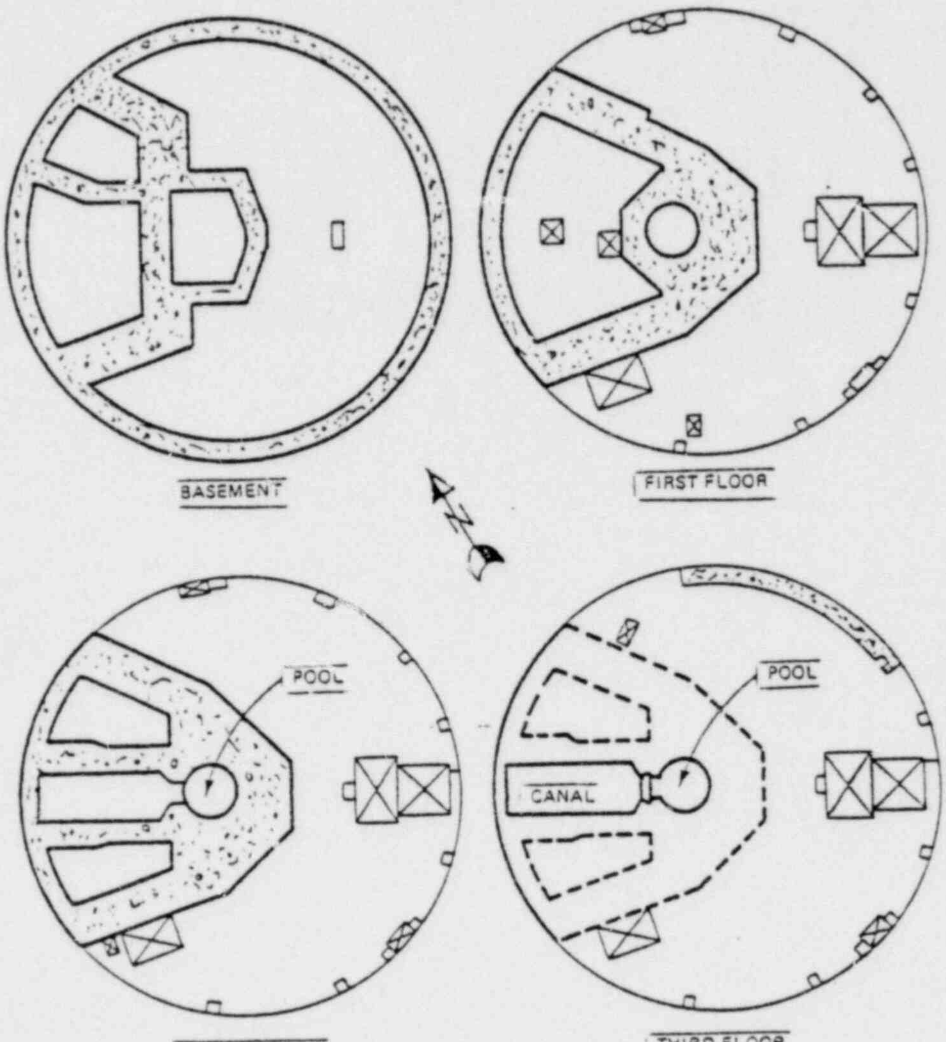

FIGURE A-2 REACTOR BUILDING FLOOR PLANS

A 4'-8"-thick by approximately 70'-diameter concrete foundation mat supports the building. In the space between the basement and the first floor slabs, the periphery of the building

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is enclosed by a cylindrical concrete wall (called the "exterior ring wall" in this testimony), which is cast in-place against the steel containment shell. The concrete core structure consists of the biological shield surrounding the pool, and two 6'-6"-thick radial walls which extend from the basement slab to the third floor. This area contains the reactor pool and the storage canal. The second and third floor slabs are connected to the concrete core and supported on the periphery by columns which extend to the exterior basement wall. Figure A-3 shows an overall isometric view of the concrete core structure.

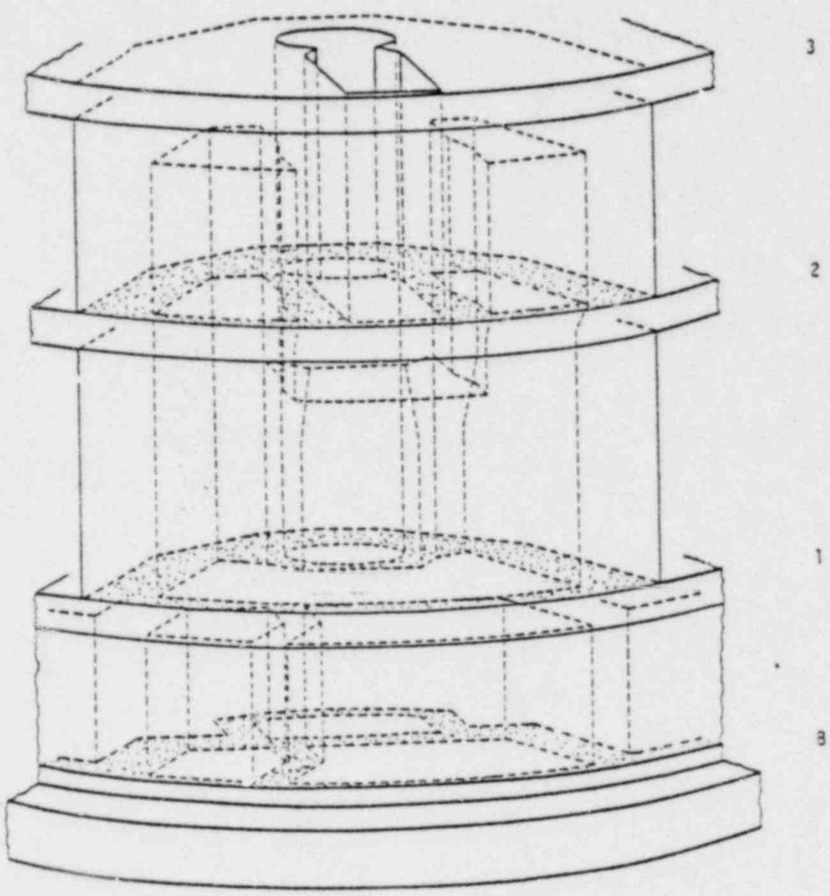

FIGURE A-3. ISOMETRIC VIEW OF CONCRETE CORE STRUCTURE

It can be seen from the above figures that the concrete portion of the Reactor Building is relatively short or squat and well-embedded in the firm foundation soil. The ratio of the height above grade to the width of the entire interior concrete structure is approximately 0.65, and it is approximately one-third embedded. This type of structure is stiff and behaves well when subjected to earthquakes. The earthquake ground motions tend not to be amplified by the structure.

A program of investigations was undertaken to demonstrate the adequacy of the concrete core structure to withstand seismic effects postulated for the site. The investigations focused on two basic earthquake phenomena:

- o Ground shaking due to an earthquake on the Calaveras or Verona faults.
- o A ground displacement, denoted "surface rupture offset," at the site due to an earthquake on the Verona fault.

The ground shaking phenomena can be visualized as follows. When seismic waves pass through the earth's crust, the ground at the site, including the ground upon which the building is supported, is moved, and this movement varies rapidly with time. From a structural engineering point of view, this ground motion, called ground shaking, can be defined as three-dimensional translation of particular points in the ground, which are either away from or near the building (Figure A-4).

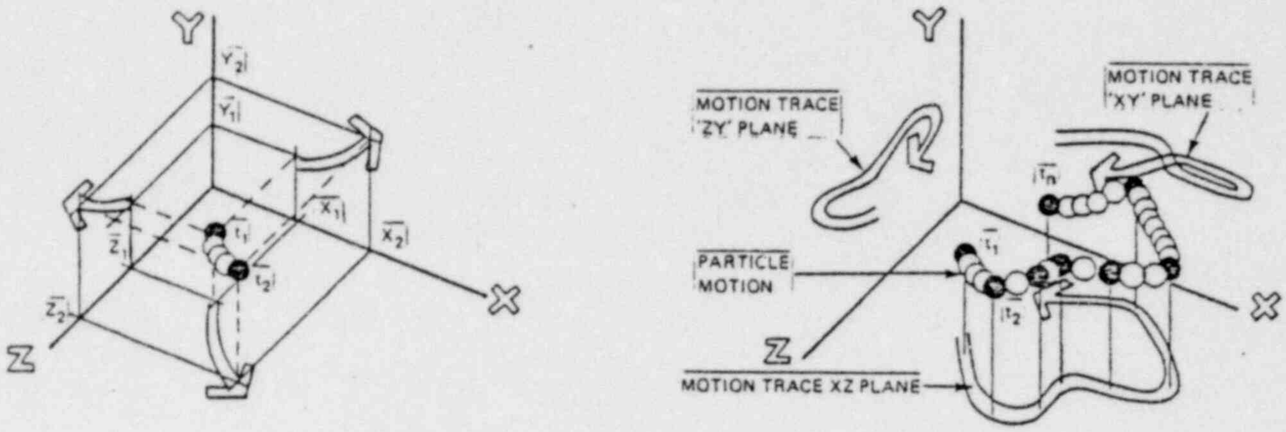

FIGURE A-4 THREE DIMENSIONAL GROUND MOTION

The three translational components of ground motion are used to describe the amplitude of the seismic ground motion (acceleration, velocity, or displacement) at any time during the earthquake. While this discussion may seem elementary, it is conceptually useful to illustrate the nature of the seismic motion. In general, recorded earthquake motions indicate similar characteristics in all three directions, and for qualitative discussions, the three components can be assumed to have similar overall characteristics. As an example of a typical accelerogram, a horizontal component of the 1952 Taft California earthquake is shown in Figure A-5.

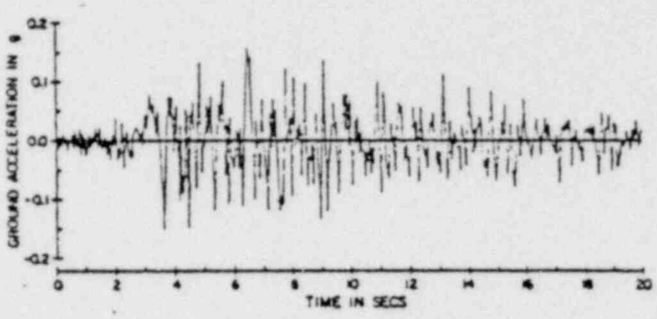

FIGURE A-5 TYPICAL EARTHQUAKE RECORD

The other two components recorded at the same sites, while having different maximum amplitudes, were similar in overall nature. Referring to Figure A-5, it can be seen that these motions have the general characteristic of faint vibrations at first for a short period of time, followed by more vigorous vibrations lasting for some time, after which the vibrations gradually disappear. The consequence of this ground shaking is a rapidly varying motion of the foundation of the building. This foundation motion, in turn, causes the walls, columns, and floors to move rapidly with time, as illustrated for an idealized, flexible, conventional three-story building in Figure A-6.

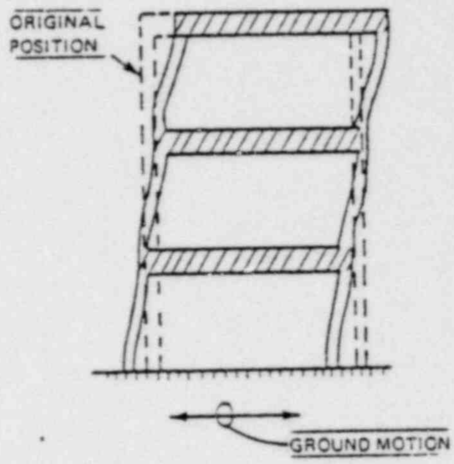

FIGURE A-6 MOVEMENT OF FLEXIBLE BUILDING

In this figure, each of the columns is deformed as the building moves with time. (For ease of illustration, no walls are shown, nor are deformations of the floors.) The column deformations produce axial forces, shear forces, and moments in the columns as illustrated in Figure A-7.

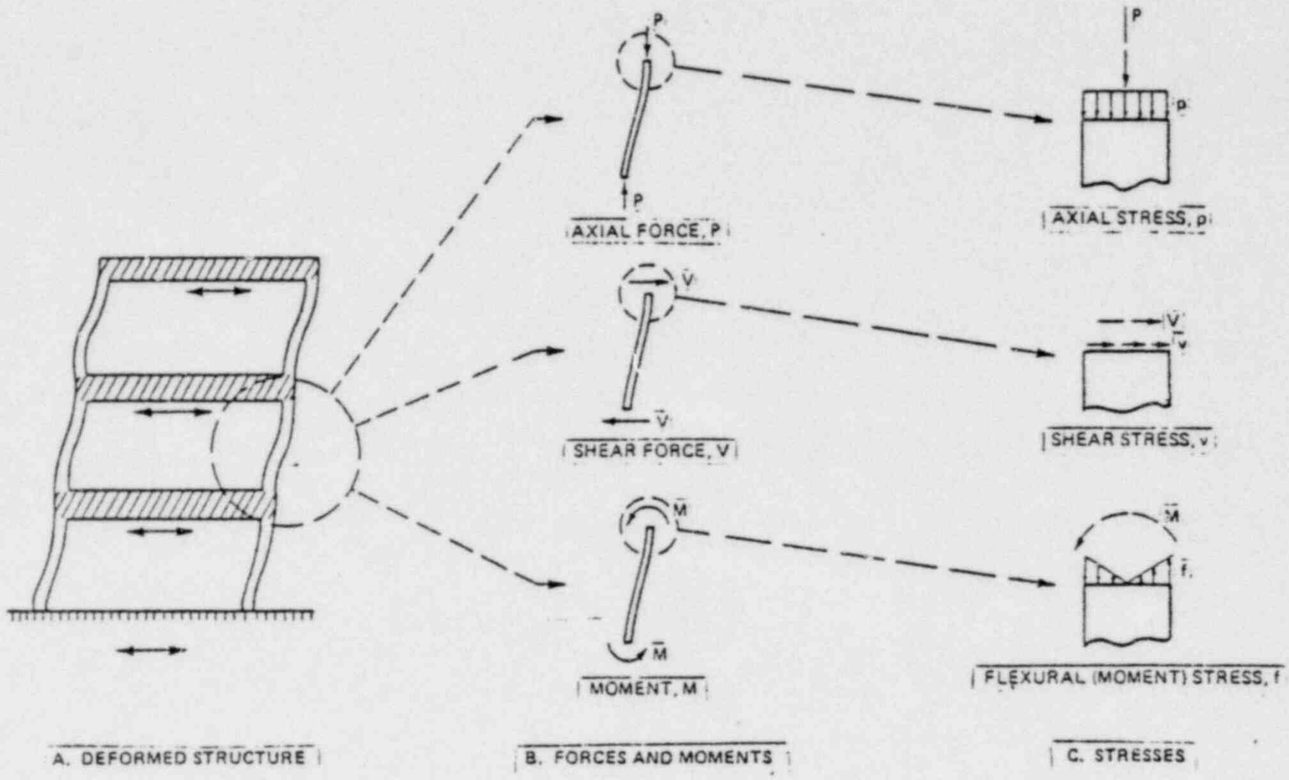

FIGURE A-7 BUILDING DEFORMATIONS

Figure A-7(A) shows the deformed structure. The forces and moments, and the direction in which they act, are shown by the arrows in Figure A-7(B). The forces and moments can be defined as a strength or power exerted on an object (i.e., the column in this case). Forces, when applied over a surface area of an object, are denoted as stresses, and are expressed in terms of force per unit area (e.g., pounds per square inch).*/ Stresses can be produced by any of the forces and moments in Figure A-7(B), as illustrated in Figure A-7(C).

It is thus the goal of the structural analyses to determine the (1) deformations of the building, (2) forces and moments, and (3) stresses. To evaluate the adequacy of a structure, these stresses are then compared against maximum permissible values, which are related to the strength of the structural material in question.

The behavior of the GETR Reactor Building will be different from that illustrated in Figure A-6. The stiff concrete core structure will not exhibit deformations as shown for the columns in Figure A-6, but it will move essentially as a rigid block, and the more significant deformations will not be in the structure, but in the foundation soils (Figure A-8).

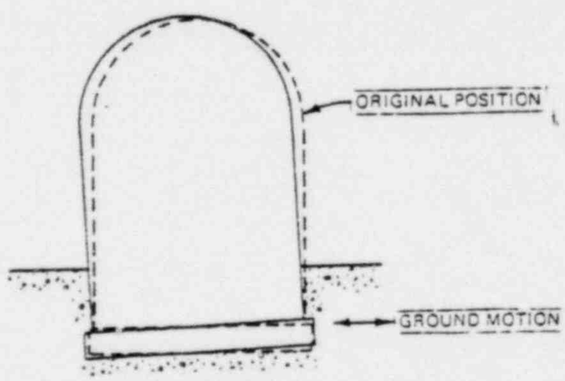

FIGURE A-8 MOVEMENT OF RIGID BUILDING

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*/ Moments can be expressed in terms of forces, which then can also be expressed by forces per unit area.

The concrete core structure will remain virtually undeformed and will move as a unit. It will be, in effect, held by the surrounding soils.

As a further introductory note, it is worthwhile to examine the concept most often used to represent pertinent engineering characteristics of earthquake ground shaking. As described previously, and illustrated in Figure A-5, the earthquake motions can be represented by a plot of motion (e.g., acceleration) versus time. Alternatively, the concept of a response spectrum can be used.

In its most general sense, a response spectrum is a way of representing the behavior of a simplified (e.g., one-story) structure when subjected to an earthquake. A one story structure is used because it can be idealized mathematically as a "single degree of freedom" system.*/ Structural dynamic analysis techniques are used to extend these ideas regarding the response system from simple to complex structures. A qualitative description of a response spectrum and its use follows.

A response spectrum represents the maximum responses of a series of single degree of freedom oscillators subjected to a given earthquake motion. This concept can be illustrated as follows. It is convenient to visualize the response spectrum in

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*/ A system in which behavior can be expressed in terms of movement in a single direction, e.g., the movement of the roof horizontally.

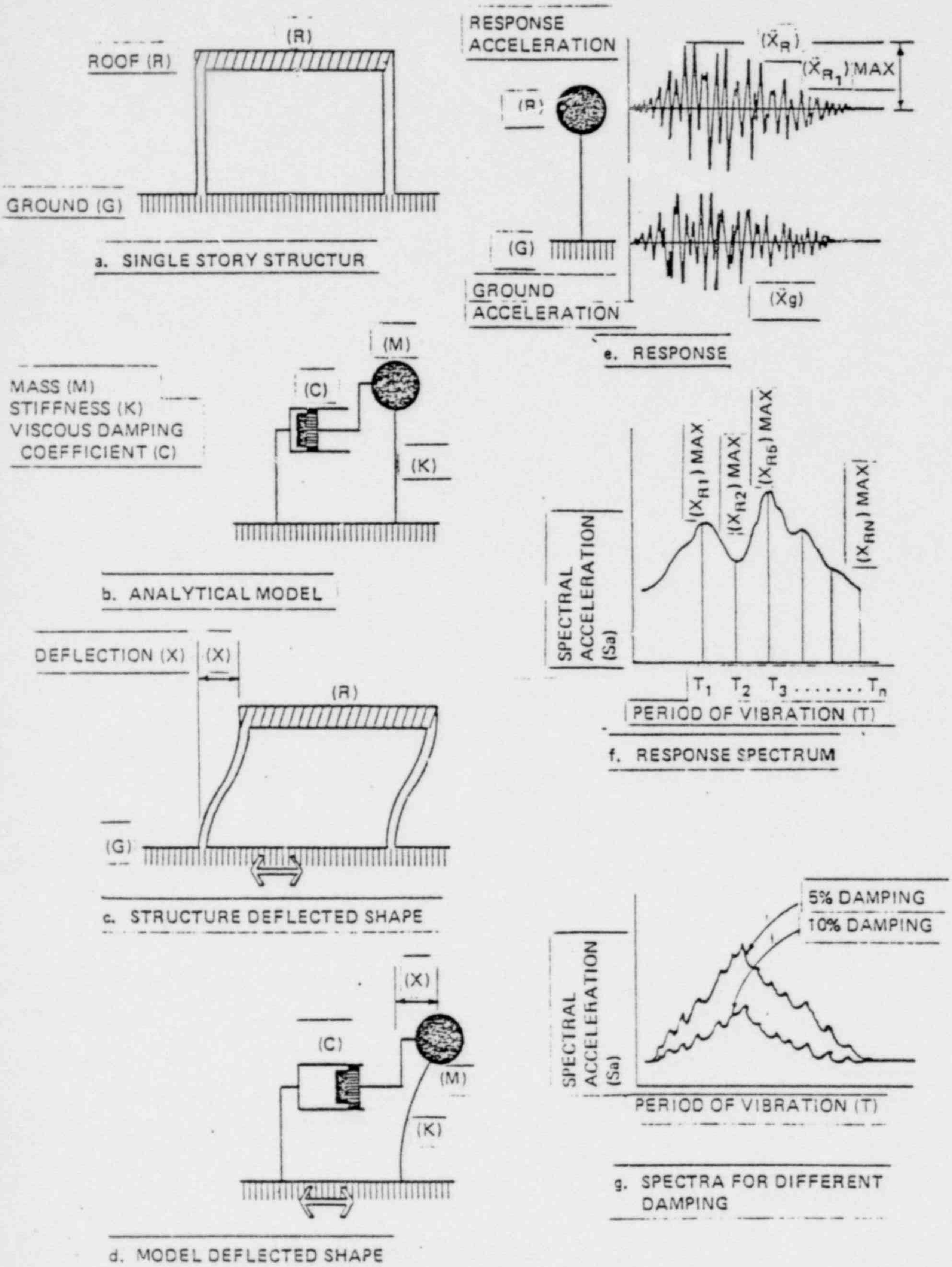

FIGURE A-9 EARTHQUAKE RESPONSE OF A SINGLE STORY STRUCTURE terms of the earthquake response of a single story structure as shown in Figure A-9.

The structure shown in Figure A-9(a) can be idealized by the analytical (i.e., mathematical) model as shown in Figure A-9(b). In this model, the mass M represents the weight of the roof and the tributary weight of the walls and columns. The stiffness K represents the stiffness of walls and columns between the ground and the roof. The energy dissipation properties of the structure can be represented by the dashpot shown in Figure A-9(b) which can be assumed to have a viscous damping coefficient of C. This dashpot is analogous to a shock absorber on an automobile.

The structure shown in Figure A-9(a) and represented by the analytical model of Figure A-9(b) has a period of vibration $T=2\pi\sqrt{M/K}$. The period of vibration of the structure is the time required for the structure to complete one full cycle of oscillation. For example, if the structure is "pulled to the right" by an imaginary "X" as shown in Figure A-9(c), and then released, the time required for it to displace to the left and return to the right is the period of the structure.

The deflected shape of this structure at any instant of time when subjected to an earthquake ground motion could be as shown in Figure A-9(c). The similar deflected shape of the single degree of freedom idealized system is shown in Figure A-9(d).

Assume that the single degree of freedom system is subjected to earthquake input motions as shown in Figure A-9(e). This input motion is shown as a plot of ground acceleration ($\ddot{X}_g$) versus time t. The response (in this case acceleration) of the single degree of freedom system can be calculated, and is shown in the top of Figure A-9(e) and represented as $\ddot{X}_R$. Assume that the system of Figure A-9(e) has a fundamental period of vibration of T_1. The maximum value of the roof acceleration time history is shown as $(X_{R1})_{max}$ in Figure A-9(e). This maximum acceleration can be plotted versus the period of vibration of the single degree of freedom system, T_1, Figure A-9(f).

This procedure can be repeated for a series of single degree of freedom systems with fundamental periods of T_1, T_2, . . . T_n, and a spectrum of maximum accelerations for this series of single degree of freedom systems will have the characteristics as shown in Figure A-9(f). The ordinate, which is the acceleration of the single degree of freedom system, is called the spectral acceleration, S_A, and is plotted versus the period of the single degree of freedom systems, T. The resulting plot shown in Figure A-9(f) is called a "response spectrum." The same procedure can be repeated for a series of systems with different damping ratios and the curves can be plotted as shown in Figure A-9(g) to produce response spectra.

Such curves are useful in design as follows. Assume that it is necessary to determine the maximum acceleration and the shears and moments in the column in a one-story building (for example, as shown in Figure A-9(a)). This can be accomplished by first calculating the weight of the roof and tributary walls and the stiffness of the walls and columns between the roof and floor. From these values the period of vibration of the building can be calculated. Using this period and the assumed damping ratio, the maximum acceleration S_A of the roof of the structure can be obtained from the appropriate response spectrum. This acceleration, when multiplied by the weight of the roof, produces the shear forces and moments in the columns. The design can then be checked to determine its ability to withstand these forces and moments.

The above has been a qualitative discussion of the ground shaking phenomenon as it affects structures; the second earthquake phenomenon considered for the GETR Reactor Building was a surface rupture offset beneath the building. This phenomenon is shown schematically in Figure A-10, and the effects on the structure can be visualized as forces exerted by the foundation soil on the structure as the movement on the fault occurs.

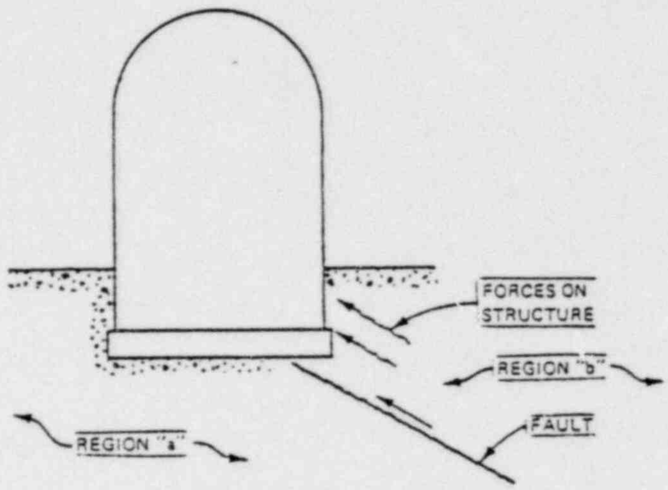

FIGURE A-10 ILLUSTRATION OF SURFACE RUPTURE OFFSET

In this figure, the earth in region "b" can be visualized as moving upwards to the left; and the earth in region "a" remaining stationary. The structure is thus held by the soil in region "a," and pushed on by the soil in region "b." The structural investigations must then focus on the adequacy of the structure to withstand this pushing force.

The concrete core structure of the GETR Reactor Building was therefore analyzed to ensure its integrity when subjected to both ground shaking and surface rupture offset effects.

The investigations were divided into three main tasks consistent with the seismic criteria for the site:

1. Analysis for effects of vibratory ground motions caused by an earthquake on the Calaveras fault (described in subsection 1 below).

2. Analyses for effects of vibratory ground motions combined with a surface rupture offset caused by an earthquake on the Verona fault (subsection 2 below).

3. Analyses for effects of vibratory ground motions caused by an aftershock (subsection 3 below).

The above analyses are described in detail in References 19, 56, and 60, and are summarized in this testimony. In addition, the numerous conservatisms in the seismic evaluations of the GETR Reactor Building are qualitatively examined in Reference 56. Topics included in this reference include characterization of earthquakes, the Verona fault, analytical models, and strength and capacity. This reference concludes that, if all individual safety margins in each main step or parameter in the evaluations were quantified, the result would be a total margin of safety significantly above that conservatively determined by the seismic evaluations of the GETR Reactor Building.

1. Evaluations for an Earthquake on Calaveras Fault

Analyses for an earthquake on the Calaveras fault were performed for the vibratory ground motion criteria recommended by the NRC staff; i.e., 0.75g effective ground acceleration anchored to the Regulatory Guide 1.60 response spectrum. Mathematical computer models were used to represent the physical characteristics of the concrete core structure and its behavior when subjected to earthquakes. The main characteristics included in the models were the physical geometry of the walls, floors, and columns of the structure; the weights of each of these components; and the stiffness (i.e., resistance to deformation) of each of these components. Also represented were the ability of the structure to dissipate energy (damping) as it moves and the deformation characteristics of the supporting soil around and beneath the structure. Where appropriate, the stiffness of components which change with time during the earthquake was also represented (called "nonlinear behavior").

Computer simulations of the behavior (called "response") of the structure when subjected to earthquake ground motions were developed. The overall process to accomplish this is illustrated schematically in Figure A-11.

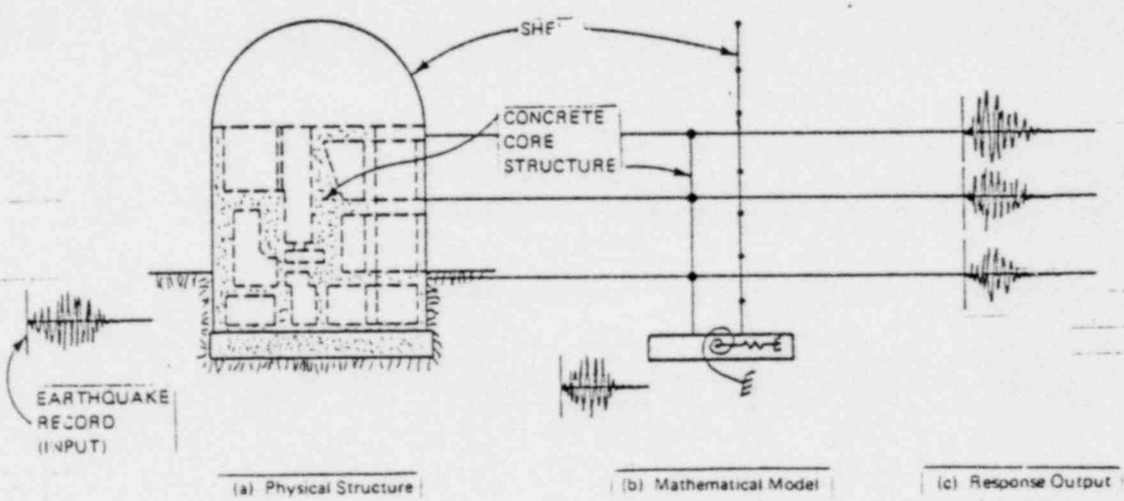

FIGURE A-11 COMPUTER SIMULATION PROCESS

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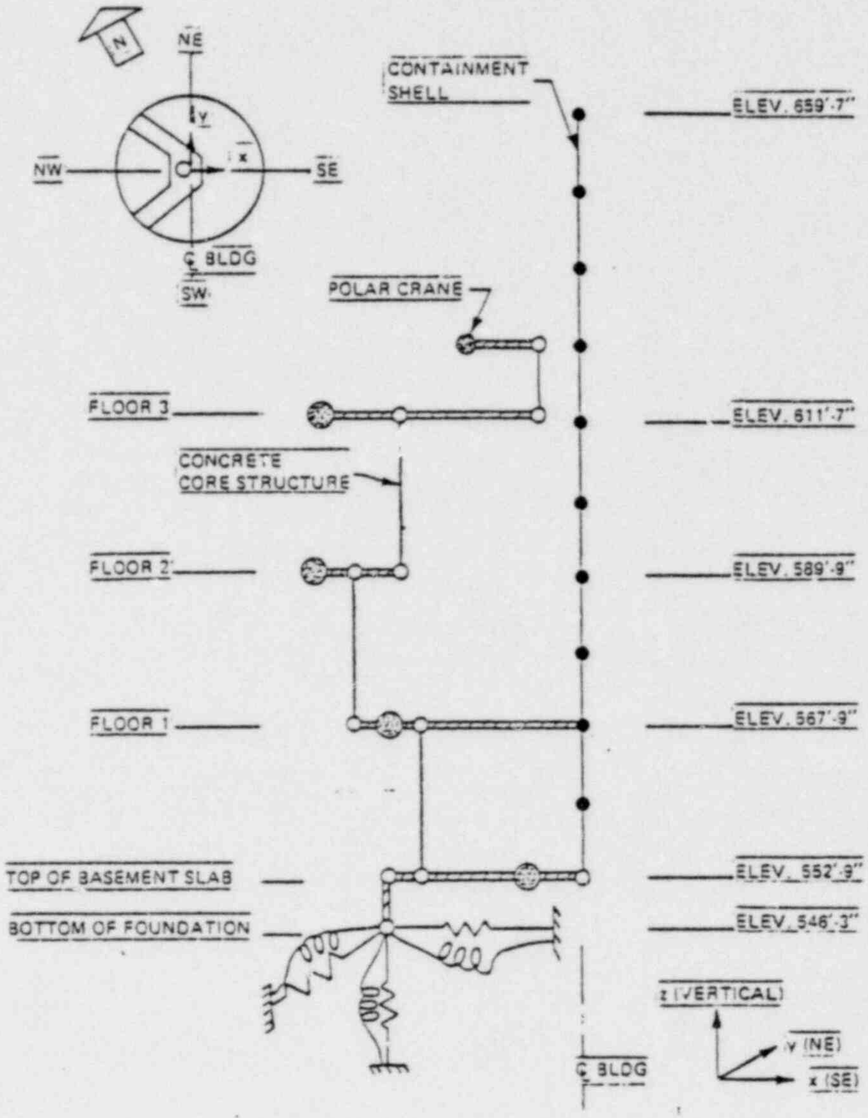
Figure A-11(a) shows the physical structure and Figure A-11(b) shows the mathematical model used in the computer analyses. In these analyses, the prescribed earthquake record is used to shake the model, and the resulting response records are obtained at the floor levels, Figure A-11(c). These response records are commonly the acceleration and displacements, as a function of time, of the floors. At the same time, the forces in all the walls and other relevant components are obtained, stresses calculated, and compared with criterion values. If these stresses are less than the criterion values, it can then be concluded that the structure is adequate to withstand the prescribed seismic effects. Note that the example in Figure A-11 only shows motions in one horizontal direction. The actual analyses included both horizontal directions and the vertical direction.

The detailed structural investigations were divided into numerous tasks and subtasks. Linear elastic analyses were first performed. These analyses employed a lumped-mass "cantilever" model with foundation springs to represent the deformations of the soil materials beneath the structure (Figure A-12).



(NOT TO SCALE)

FIGURE A-12 MATHEMATICAL MODEL FOR LINEAR ELASTIC DYNAMIC ANALYSES

Torsional effects (i.e., the possibility of twisting of the structure about a vertical axis) were considered by including eccentricities between the centers of mass and rigidity at each floor level. Shear forces and moments were computed at all levels, and in-structure response spectra were generated at all floor level elevations for later use in the seismic investigations of supported piping and equipment. (The definition of in-structure response spectra is presented in Section B of this testimony.)

To fully examine the sensitivity of the behavior of the Reactor Building to the important parameters which could influence this behavior, a series of parametric analyses were performed to determine the influence of soil shear modulus, average area of contact between the building and the soil, influence of variation in modal damping, contributions of torsion, and foundation embedment effects. In addition, potential nonlinear effects were investigated in detail by performing nonlinear analyses. For this purpose, two mathematical models were employed. The first consisted of a single lumped mass cantilever supported on soil springs, which was used to investigate nonlinearities associated with potential uplift and sliding of the Reactor Building. The second model consisted of two lumped mass cantilevers connected by slab elements and supported on soil springs, which was used to investigate nonlinearities associated with possible cracking of the Reactor Building slabs (exterior to the concrete core structure) and the uplift behavior of the building. These nonlinear analyses confirmed the conservatisms of the results of the linear elastic analyses.

After the dynamic response of the Reactor Building concrete core structure was determined by performing the analyses described above, stress analyses were performed using a detailed finite element model of the reactor concrete core structure. This model represented the detailed geometry of the concrete core structure, and consisted primarily of three-dimensional solid elements. Forces obtained from the dynamic analyses were applied in a conservative fashion to determine internal stresses within the concrete core structure. Manual calculations were also performed to determine the stresses in the concrete core structure to confirm the stresses obtained from the finite element analyses.

From the analyses described above, it was found that the stresses in the concrete core structure which surrounds the pool and the storage canal, and which also supports and protects the safety-related equipment and components necessary for safe shutdown, induced by an earthquake on the Calaveras fault were smaller than the capacity stresses. These stresses were based on the forces obtained from linear elastic dynamic analyses. Nonlinear analyses, including the nonlinearities due to potential uplift and sliding of the Reactor Building at the foundation slab-soil interface, were also performed. The forces obtained from these nonlinear analyses were smaller than those obtained from the linear analyses, which therefore confirmed the conservatism of the linear analyses. The nonlinear analyses also demonstrated that the Reactor Building is stable against potential uplift and sliding and that the soil pressures remain below the soil capacity. The analyses also showed that, although some cracking of slabs exterior to the safety-related concrete core structure could occur, the deformations of these slabs would be small, resulting only in minor and insignificant non-safety related cracking.

It was therefore concluded that integrity of the Reactor Building concrete core structure is assured for an earthquake on the Calaveras fault.

2. Evaluations for an Earthquake on the Verona Fault

A series of investigations were performed to demonstrate that the concrete core structure of the GETR Reactor Building is adequate to withstand the effects of combined vibratory motions and surface rupture offset due to an earthquake on the postulated Verona fault. These investigations were performed for the ground motion parameters recommended by the NRC staff; i.e., 0.60g effective ground acceleration anchored to the NRC Regulatory Guide 1.60 response spectrum, combined with a 1.0 meter offset. Previous testimony has indicated that, if an earthquake were to occur on the Verona fault, the soil materials beneath the Reactor Building would deform in such a way that the failure zone would bypass the Reactor Building foundation. In such a case, the building would only be subjected to vibratory ground motions. Since these motions would be less severe than for the Calaveras criteria described above in testimony Section A.1, it can be concluded without further investigation that the concrete core structure is adequate to withstand an earthquake on the Verona fault. In addition, as also concluded in previous testimony, the surface rupture offset under the foundation at the GETR site is an event with a probability so low that one could exclude it from the design bases.

Even though (1) the surface rupture offset failure zone will bypass the Reactor Building foundation, and (2) the probability of surface rupture offset at the site is low, the surface rupture offset was very conservatively assumed to occur; and the adequacy of the Reactor Building concrete core structure was evaluated to determine its adequacy to withstand this occurrence. The resulting structural investigation showed that the concrete core structure is adequate to withstand the combined load case of vibratory ground motions and a surface rupture offset.

The same basic structural analysis procedures were used as described previously in testimony Section A.1 for the Calaveras earthquake case. Computer models were used to simulate the physical characteristics of the structure and its behavior when subjected to both vibratory motions and the surface rupture offset. To avoid repetition, descriptions of these procedures will not be presented in this section of testimony.

For the earthquake on the Verona fault, the loading produced on the GETR Reactor Building depends upon the point at which the surface rupture offset is assumed to intersect the foundation or side walls of the Reactor Building structure. Thorough investigations of the different possible locations of intersection were made. As a result, two basic loading cases were identified.

The first basic case (Case 1) primarily involves the potential for production of soil pressures on the exterior ring wall between the basement and first floor levels of the Reactor Building. Case 1 is illustrated in Figure A-13.

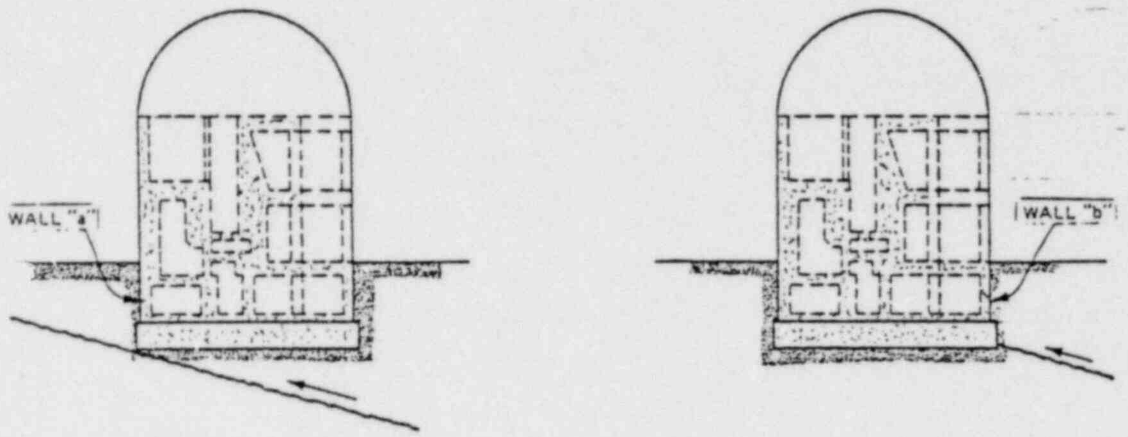


FIGURE A-13 CASE 1 – POTENTIAL FOR SOIL PRESSURES ON WALLS

In Case 1A in this figure, the surface rupture offset is assumed to intersect the horizontal plane of the base of the foundation at a location slightly to the left (on the figure) of the base slab of the Reactor Building. In this case, the primary area of concern was the soil pressure loading on wall "a," which is a short section of the exterior ring wall. This pressure loading is caused by the force required to push the wedge of soil on the left-hand side of the Reactor Building upwards and to the left on the inclined plane of the fault. Analyses were performed to determine this loading and the resulting effects on wall "a" and the concrete core structure. It was determined that there may be some cracking and deformation of the ring wall between the basement and the first floor due to the soil pressures against the ring wall on the left-hand side of the building. It was also determined that this deformation has no adverse effect on the concrete core structure, since the concrete core structure need not rely on wall "a" for support. If the support provided by wall "a" were totally neglected, the remaining walls still provide adequate support. (See Figure A-14).

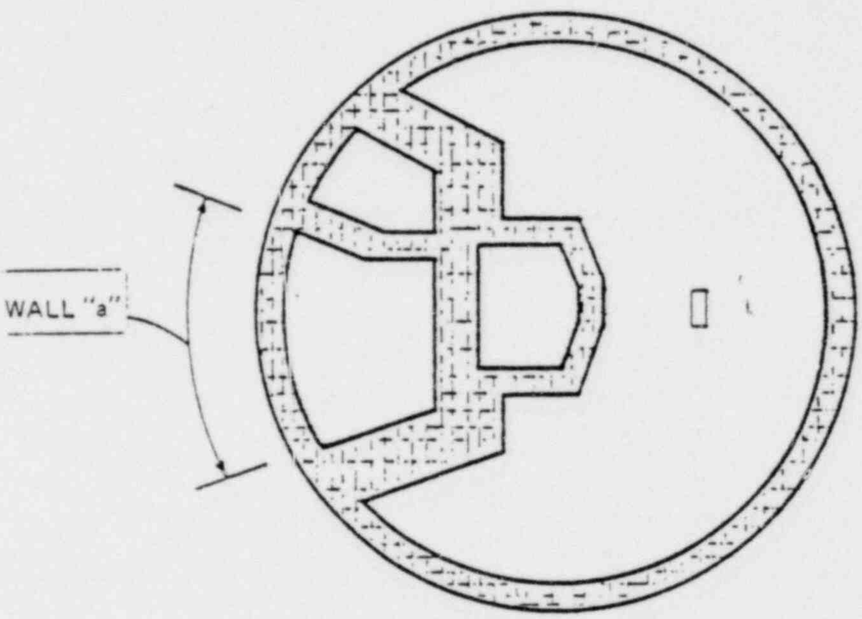


FIGURE A-14 PLAN VIEW, BETWEEN BASEMENT AND FIRST FLOOR LEVELS, CASE 1A

In Case 1B of Figure A-13, the surface rupture offset was assumed to intersect the Reactor Building on a vertical plane on the right-hand side of the figure. This case is similar to Case 1A above, except that the primary area of interest was the soil pressure loading on wall "b," which is also a portion of the exterior ring wall between the basement and first floor levels. Analyses indicated that, similar to wall "a," wall "b" may be deformed and cracked due to the soil pressures. This deformation would have no effect on the concrete core structure, since the core structure does not rely on wall "b" for its support. (See Figure A-15).

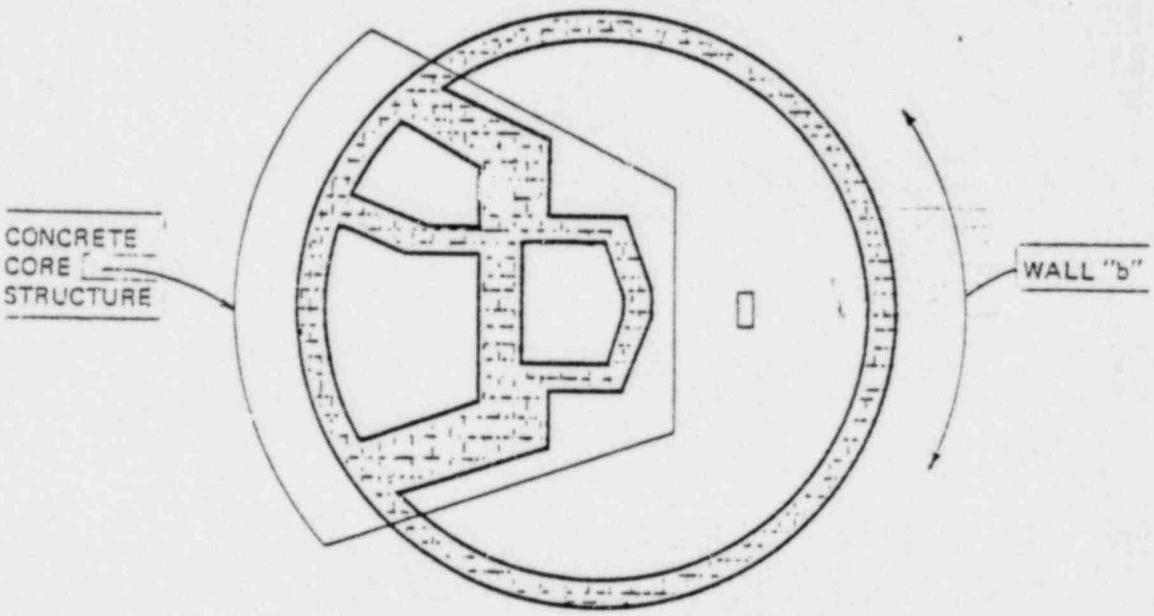


FIGURE A-15 PLAN VIEW BETWEEN BASEMENT AND FIRST FLOOR LEVELS, CASE 1B

It was therefore concluded that the concrete core structure is adequate to withstand the loadings represented by Case 1, which primarily produces soil pressures on the exterior ring wall between the basement and first floor levels of the Reactor Building.

In the second basic case (Case 2), the surface rupture offset is assumed to intersect the concrete foundation mat of the Reactor Building. Case 2 is illustrated in Figure A-16.

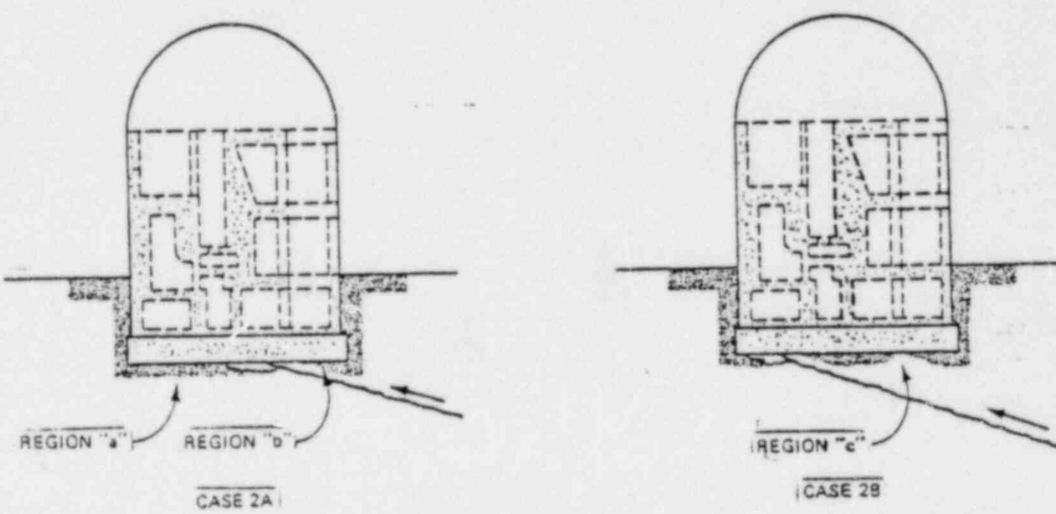


FIGURE A-16 CASE 2 – POTENTIAL FOR LOSS OF LOCAL FOUNDATION SOIL SUPPORT

In Case 2A, the surface rupture offset is assumed to intersect the base of the foundation to the right of the building's center of gravity (which is located about 5' to the left of the building geometrical centerline), such that a potential loss of support could theoretically be developed in the region of the center of the Reactor Building. In this case, the movement of the foundation soil in region "b" could cause the Reactor Building to tilt slightly. The Reactor Building would then be supported by the foundation soils in regions "a" and "b" and, in effect, would span as a beam between these two regions. Detailed investigations of this case showed that there could be the potential for cracking of the concrete and yielding of the reinforcing steel in the foundation base mat, exterior ring walls (e.g., wall "b" shown in Figure A-15), and floor slabs, all of which are exterior to the concrete core structure. These phenomena would not adversely affect the concrete core structure, since these structural components are not essential to the integrity of the concrete core structure. It was therefore concluded that the concrete core structure is adequate to withstand the loadings represented by Case 2A.

In Case 2B of Figure A-16, the surface rupture offset is assumed to intersect the foundation at a point to the left of the center of gravity, such that a potential loss of support could theoretically be developed in the region of the left-hand side of the Reactor Building. In this case, the movement of the foundation soil in region "c" could cause the Reactor Building to lift and tilt slightly, and primarily be supported by the foundation soil in region "c." This theoretically could produce an unsupported portion of the Reactor Building to the left of region "c."

A series of analyses were undertaken to demonstrate that the concrete core structure is adequate to withstand the loading represented by Case 2B. It was determined, based on soil pressure capacity analyses, that there are physical limits to the unsupported portion, beyond which the structure will tilt slightly and be supported in a manner analogous to Case 2A. It was also demonstrated that, for the unsupported cases conservatively assumed to exist, the capacity of the concrete core structure was above the induced loading. It was therefore concluded that the concrete core structure is adequate to withstand loadings represented by Case 2B.

In conclusion, even though (1) the surface rupture offset failure zone was shown to bypass the Reactor Building foundation, and (2) the probability of a surface rupture offset under the foundation at the site is low, a surface rupture offset was assumed to occur for the purposes of the structural investigations. The resulting loadings produced on the Reactor Building depended upon the point at which the surface rupture offset was assumed to intersect the building. Thorough investigations identified two basic loading cases which needed to be considered. Case 1 primarily involves the production of soil pressures on the ring wall between the basement and first floor levels. In Case 2, the surface rupture offset is assumed to intersect the Reactor Building concrete foundation mat. This case primarily involves the potential for loss of support of the structure by the foundation soil in certain regions. Detailed structural analyses demonstrated that the concrete core structure is adequate to withstand the loadings on the structure produced in both of these cases, which led to the conclusion that the concrete core structure is adequate to withstand the combined vibratory ground motion and surface rupture offset due to a postulated earthquake on the Verona fault.

3. Post-Offset Analyses

After it was demonstrated that the concrete core structure of the Reactor Building is adequate to meet the earthquake criteria described above, an analysis was performed, at the suggestion of the NRC, to demonstrate that the concrete core structure could resist aftershock ground motions. For these conditions, a conservative value of 0.75g maximum ground acceleration was selected for the evaluation purposes (although it would be more reasonable to use a smaller value for this purpose).

In these analyses, it was conservatively assumed that the main shock had damaged the portion of the building exterior to the concrete core structure to the extent that the rest of the structure, including all concrete slabs and walls exterior to the concrete core structure, had lost their capacity to further resist earthquake effects (even though the analyses described in

Sections A1 and A2 of this testimony demonstrated that this is not the case). It was also conservatively assumed that the concrete core structure had to resist the seismic forces induced by the weight of all structural components exterior to the concrete core structure.

The primary concern of the analyses was the stability (against overturning) of the concrete core structure, as well as stresses within the structure. The same basic structural analysis procedures were used as described in Section A.1 of this testimony. Computer models were used to simulate the physical characteristics of the structure described in the paragraph immediately above, and its behavior when subjected to earthquake motions.

Nonlinear dynamic time history analyses were carried out to investigate the effects of this ground motion on the safety-related portion of the structure. Nonlinearities due to potential partial uplift at the foundation slab-soil interface, as well as at the interface of the interior concrete structure and the foundation slab, were investigated. Maximum building rotation, uplift, horizontal displacement, and shears and moments were computed.

The results of the analyses demonstrated that the maximum building rotation would be only a fraction of a degree and that the vertical uplift of the base slab would be on the order of a few inches. When compared against the corresponding shears and moments obtained from the linear dynamic analysis for the design basis Calaveras event, it was observed that the values obtained from the nonlinear post-offset analyses were about 25 to 30 percent lower than those obtained from the linear elastic dynamic analyses. Thus the stresses corresponding to the post-offset analyses would be about 25 to 35 percent lower than those corresponding to the linear elastic dynamic analysis. The forces and corresponding stresses induced under post-offset conditions would thus be much less than those obtained from linear elastic analyses for pre-offset conditions.

It was therefore concluded that the concrete core structure of the Reactor Building would be stable and the stresses in the structure would be within acceptable limits; thus the concrete core structure is adequate to withstand aftershock ground motions.

4. Conclusions

The investigations described in this section of testimony demonstrated that the GETR Reactor Building concrete core structure, which supports the systems and components important to safety, is assured. The investigations were divided into three main parts consistent with the seismic criteria for the site:

a. Analyses for effects of vibratory ground motions caused by an earthquake on the Calaveras fault.

b. Analyses for effects of vibratory ground motions combined with a surface rupture offset caused by an earthquake on the Verona fault.

c. Analyses for effects of vibratory ground motions caused by an aftershock.

The following summarizes the conclusions resulting from these investigations:

a. <u>Evaluations for an Earthquake on the Calaveras Fault</u>

The analyses shows that the stresses in the concrete core structure (which surrounds the pool and the storage canal, and which also supports and protects the safety-related equipment and components necessary for safe shutdown) induced by an earthquake on the Calaveras fault were smaller than the capacity stresses. These stresses were based on the forces obtained from linear elastic dynamic analyses. Nonlinear analyses, including the nonlinearities due to potential uplift and sliding of the Reactor Building at the foundation slab-soil interface, were also performed. The forces obtained from these nonlinear analyses were smaller than those obtained from the linear analyses, which therefore confirmed the conservatism of the linear analyses. The nonlinear analyses also demonstrated that the Reactor Building is stable against potential uplift and sliding, and the soil pressures remain below the soil capacity. The analyses also showed that, although some cracking of slabs exterior to the safety-related concrete core structure may occur, the deformations of these slabs will be small, resulting only in minor and insignificant non-safety related cracking.

It was therefore concluded that integrity of the Reactor Building concrete core structure is assured for an earthquake on the Calaveras fault.

b. Evaluations for an Earthquake on the Verona Fault

Even though (1) the surface rupture offset failure zone was shown to bypass the Reactor Building foundation, and (2) the probability of a surface rupture offset under the foundation at the site is low, the surface rupture offset was assumed to occur for the purposes of the structural investigations. The resulting loadings produced on the Reactor Building depended upon the point at which the surface rupture offset was assumed to intersect the building. Thorough investigations identified two basic cases which needed to be considered. Case 1 primarily involved the production of soil pressures on the exterior ring wall between the basement and first floor levels. In Case 2, the surface rupture offset was assumed to intersect the Reactor Building concrete foundation mat. This case primarily involved the potential for loss of foundation soil support of the structure in certain regions. Detailed structural analyses demonstrated that the concrete core structure is adequate to withstand the loadings on the structure produced in both of these cases, which permitted the conclusion that the concrete core structure is adequate to withstand the combined vibratory ground motion and surface rupture offset due to a postulated earthquake on the Verona fault.

c. Post-Offset Analyses

After it was demonstrated that the concrete core structure of the Reactor Building is adequate to meet the earthquake criteria described above, an analysis was performed, at the suggestion of the NRC, to demonstrate that the concrete core structure could resist aftershock ground motions. It was determined that the concrete core structure of the Reactor Building would be stable and the stresses in the structure would be within acceptable limits. This led to the conclusion that the concrete core structure is adequate to withstand aftershock ground motions.

d. Summary for All Evaluations

Finally, for all cases, it was concluded that the detailed analyses performed for the vibratory ground motions and surface rupture offset demonstrate that the concrete core structure which surrounds the pool and storage canal will be adequate in the event that major earthquake motions and/or surface rupture occur at the GETR site. Thus, the structural and mechanical requirement to assure the integrity of the concrete core structure (which supports other systems and components important to safety) is met.

B. Integrity of Reactor Pressure Vessel

All safety-related piping and equipment, as modified, have been shown to be adequate to meet the specified seismic criteria. The piping and equipment described in this section of testimony are those items which are necessary to meet the functional requirement of keeping the fuel elements covered with water. This is accomplished by meeting the structural and mechanical requirement of keeping the fuel element containers intact. These containers consist of the reactor pressure vessel and associated piping and components, and the canal storage tanks and associated appurtenances. This section of the testimony addresses the reactor pressure vessel and associated piping and components. The canal storage tanks are discussed in Section C of the testimony.

The basic approach was to ensure that the fuel elements will remain covered by verifying the adequacy of or modifying any component which is required to maintain the water in the reactor pressure vessel and pool. Modifications were in the form of adding seismic restraints (i.e., braces) to the piping or component to restrict movement during seismic events.

The general physical phenomena considered in the evaluation process were as follows. As described in the previous section of this testimony, when seismic waves pass through the earth's crust, the ground upon which the building is supported is moved, and this movement varies rapidly with time. The consequence of this ground motion is a rapidly varying motion of the foundation of the building. This foundation motion, in turn, causes the walls, columns, and floors to move rapidly with time (refer back to Figure A-6). In general, this movement of the building is transmitted to the components in the building, such as piping and equipment, and has two main influences on these components.

First, the overall change of shape of the building may be imposed on the component. For example, two adjacent floors or stories in a building may move (displace) different amounts, as illustrated for the flexible three-story structure in Figure B-1.

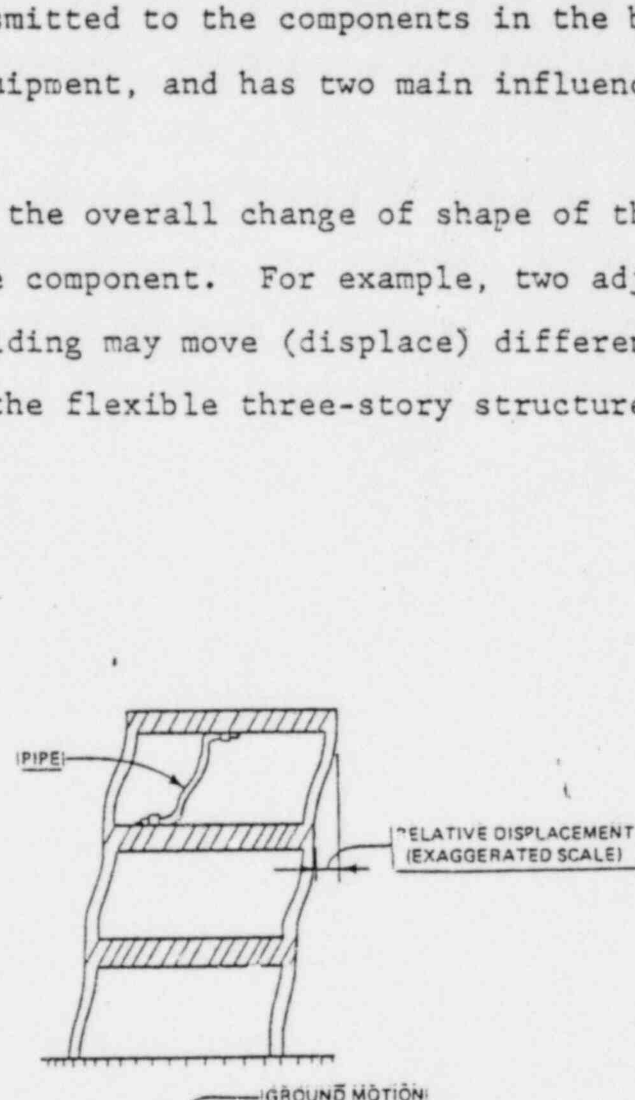


FIGURE B-1 ILLUSTRATION OF RELATIVE DISPLACEMENTS

The difference between these two amounts of displacement is termed the relative displacement, i.e., one floor displaces more (or less) relative to the other. If a component (e.g., a pipe) is routed between and connected to these two floors, it will be forced to displace the same as the floors, and thus one end of the pipe will be moved relative to the other (Figure B-1). This relative displacement of the two ends of the pipe can cause stresses in the pipe, and it must be shown that the component can withstand these stresses. Thus, all essential components in the building must be able to withstand "relative displacement effects." (Such effects are nearly always insignificant for stiff, massive buildings, such as the GETR concrete core structure.)

The second main influence of the movement of a building on a component supported in the building is caused by the rapid oscillating motion of the building. In regard to the component, this building motion is directly analogous to the ground motion; as the ground motion shakes the building, the building motion shakes the component in the building and causes it to vibrate (see Figure B-2).

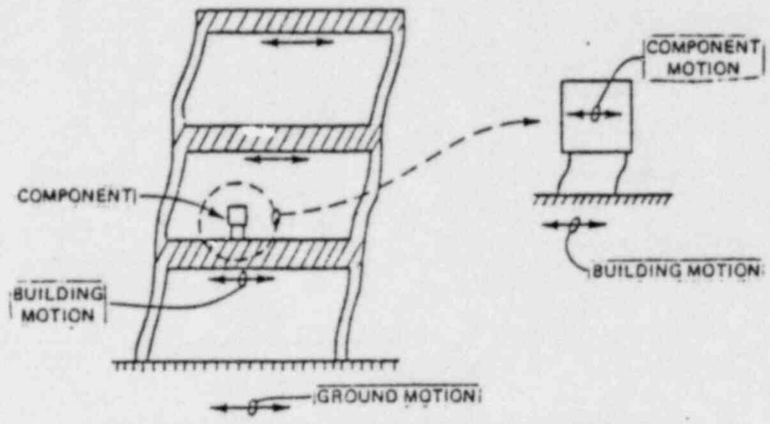


FIGURE B-2 ILLUSTRATION OF VIBRATIONAL EFFECTS

This shaking or vibrating can cause stresses in the component, and it must be shown that the component can withstand these stresses. Thus, all safety-related components in the building must be able to withstand these "vibrational effects."

As was the case with the earthquake ground motions described in Section A of this testimony, the behavior of the supported component subjected to building motions is represented by a response spectrum. This response spectrum, called an "in-structure response spectrum," or a "floor response spectrum," is a way of representing the behavior of a simple, idealized component (e.g., a tank) when subjected to earthquake-produced building motions. A qualitative description of the procedure used to develop an "in-structure response spectrum" is illustrated in Figure B-3.

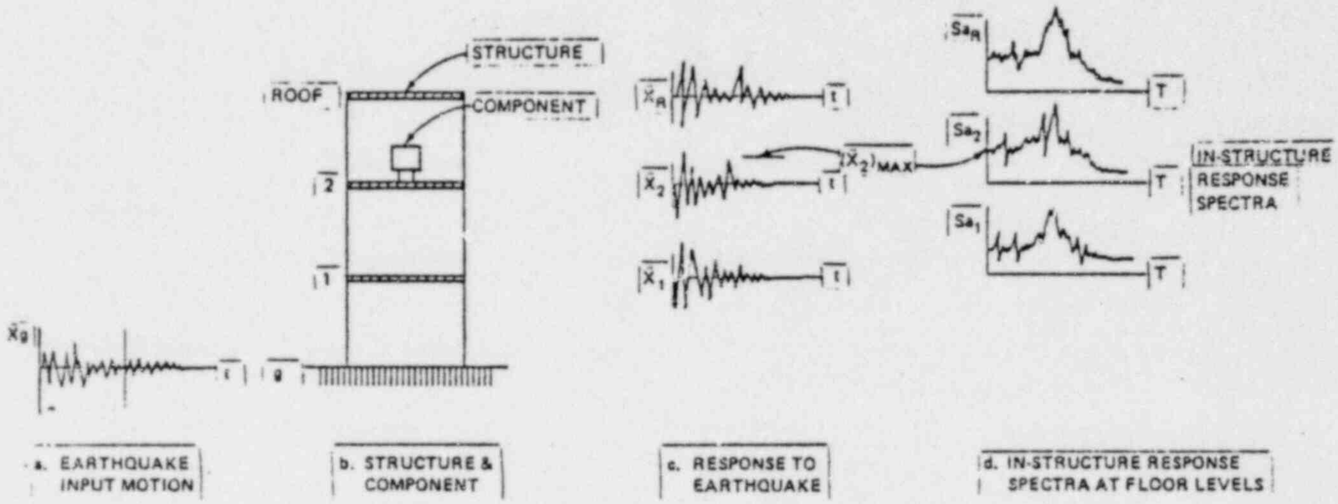


FIGURE B-3 STRUCTURE AND COMPONENT RESPONSE

The earthquake input motion, in the form of ground acceleration, is shown in Figure B-3(a). The structure (and component) subjected to this motion are shown in Figure B-3(b). The response of the structure, in the form of the acceleration records at each floor, are shown in Figure B-3(c). These records are obtained by the computer simulation and analysis procedures described in Section A of this testimony. These records are subsequently used to calculate in-structure response spectra, Figure B-3(d), using procedures also described in Section A. These in-structure spectra represent the behavior of the supported component. The dynamic analysis method used to actually calculate the behavior of the component subjected to the building motions is called the "response spectrum method." The above illustrative example has shown ground and building motions in one horizontal direction only. The actual analyses are carried out for both the horizontal and vertical directions. In addition, structural dynamic analysis techniques are used to extend the use of the response spectrum from simple to complex systems.

All of the safety-related components in the GETR Reactor Building were analyzed to ensure that they could withstand both relative displacement and vibrational effects.

Technically, this was accomplished as follows. As stated previously, linear elastic dynamic analyses of the GETR Reactor Building were performed utilizing a three-dimensional lumped mass analytical model of the soil-structure system. Time history modal superposition dynamic analyses were performed to determine time histories of accelerations at the various floor levels. These time history analyses were performed for the horizontal and the vertical directions. The time histories thus obtained were used to calculate the in-structure response spectra at the floor levels of the concrete structure. Envelope spectra were then generated from in-structure response spectra, taking into account the range of parameters that could influence the analytical results. The amplitudes and widths of the peaks of the in-structure response spectra were thus conservatively determined. These spectra were then used in the evaluations of the reactor pressure vessel and associated piping and components.

The relative displacement effects were examined for all safety-related components. The relative displacements of the concrete core structure, which supports the safety-related piping and equipment, were determined to be small and thus would not produce significant stresses in the piping and equipment.

Two commonly used engineering approaches were used to examine the safety-related piping and equipment; the first was used for geometrically complex systems, and the second was used for simpler systems. In the first approach, seismic dynamic analysis procedures which incorporated analyses for each item of piping or equipment were performed using detailed three-dimensional mathematical models to represent the important physical characteristics of the item. The dynamic analyses for each item were performed separately for each horizontal direction and the vertical direction. In accord with standard engineering practice, the response results for each of these three analyses were then combined by the square root of the sum of the squares (SRSS) method. In the second approach, the small items of piping and equipment were analyzed by simplified dynamic analysis methods, wherein a static load equal to a multiple (1.5) of the peak of the floor response spectrum was used. This is also in accordance with standard engineering practice.

Each item of piping or equipment was evaluated for seismic effects acting simultaneously with the appropriate normal operational loads, such as dead load (the weight of the item), temperature, and pressure. Conservative allowable stresses were selected based upon values from codes and handbooks which are applicable to the construction materials used for the GETR facility. The seismic adequacy of piping and equipment is reviewed in Reference 60. Details of individual analyses are given in the technical reports referenced in each section below.

The major elements evaluated included the:

1. Reactor Pressure Vessel and Primary Cooling System
2. Primary Heat Exchanger
3. Reactor Pressure Vessel and Pool Drain Lines and Poison Injection Line
4. Safety-Related Valves
5. Pool Heat Exchanger
6. Control Rod and Incore Shuttle Assemblies

The evaluations of each of these are described in the following testimony.

1. <u>Reactor Pressure Vessel and Primary Cooling System</u>

An analysis of the reactor pressure vessel (RPV) and primary cooling system piping was performed for normal plant operating conditions and earthquake conditions. The primary cooling system circulates water between the RPV, which is located in the RPV pool near the center of the Reactor Building, and the primary heat exchanger HE-101. The water circulated by the primary cooling system maintains the operating temperature of the RPV at normal operating levels. The primary cooling system is shown schematically in Figure B-4.

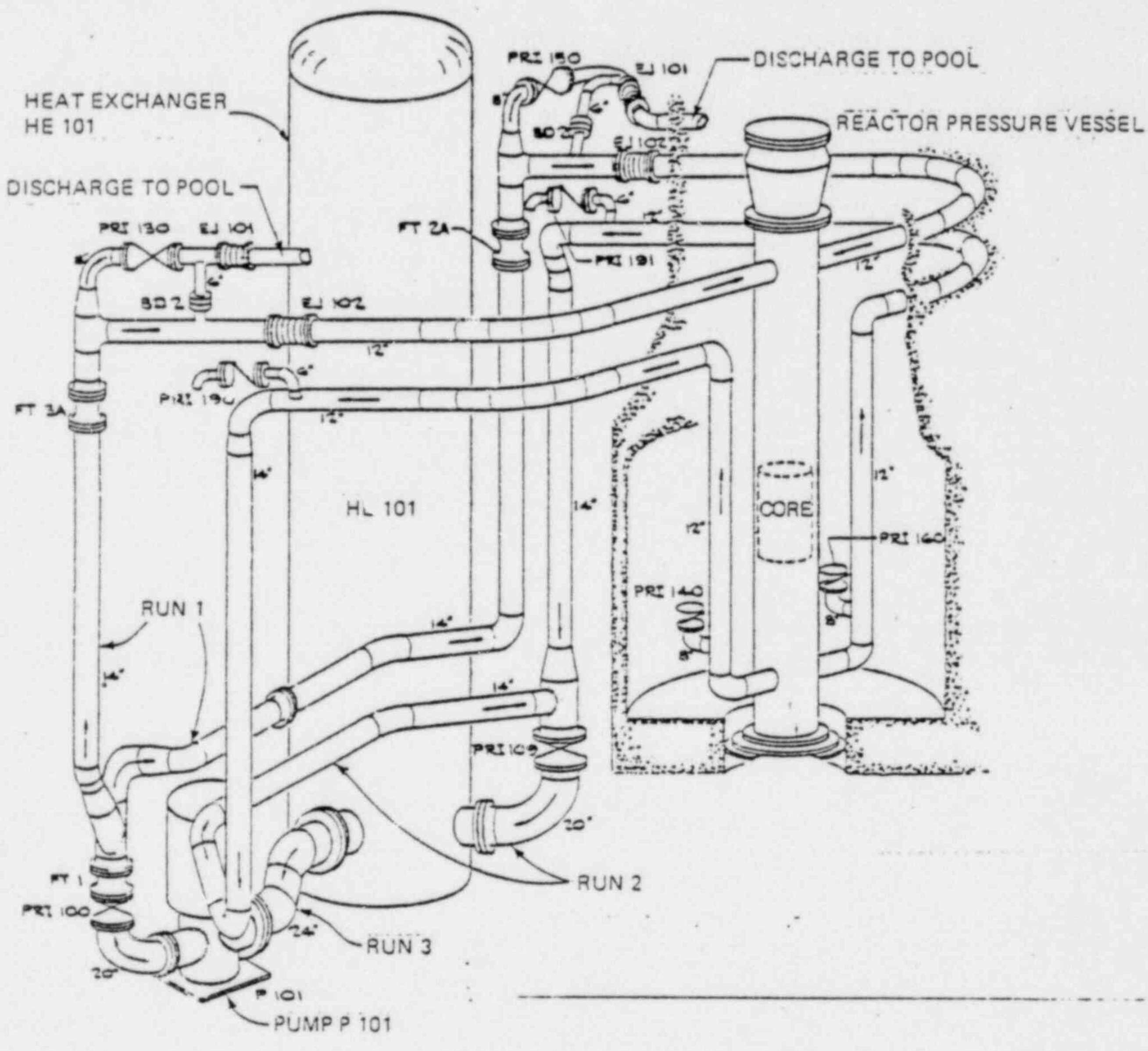


FIGURE B-4 SCHEMATIC VIEW OF PRIMARY COOLING SYSTEM

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Flow to the RPV is through Run 1 of the primary system, which also includes the emergency cooling system piping and valves. The cooling water is returned to HE-101 through Run 2 of the primary system. Run 3 is the short pipe from HE-101 to the primary pump P-101.

The primary cooling system consists of piping and fittings made of 6061-T6 aluminum. Run 1 exists from the primary pump (P-101) as a 20-inch diameter pipe, branches into two 14-inch diameter pipes, and is then reduced to 12 inches diameter before connecting to the RPV. Run 2 returns water from the RPV to HE-101. This line consists of two 12-inch diameter pipes; one increases to 14 inches and then both merge into one 20-inch line before connecting to HE-101. Run 3 is a 24-inch line.

Shown in Figure B-5 are the restraints that either have been or are in the process of being added to the primary system to increase its strength to a level such that it is capable of resisting the postulated seismic motions.

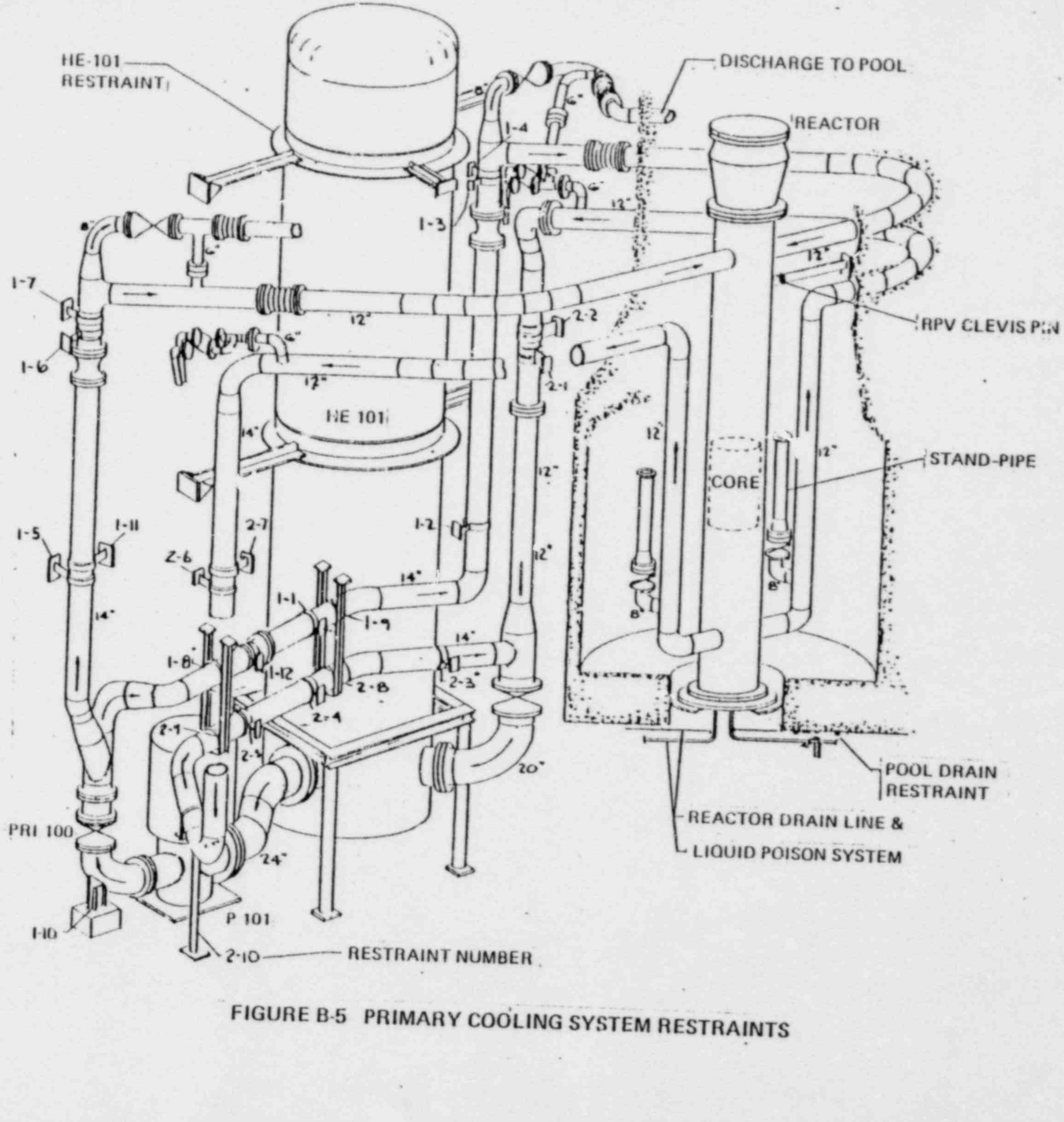

FIGURE B-5 PRIMARY COOLING SYSTEM RESTRAINTS

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Two modifications were made to the reactor pressure vessel itself. The first was to strengthen the support near the top of the reactor pressure vessel. The top of the RPV is laterally supported by three struts which are anchored to the pool wall (Figure B-6).

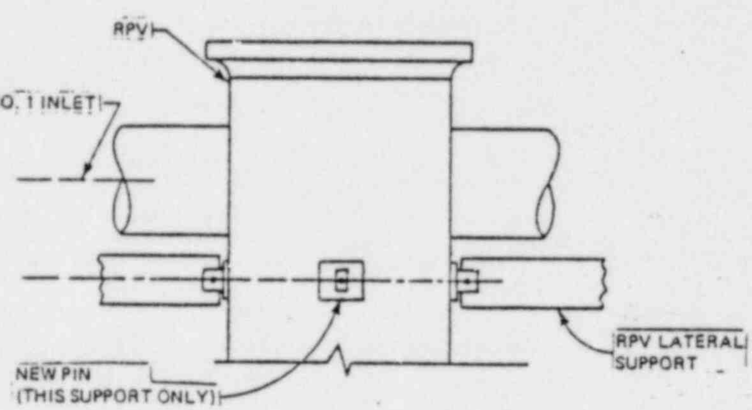

FIGURE B-6 ELEVATION VIEW OF TOP OF RPV

After evaluating the analytical results, it was recommended that one of the 0.5-inch diameter stainless steel pins which connect the RPV lug to the strut be replaced by a pin with a material having a yield stress of greater than 63 ksi. This modification has been made. The second modification was the addition of standpipes above each of the check valves (PRI-140 and PRI-160) to assure that the core will remain covered with water. The standpipes are shown in Figure B-5. A complete description of the seismic evaluation of the RPV and primary cooling system is given in Reference 22.

Two basic mathematical models were used in the evaluation. The first was of primary system Run 1 and the RPV and internals; and the second run was of primary system Run 2. As an example, the mathematical model of Run 1 and the RPV and internals are shown in Figures B-7 and B-8, respectively.

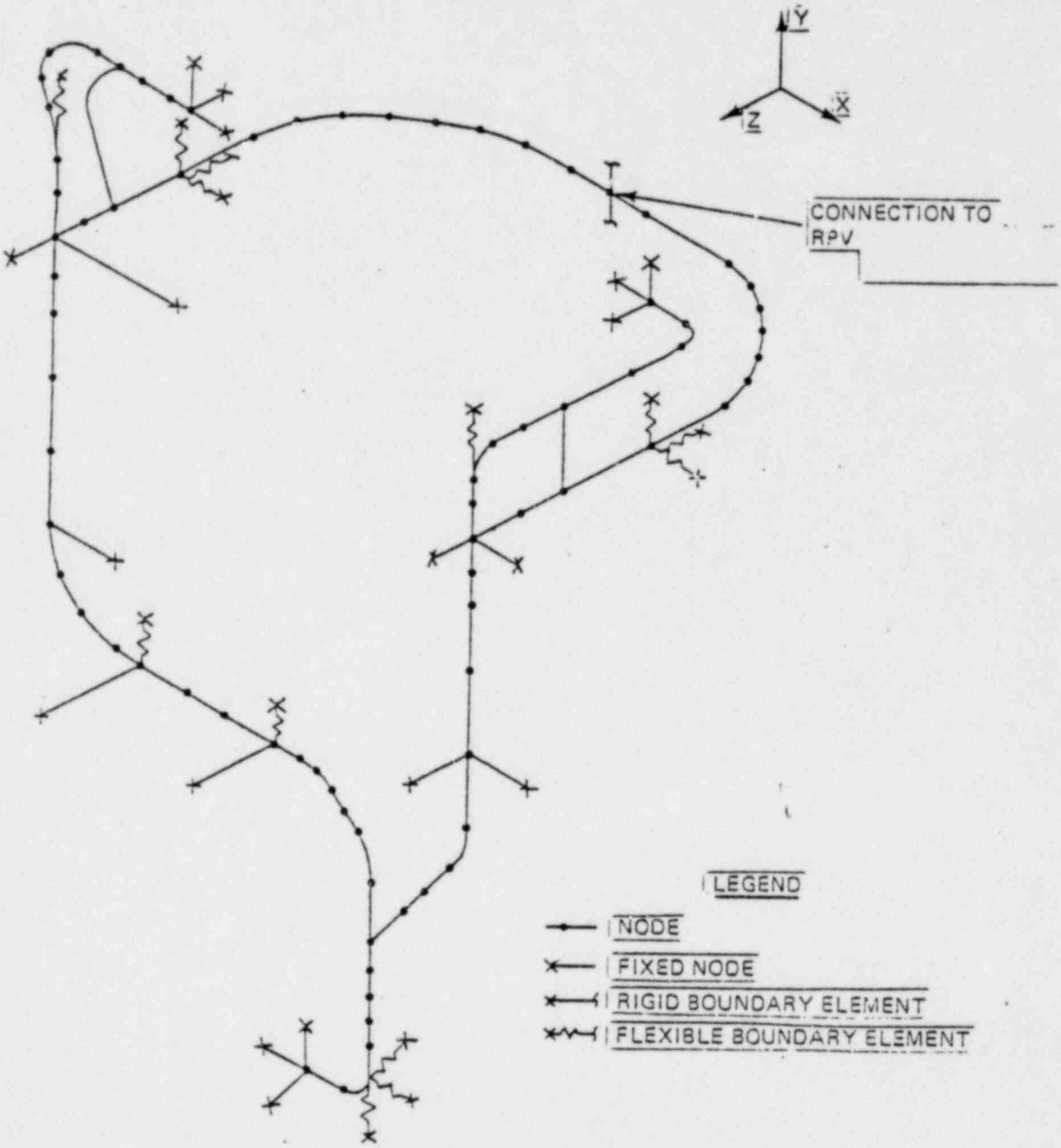

FIGURE B-7 PRIMARY COOLING SYSTEM RUN 1 MATHEMATICAL MODEL

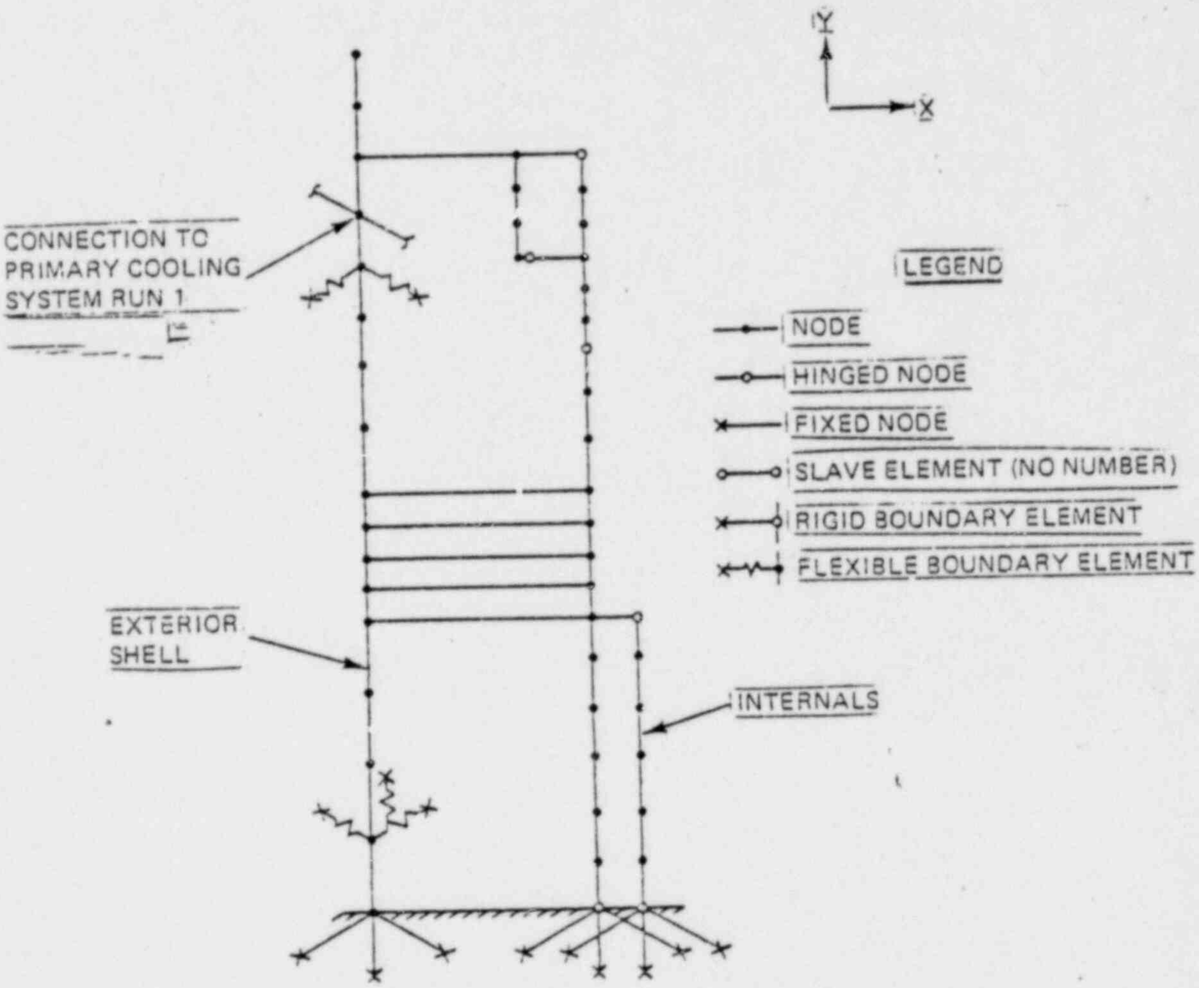

FIGURE B-8 REACTOR PRESSURE VESSEL MATHEMATICAL MODEL

Conventional dynamic analysis procedures were used to develop these models. The weight of the pipe or RPV shell was assumed to be concentrated at the node points as shown. Flexible pipe elements were used between nodal points, except for valve components which were represented as being rigid. Boundary elements were used at the supports to obtain reaction forces and moments. Axial, shear, flexural, and torsional deformations were included. Static analyses were conducted to obtain stresses due to the pressure, temperature, and sustained vertical loads. Dynamic analyses using the standard response spectrum method were performed to obtain the stresses due to the earthquake load. Internal forces from all loading conditions were obtained, and stresses were then calculated from the member forces resulting from pressure, temperature, earthquake, and sustained vertical loads. These stresses were combined and compared to the capacity stress values. Forces in the piping restraints were obtained by the same procedure and computed values compared with the capacities. In addition, stresses in the RPV shell and internal components were computed using the forces and moments from the computer analysis output.

It was determined that the stresses in the piping, piping restraints, RPV lateral braces, RPV shell, internals, and standpipes were within acceptable limits. It was thus concluded that the primary cooling system, comprised of the piping, piping restraints, and the RPV (and its internals), meets the structural and mechanical requirement of keeping the reactor pressure vessel fuel element containers intact.

2. Primary Heat Exchanger

As described above, the primary heat exchanger (HE-101) is attached to the primary cooling system, which is, in turn, connected to the reactor pressure vessel. Figures B-4 and B-5 above show schematic views of HE-101. The heat exchanger is not a safety-related component; however, new restraints were installed to prevent potential damage to the RPV caused by movement of HE-101. The primary heat exchanger structural supports were strengthened to ensure that HE-101 does not move. A detailed description of the seismic evaluation of HE-101 is presented in Reference 22.

The existing HE-101 is a two-pass U-type shell and tube unit. The shell is constructed of 1/2-inch thick aluminum plate. This heat exchanger will be replaced in the near future by a new HE-101 with stainless steel shell. The analyses were performed for the existing HE-101; appropriate analyses and modifications will be performed as required to ensure the integrity of new HE-101. Virtually identical supports will be used, which will be as strong as or stronger than those used for the existing HE-101.

The seismic restraint modifications for HE-101 are shown in Figure B-9.

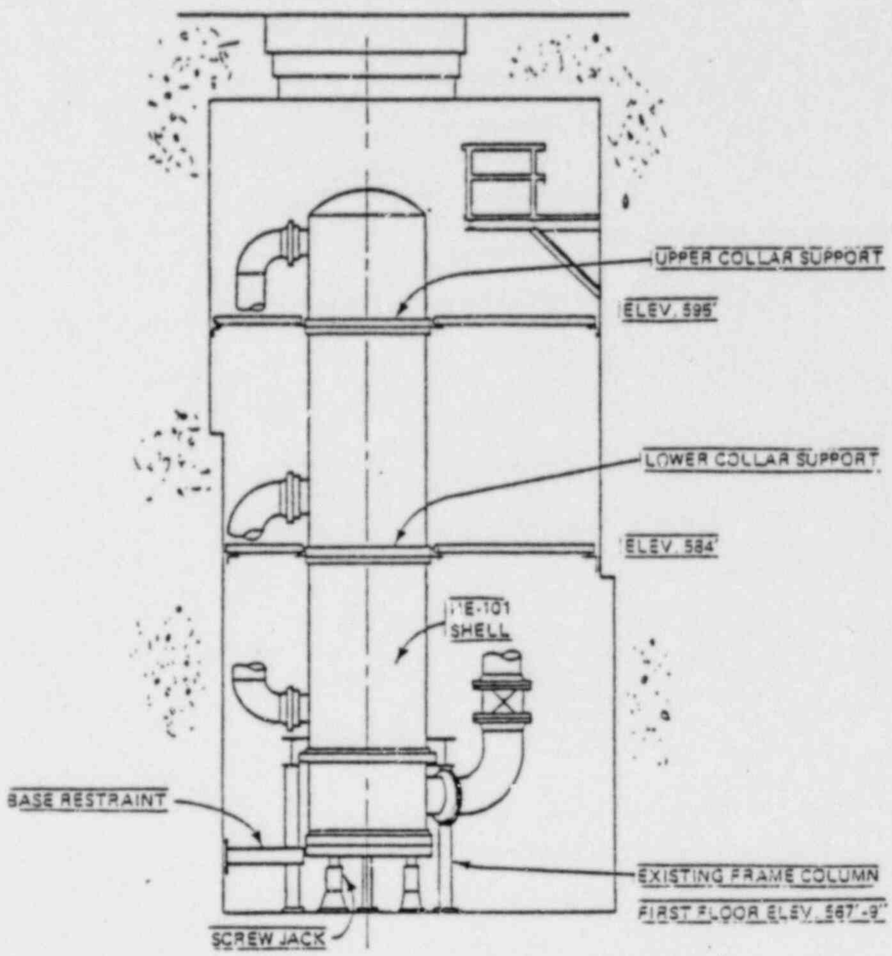

FIGURE 3-9 HE101 SEISMIC RESTRAINTS

These consist of two new support collars near the top and mid-height of the heat exchanger, which are braced against the surrounding walls to prevent lateral movement of the shell. In addition, the bottom flange of the heat exchanger channel will be braced directly to the adjacent walls by the base restraint shown in Figure B-9. Four screw jacks will be located under the bottom flange to resist the downward forces due to seismic motions.

The two new support collars are located around the HE-101 shell approximately at elevations 595 and 584 feet. The collars are braced with struts to the surrounding walls to prevent lateral movement of the shell. There are four struts at each support collar level. They act in compression, except for the two struts located on the lower collar which are attached to the north wall. These two struts are also designed to carry tensile forces with the use of embedded rock bolts.

The bottom flange of the heat exchanger channel is braced to the north and west walls. Anchorage to the west wall consists of rock bolts embedded in the concrete wall. Anchorage to the north wall consists of two-inch diameter rods placed in holes drilled through the six-foot, six-inch wall. Four screw jacks, which resist downward forces, are located under the bottom channel flange.

Analytical models of the heat exchanger and its restraints, as well as the shell and support collars, were developed using the same procedures as described above for the RPV and primary cooling system. The maximum stresses in the shell support collar and restraints were obtained. All values were found to be within allowable limits. It was therefore concluded that seismic movement of heat exchanger HE-101 is prevented by the new restraints, and that HE-101 would not cause any damage to the safety-related RPV.

3. Reactor Pressure Vessel and Pool Drain Lines and Poison Injection Line

Evaluations of the piping and new seismic restraints for the pool drain line, reactor pressure vessel drain line, and the poison injection line were performed. All three lines are located in the Reactor Building at elevations beneath the RPV and pool (see Figure B-5). These piping systems consist of small diameter pipes or tubing, on the order of one-half, one-, and two-inch diameter. Materials are either carbon or stainless steel. These lines were examined to verify that there would be no failure of the piping which could, in turn, cause water to drain from the RPV or pool during or following a seismic event. It was determined that seismic restraints were required to accomplish this. A complete description of the seismic evaluation of these lines is presented in Reference 22.

Although these lines are located beneath the RPV and pool near the first floor level, the analyses were conservatively based on in-structure response spectra at the third floor, which are higher in amplitude than the spectra at the first floor. The analyses were performed to obtain the forces and stresses in each of the lines and their restraints. Maximum permissible restraint spacings were obtained, as were the forces in the restraints themselves.

The analyses were performed using either a conservative simplified procedure, where the load on the piping system was taken as 1.5 times the peak spectral acceleration, or a more detailed dynamic analysis approach. Detailed dynamic analyses were performed for selected portions of the piping system by conventional response spectrum methods. The dynamic analyses for these components primarily focused on those systems which contained valves. For these analyses, the piping and valve components were assumed to act as single degree of freedom systems, which was a reasonable assumption since the valves are relatively heavy and stiff when compared to the adjacent piping.

The new seismic restraints were found to be adequate to maintain the integrity of the small piping systems during and following the maximum postulated seismic events. The computed pipe stresses were less than allowable values, and the restraint anchorages had adequate capacities to withstand the seismic loadings. It was therefore concluded that the pool drain line, reactor pressure vessel drain line, and the poison injection line would remain intact and would prevent water from draining from the RPV or pool during or following a design basis seismic event.

4. Safety-Related Valves

All valves necessary for the operation of the safety-related systems were seismically qualified to verify that they would operate as required. Seismic qualification tests and analyses of 16 types of valves from the safety-related piping systems were performed. These 16 types of valves represented a total of 81 valves which are part of the safety-related systems. The valves were qualified by vibration testing and/or analysis. Testing was used as the primary basis for the seismic qualification, while analyses were performed for some of the valves to provide additional assurance that the valves can resist the design basis seismic event. The qualification tests included simulation of the internal pressure environment and functional operation which would be required during an earthquake. Prior to testing each valve, a test plan and detailed test procedure were developed. Testing equipment was fabricated so that each valve could be mounted on a shake table and pressurized similar to the conditions anticipated in the piping system before and after the postulated seismic event. The details of the valve evaluation program are given in Reference 22.

A frequency test of each valve was first performed to determine dynamic properties and to identify natural frequencies, if any, below 33 Hz. (If a valve has frequencies below 33 Hz, then the earthquake motions that are input to the valve by the piping may be amplified due to the dynamic characteristics of the valve -- just as ground motions may be amplified by a building.) These tests were performed for three orientations of each valve. It was found that valves with operators were the only valves with natural frequencies below 33 Hz.

The next step was to perform a proof test of each valve to determine whether the valve would function properly during the design basis seismic event. Each valve was pressurized similar to its operating condition, and the required operations such as opening, closing, and maintaining pressure integrity were tested. The shake table motion used to proof test each safety-related valve was developed to conservatively represent the vibratory motion that would be felt by each valve during the design basis seismic event. By controlling the amplitude and frequency content in the shake table motion, the accelerations produced test response spectra which enveloped the corresponding required response spectra values that applied at the location in the Reactor Building where the piping system containing the valve was anchored.

Each valve was systematically tested and qualified. It was shown that each valve meets the structural and mechanical requirements for the safety-related piping and equipment.

5. Heat Exchanger HE-102

Lateral restraints were installed to restrain the pool heat exchanger (HE-102). The pool heat exchanger provides cooling water to the pool which surrounds the reactor pressure vessel. HE-102 is part of the secondary cooling system which circulates water to and from the cooling tower. Although the pool heat exchanger is not a safety-related component, the lateral restraints were designed to prevent potential damage to the primary piping system, which could in turn induce stresses in the RPV. The purpose of adding the restraints to the pool heat exchanger was to prevent it from falling onto the adjacent primary piping system during the design basis seismic event.

The seismic restraints consist of stainless steel cables wrapped around the heat exchanger shell at two separate vertical locations (Figure B-10).

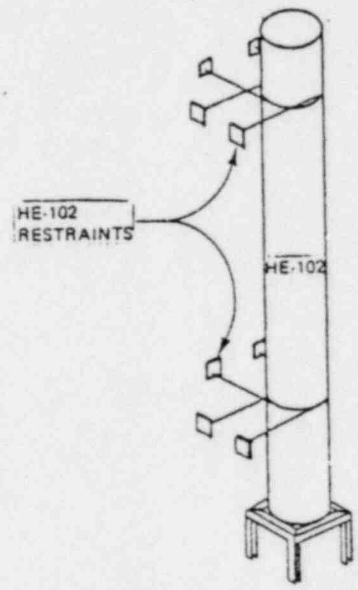

FIGURE B-10 HEAT EXCHANGER HE-102

The cables are attached to two turnbuckles, connected to assemblies which are anchored to the Reactor Building concrete walls by virtue of steel wall plates and concrete anchor bolts. The analyses (Reference 22) demonstrated that the seismic restraint system complies with the seismic criteria and has adequate capacity to prevent the pool heat exchanger from falling onto the primary cooling system.

6. Control Rod and Incore Shuttle Assemblies

Seismic evaluations were performed for the control rod and incore shuttle drive assemblies for GETR. There are six control rod and one incore shuttle drives located in the Reactor Building beneath the reactor pressure vessel. Both the control rod drive and incore shuttle drive assemblies extend from beneath the RPV, through the subpile room, into the access gallery room at the basement level. The control rod drives are shown in Figure B-11; the incore shuttle drive is similar.

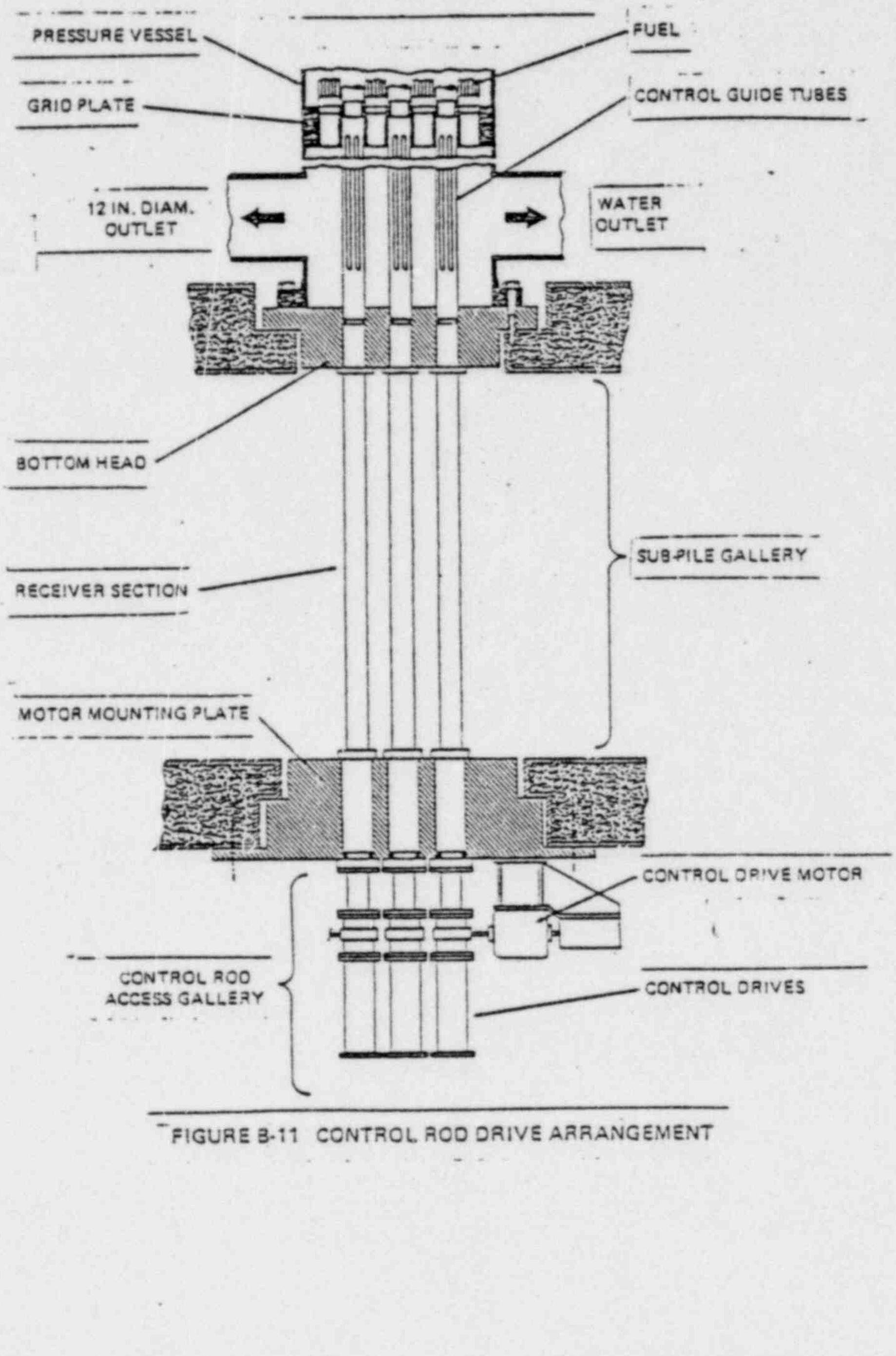

FIGURE B-11 CONTROL ROD DRIVE ARRANGEMENT

POOR ORIGINAL

These assemblies were evaluated to verify that they will maintain their integrity during the design basis seismic event and thus prevent any water from draining from the RPV. Although these assemblies are located beneath the RPV, the seismic analyses were conservatively based on the in-structure response spectra at the third floor, which are higher in amplitude than the spectra at the first floor. Analyses were performed to obtain the forces and stresses in each of the assemblies, and the sustained vertical load plus the internal pressure were used to compute to the normal operating stresses, which were combined with the seismic stresses. Conservative assumptions were made in computing the frequencies, moments, and stresses in the assemblies.

It was determined that the computed stresses are well below allowable values. It was thus demonstrated that the control rod and incore shuttle drive assemblies will maintain their structural integrity during and following the postulated seismic events.

7. Conclusions

In conclusion, it was demonstrated by analysis, modification and analysis, or testing that all safety-related piping and equipment are adequate to resist the motions induced by the postulated seismic events for the GETR site. These components, therefore, meet the structural and mechanical requirement that the fuel element containers remain intact, and the functional requirement that the fuel elements in the reactor pressure vessel remain covered with water.

C. Third Floor Missile Impact System and New Fuel Storage System

In what follows, the analysis of the third floor missile impact system and new fuel storage system are addressed to show that both will satisfy the mechanical and structural requirements for the design basis seismic event at the GETR.

1. The Third Floor Missile Impact System

The Third Floor Missile Impact System provides protection of safety systems, critical components and structures located on the third floor of General Electric Test Reactor Building from possible damage due to postulated collapse of the Polar Crane Trolley Assembly. The impact system consists primarily of structural bents topped with honeycomb blocks. Any possible collapse configuration of the polar crane assembly is arrested by this impact system, with the honeycomb pads minimizing the impact loading on the bents. Design and analysis of the impact system is effected by straight-forward analytical methods which further enhances the system reliability. In addition, the impact system stands alone, and so is not affected by the behavior of the reactor containment shell. As a result, the impact system constitutes a most reliable, independent protection system for the third floor safety systems and critical components.

a. <u>Description of Impact System</u>

The Third Floor Missile Impact System consists of a series of structural bents augmented by the elevator structural assembly. These structural bents along with the elevator structure are integrated into a barrier system which protects the safety systems and critical components located on the third floor of the GETR building from impact due to a postulated collapse of the polar crane bridge and trolley assemblies. The locations of these bents were arrived at with the aid of accurately scaled models of the third floor area and the polar crane assembly to assure the bents provide a complete circle of protection for the third floor area (see Figures C-1 and C-2).

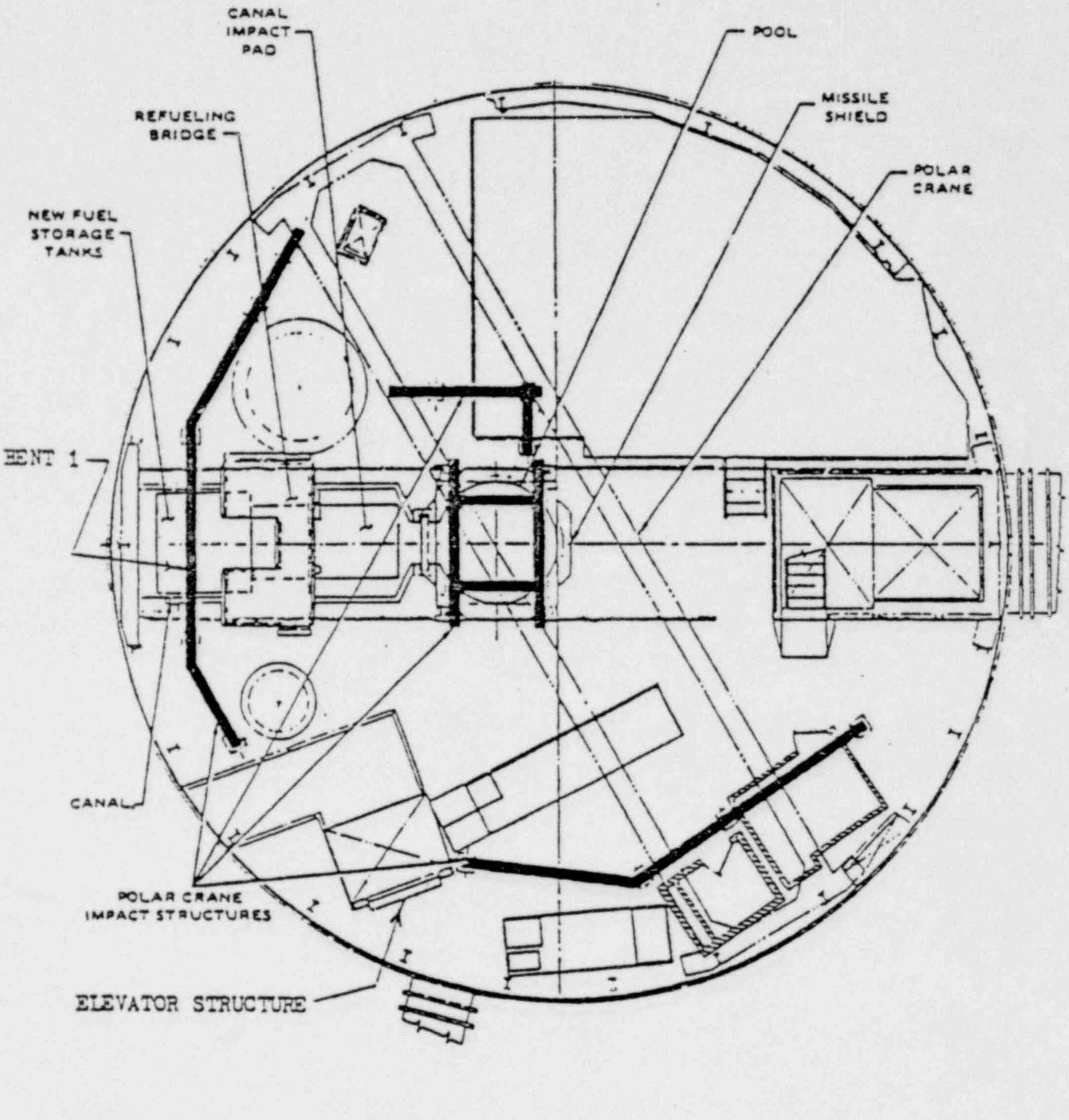

Figure C-1. Third Floor Plan of Reactor Building
Showing Polar Crane Impact Structure

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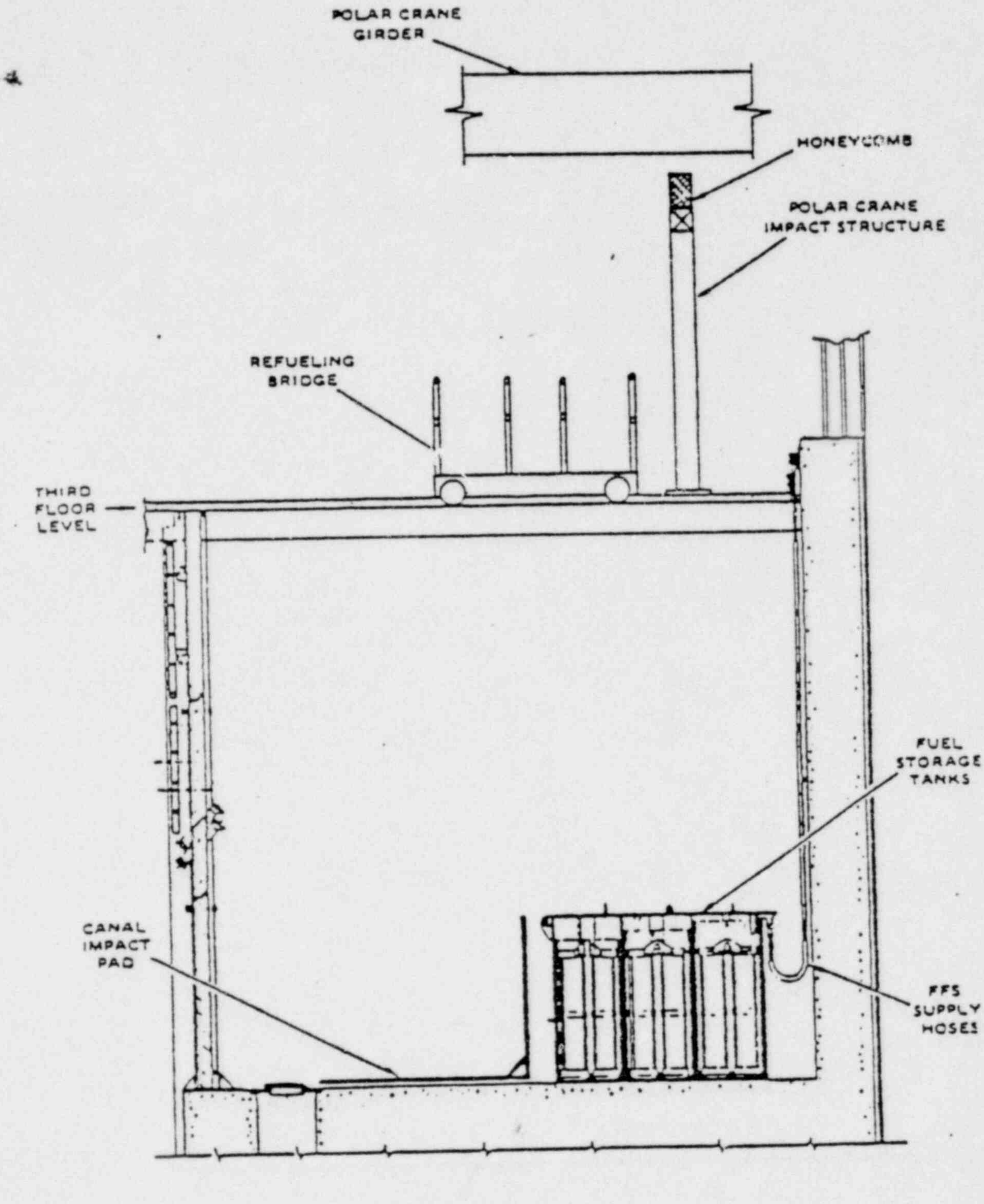
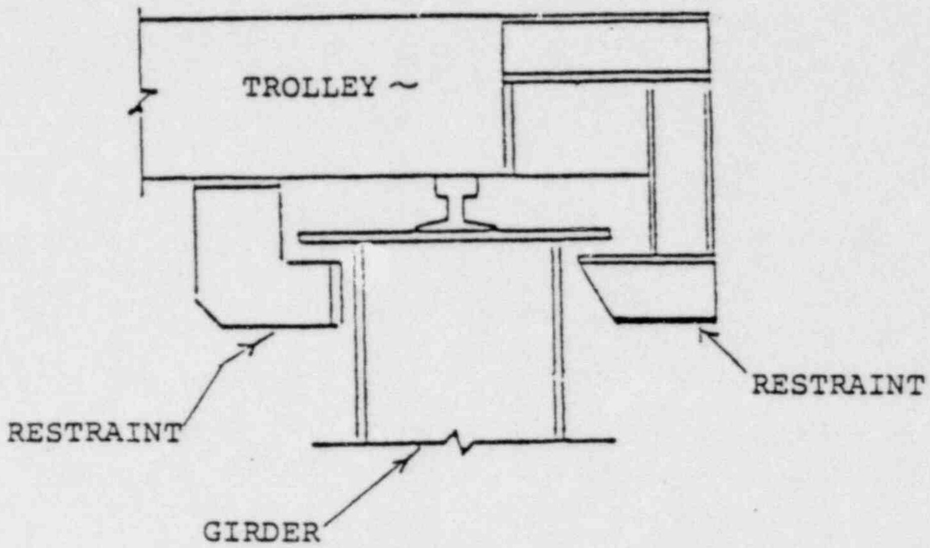


Figure C-2. Elevation of Canal Area Showing Polar Crane Impact Structure with Honeycomb Bed

The impact protection bents are all fabricated from 10 inch square structural tubing with 1/2 inch wall thickness. The bents are anchored to the concrete floor by means of square base plates which are welded to the bents. These base plates are, in turn, attached to the concrete floor slab by means of four anchor bolts; one at each corner of the base plate, and embedded over 20 inches into the concrete floor slab. An 18 inch deep bed of honeycomb is installed atop all of the bent girders to mitigate the postulated impact of polar crane assembly. Clearance between the bottom of the polar crane assembly and the top of the honeycomb beds is limited to 6 inches (see Figure C-2).

In order to prevent the crane trolley from becoming a missile, a restraint system designed to maintain the integrity of the trolley and bridge assembly was attached to the trolley structure. This restraint system also utilizes honeycomb pads to limit impact loads and constitutes an integral part of the protection system (see Figure C-3).



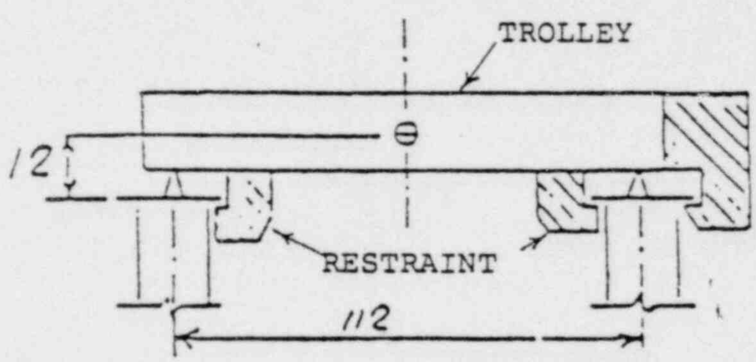


Figure C-3. Sketches of Trolley Restraint System

b. Design Criteria For Impact System

Two design loading conditions were applied to the bent structural analyses. The first loading condition consists of simultaneous loading due to impact forces and peak seismic acceleration of the free-standing bent, while the second loading condition consists of peak seismic acceleration of the coupled bent and collapsed polar crane assembly.

The first loading condition represents the maximum possible loading of the impact system at the time of postulated collapse of the polar crane assembly. In this loading condition the polar crane assembly impacts the protective bents at the instant the bents are experiencing the design basis seismic loading. The second loading condition represents the maximum possible loading of the impact system after a postulated collapse of the polar crane assembly. In this loading condition the bents experience the design basis seismic loading while supporting the collapsed polar crane assembly. This second loading condition envelopes any possible after-shock loadings.

c. Methods of Analysis and Results for Impact System

The methods and results of the analysis are fully reported in reference 22. The required depth of honeycomb atop the impact bents was determined by an energy balance wherein the loss in potential energy due to the postulated collapse of the polar crane assembly is equated to the inelastic strain energy developed in the honeycomb material (see Figure C-4).

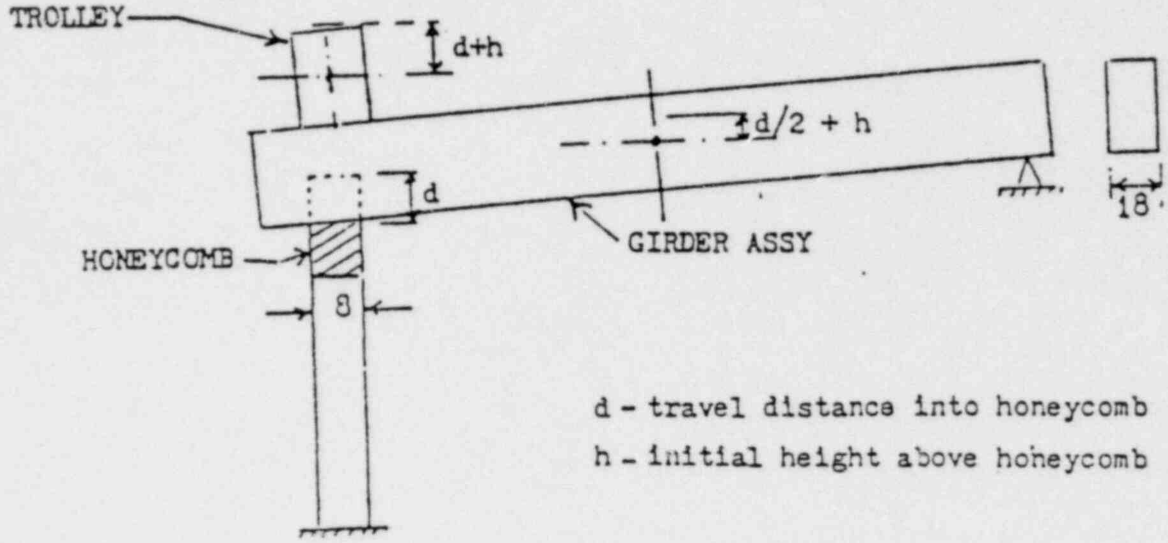


Figure C-4. Configuration of Girder Impact on Honeycomb Bed

In addition, possible amplification of the honeycomb crush loads due to vibratory motion of the honeycomb was conservatively accounted for by applying a factor of 2 to the loads required for an energy balance.

The method of maximum modal response was applied for evaluation of effects of seismic loadings on the bents. In this method, all vibratory modes under 33 Hz are first determined. The maximum acceleration in each of these modes is then determined from the appropriate response spectra, and these accelerations are applied to determine the maximum stresses in each mode. The effects of the different modes are then combined by an SRSS combination. In cases where impact loadings are also postulated, the bent stresses due to impact are directly added to SRSS combination of stresses. In all cases the stresses were within allowable limits.

The maximum anchor bolt loading is arrived at by transforming the orthogonal moments at the bent base as given by the finite element analysis into a second set of orthogonal moments in the directions of the base plate diagonals. In this transformed configuration each orthogonal moment acts on only one anchor bolt. The maximum bolt load is then determined by assuming that only one bolt is in tension and that the maximum bearing pressure on the base plate equals the concrete compressive stress. In all cases, the maximum bolt loads were within allowable values.

d. Conclusions

The third floor missile impact system constitutes a simple and reliable, independent system for protection of the safety systems and critical components located on the third floor of the reactor building. The analyses show that the impact system is capable of functioning successfully under loadings associated with design basis seismic accelerations of the reactor building.

2. New Fuel Storage System

The New Fuel Storage System is designed to maintain structural integrity and thereby a fluid environment for spent fuel under simultaneous loadings due to normal operating conditions and design basis seismic accelerations. The system consists of three separate inner fuel tanks within one outer tank. Incorporated in this system configuration is a redundancy which greatly enhances the system reliability. Both the inner and outer tanks are designed to maintain structural integrity and fluid retaining boundaries under the postulated loadings. Further, the system is designed so that both the inner and outer tanks can function individually as well as in combination under the design loadings. From the analyses described in this section, it can be concluded that the New Fuel Storage System is capable of performing the design function of maintaining a fluid environment for the spent fuel under loading associated with the maximum seismic event postulated for the reactor building.

a. Description of New Fuel Storage System

The New Fuel Storage System consists of three separate inner fuel tanks contained within one outer tank. The outer tank rests without restraints on the canal floor. The inner tanks, constructed from one-quarter inch 304 stainless steel plate, are the primary structures for insuring that water always surrounds the irradiated fuel. These inner tanks are constructed with divider plates to maintain fuel rack separation. The outer tank is fabricated from one-half inch 304 stainless steel plate, and is designed to provide both protection for the three inner tanks and a secondary means for water retention around the fuel (see Figures C-5 and C-6).

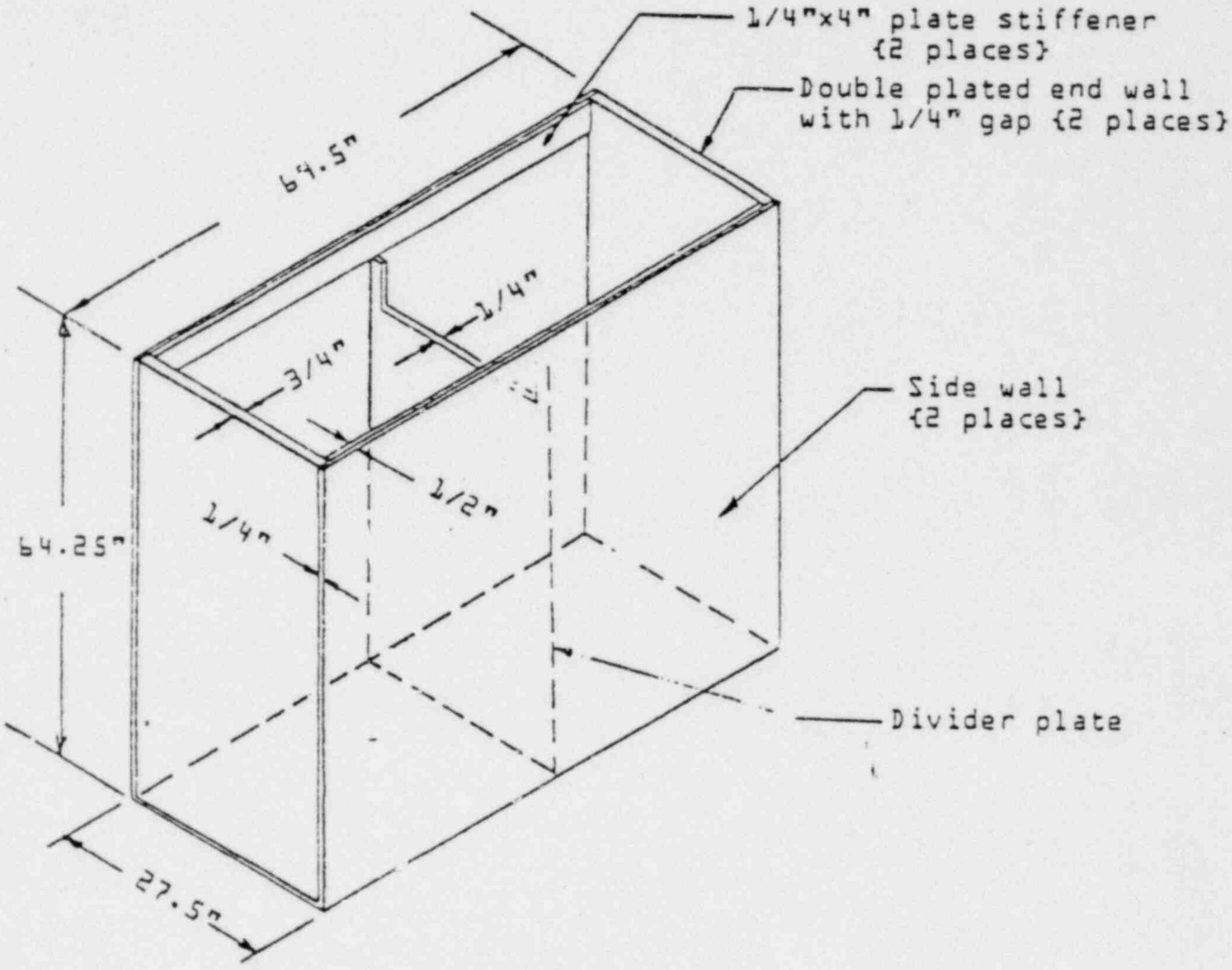


Figure C-5. Inner Fuel Storage Tank

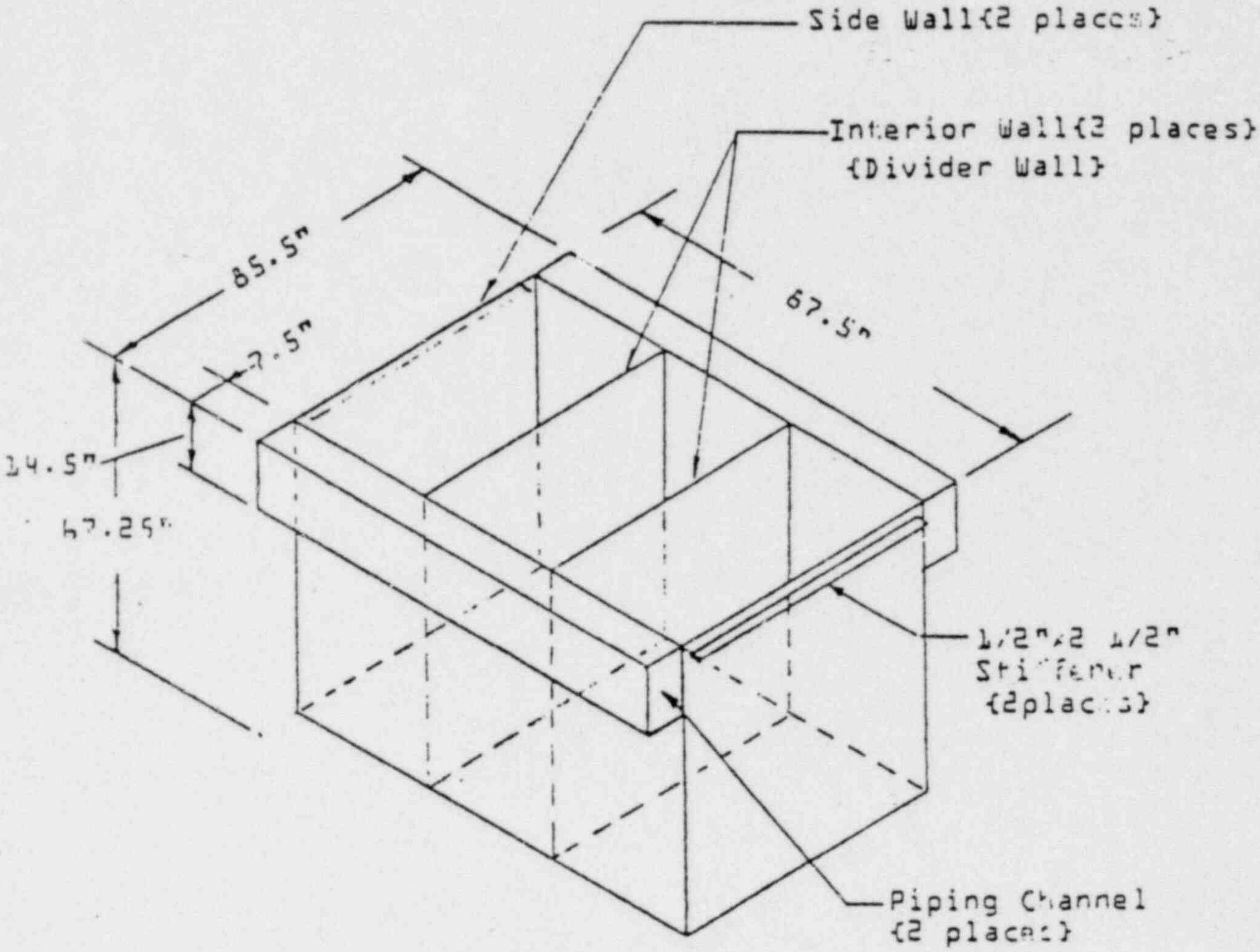


Figure C-6. Outer Fuel Storage Tank

b. <u>Design Criteria for New Fuel Storage System</u>

The fuel storage system is designed to maintain structural integrity under the simultaneous loadings due to normal operating conditions and design basis seismic accelerations. However, the normal operating loadings are negligible by comparison with the postulated seismic loadings. For purposes of analysis, both the inner and outer tanks are considered rigidly fixed at the base points in order to produce maximum lateral loading in the tanks. In all analyses the tanks are considered filled with a full complement of water and fuel while the canal is assumed dry. This combination of a full tank in a dry pool produces the maximum stresses in the tank walls.

c. Method of Analysis and Results for New Fuel Storage System

The methods and results of the analyses are fully reported in reference 22. In the structural evaluation of the inner fuel tanks, both the side walls and the divider plates were subjected to a detailed, finite element, maximum modal analysis. The seismic accelerations applied in these analyses were taken from the response spectra specified for the fuel pool area. Spectra values corresponding to 3% damping were applied in the modal analyses. Evaluation of the results of the detailed analyses was performed in accordance with Section III, Appendix F of the ASME Boiler and Pressure Vessel Code, and all stresses were within allowable limits.

In the structural evaluation of the outer fuel storage tank a detailed, finite element, modal analysis was performed on one of the walls which divide the tank into three equal compartments. These divider walls are the most flexible and heavily loaded component of the outer tank and are therefore the choice for detailed analysis. The loading applied to the divider walls consists of the impact due to rocking of the inner tanks as well as the inertia forces due to the mass of the inner tank. Evaluation of the results of the detailed analysis of the divider wall was performed in accordance with Section III, Appendix F of the ASME Boiler and Pressure Vessel Code, and all stresses were within allowable limits.

d. Conclusions

The analyses performed on the New Fuel Storage Tanks ans Support System show the stresses in both the inner and outer tanks to be within the appropriate allowables specified in Section III of the ASME Boiler and Pressure Vessel Code. These analyses are predicated on acceleration loadings associated with the design basis seismic event. Consequently, it can be concluded that the new fuel storage system is capable of withstanding the design basis seismic event for the GETR building and will remain functional.

D. Integrity of Fuel Flooding System

This section of testimony summarizes the results of the structural design and analysis of the GETR Fuel Flooding System (FFS) for the effects of earthquake-induced forces due to postulated vibratory ground motions and surface rupture offset. The FFS is designed to meet the system requirement of keeping the fuel elements covered by replacing water in the containers (reactor pressure vessel and canal fuel storage tanks) due to evaporation or boil-off. The FFS is designed to automatically provide the water to the RPV and the canal fuel storage tanks located in the Reactor Building. The system will activate at a low-level motion induced by an earthquake and will assure that the fuel located in the Reactor Building will be covered by water for an extended period of time without assistance from personnel at GETR. The details of the structural analyses of the FFS are presented in Reference 22.

The FFS consists of two redundant reservoir and piping systems as shown schematically in Figure D-1.

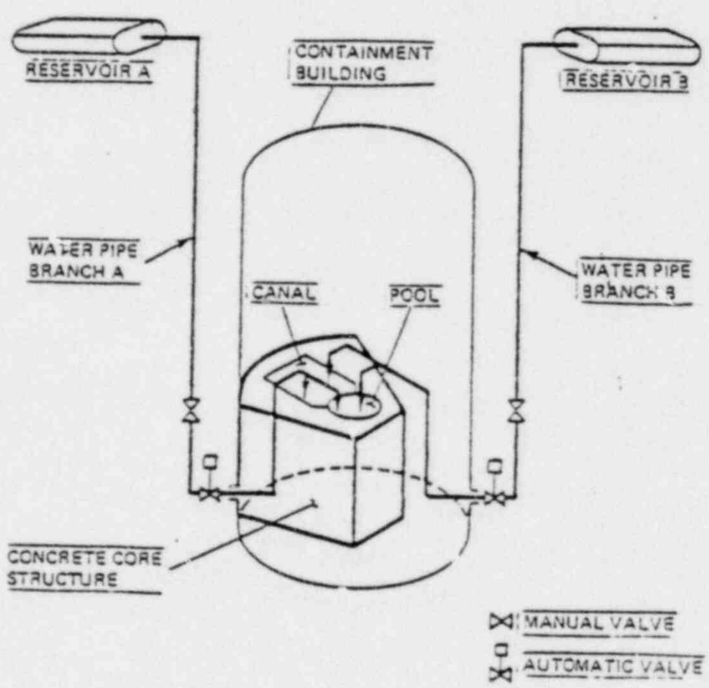


FIGURE D-1. FUEL FLOODING SYSTEM (FFS)

Each system is capable of delivering the required flow rate to the RPV and canal storage tanks. Four 50,000-gallon, flexible, nylon reinforced water reservoirs are placed on two hills adjacent to the Reactor Building as shown in the location map in Figure D-2.

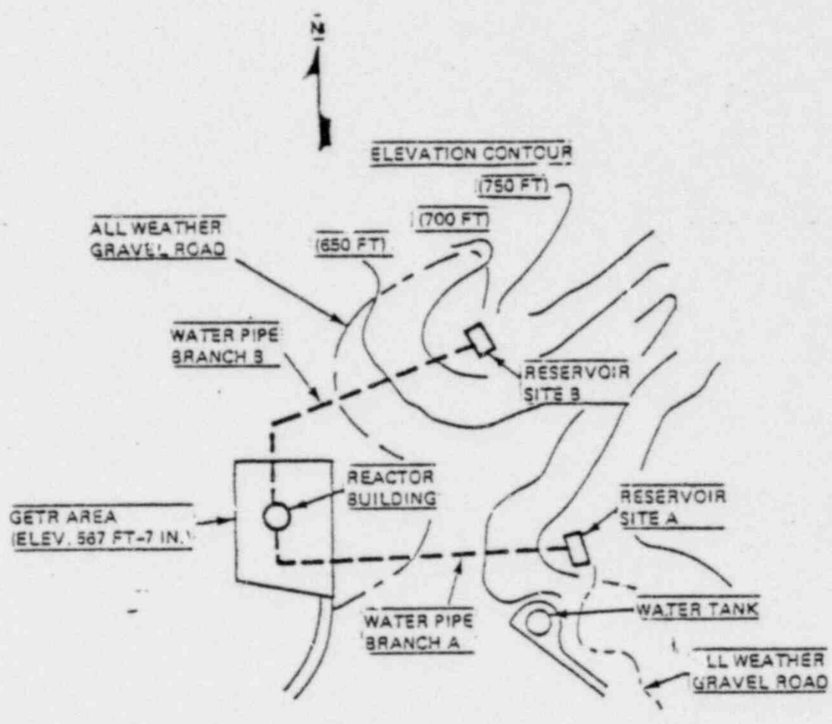


FIGURE D-2. FFS LOCATION MAP

The two reservoirs at each site supply water through flexible, reinforced, synthetic rubber pipes to the Reactor Building. Each water supply line approaches and passes through the containment shell from a different angle and follows a separate route to the fuel storage tanks in the canal and to the RPV.

1. <u>Design of FFS Structures</u>

Each necessary component of the FFS was identified, designed, and evaluated. The structures and systems which were included in the FFS structural analysis were the FFS reservoirs, FFS reservoir retaining structures, and reservoir valve wells (at the reservoir sites, Figure D-3).

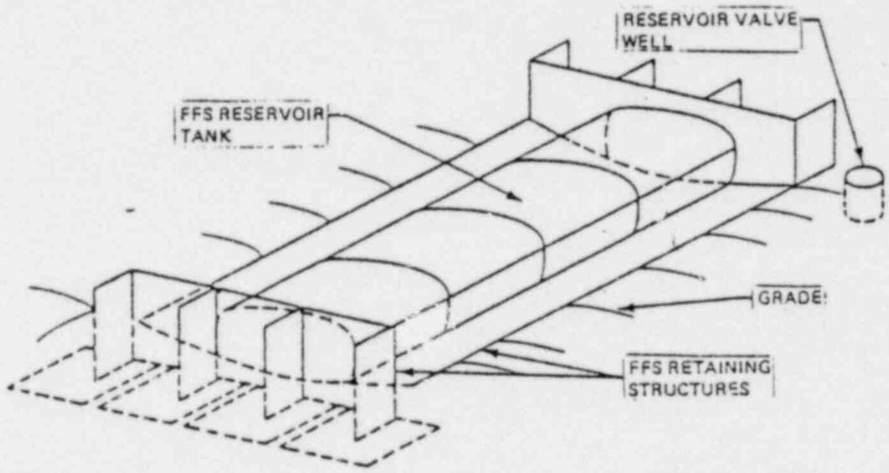
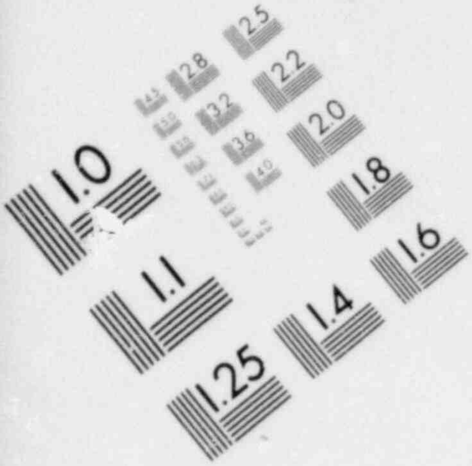


FIGURE D-3. COMPONENTS OF FFS AT RESERVOIR SITE

IMAGE EVALUATION
TEST TARGET (MT-3)

6″



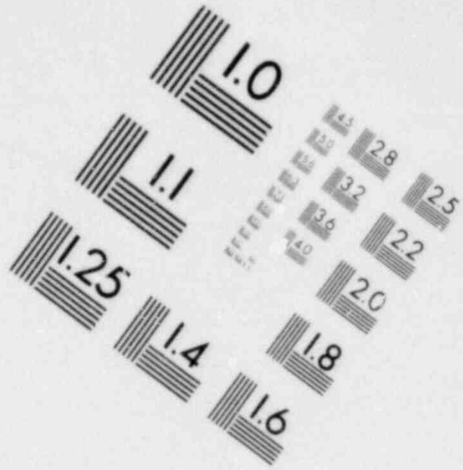
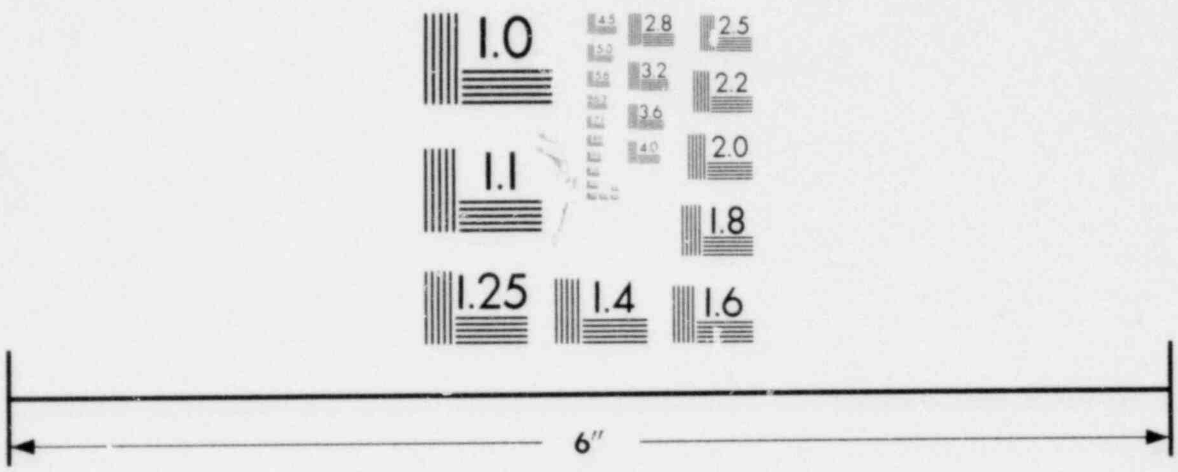
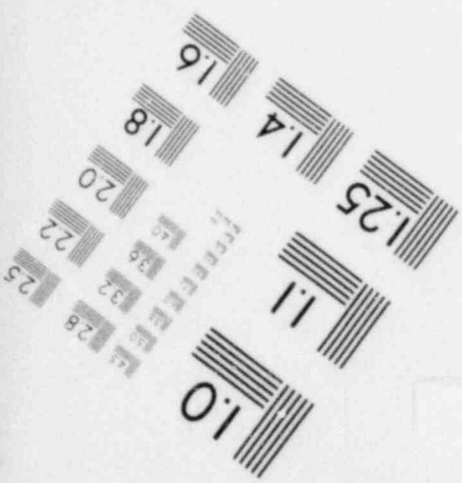
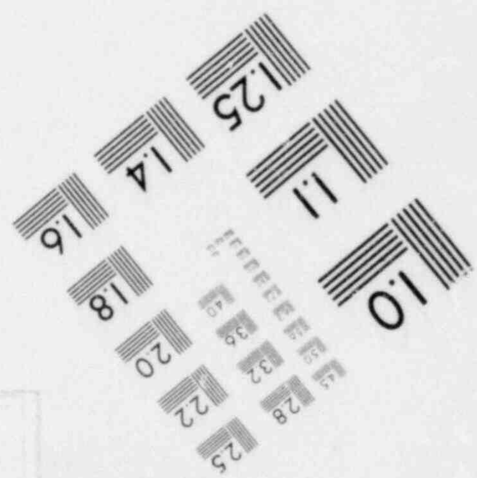


IMAGE EVALUATION
TEST TARGET (MT-3)







Also included were the water pipes (Figure D-2); and the steel shield pipes, water pipe/shield pipe transition pad, and penetration valve wells in the GETR yard area (Figure D-4).

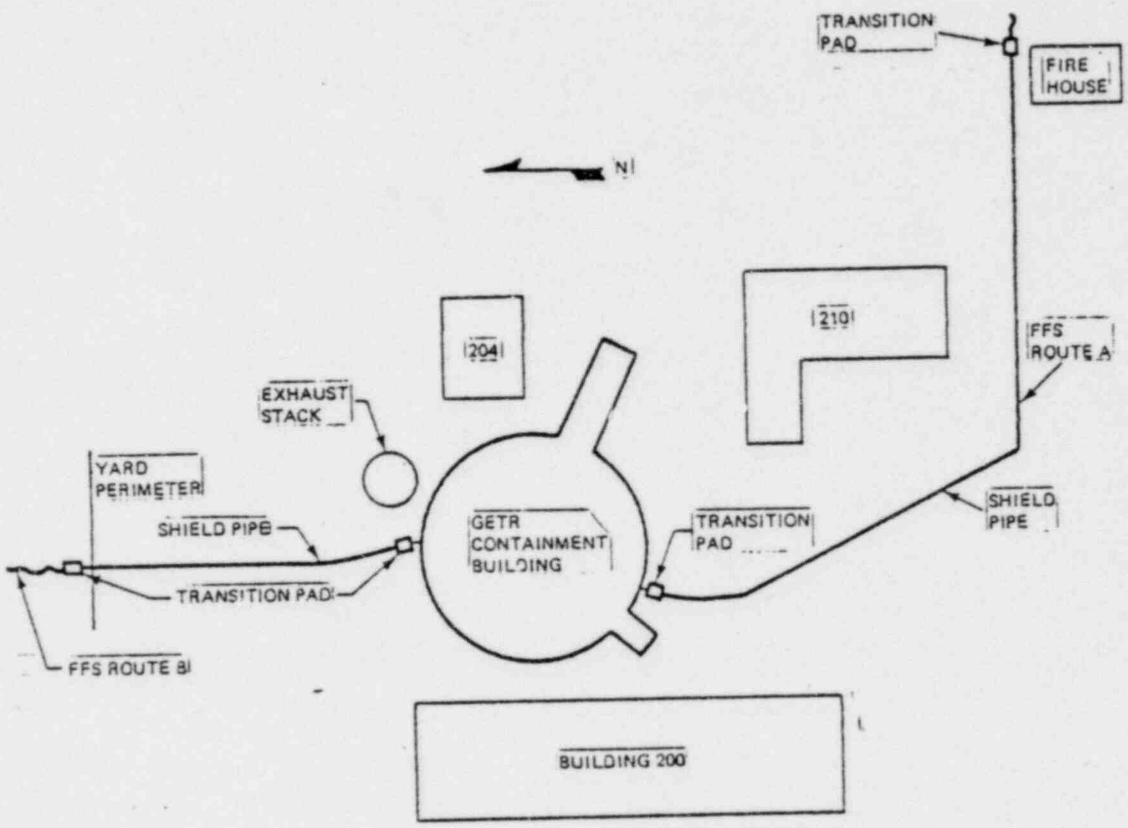


FIGURE D-4. FFS LINES IN GETR YARD AREA

The FFS reservoirs consist of four 50,000-gallon, flexible, nylon-reinforced, rubber reservoirs. The vibratory ground motions may cause the water in the reservoirs to slosh. Conservative analyses were performed to demonstrate that the membrane stress resulting from sloshing in the reservoirs is less than the ultimate strength. A foundation of sufficient size and radius was determined such that the flexible reservoirs will not displace an excessive amount (Figure D-5).

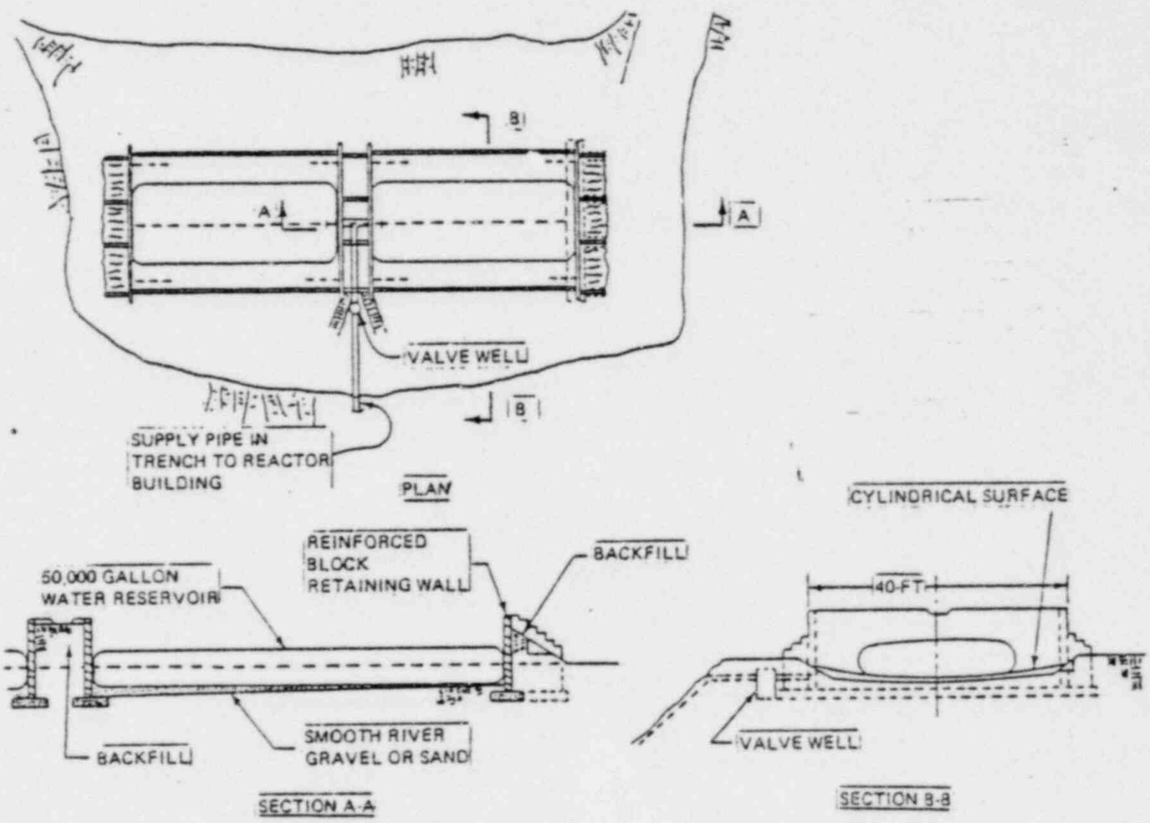


FIGURE D-5 TYPICAL FFS RESERVOIR AND RETAINING STRUCTURE

POOR ORIGINAL

The flexible water pipe between the reservoir and the valve wells was arranged so that adequate slack will be provided. The reservoirs were evaluated for a surface rupture offset, and it was demonstrated that the reservoirs have adequate restraints to withstand this phenomenon. These restraints are reinforced concrete block retaining wall structures to be constructed at the ends of each of the reservoirs (Figure D-6).

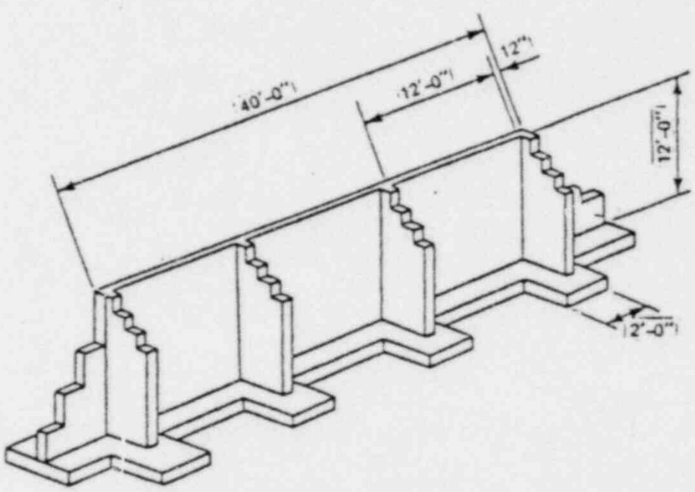


FIGURE D-6. TYPICAL END RETAINING WALL

Conservative loads on the walls and wall footings due to the soil behind the walls and potential sloshing from the reservoir were determined and used in the design.

The reservoir valve wells (Figure D-7) are simple, reinforced concrete structures which contain the FFS valves and control instruments at the juncture of the FFS lines from the reservoirs and the FFS line to the Reactor Building.

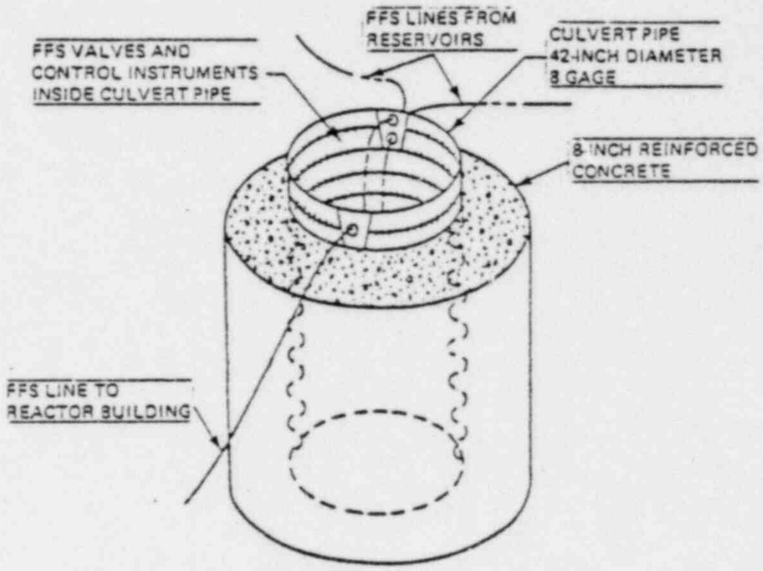


FIGURE D-7. RESERVOIR VALVE WELL

These valve wells consist of cylindrical, reinforced concrete sections surrounding an eight-gauge metal culvert pipe. The valve well structures are compact and will respond as rigid bodies when subjected to the ground motions and can easily withstand forces imposed by the postulated seismic events.

A 1.5" diameter, reinforced, synthetic rubber pipe is used to transport water from each reservoir site to the Reactor

Building. Each pipe is placed in an S-shaped configuration in an eight-inch deep trench, which is backfilled with gravel placed level with t. ground surface (Figure D-8).

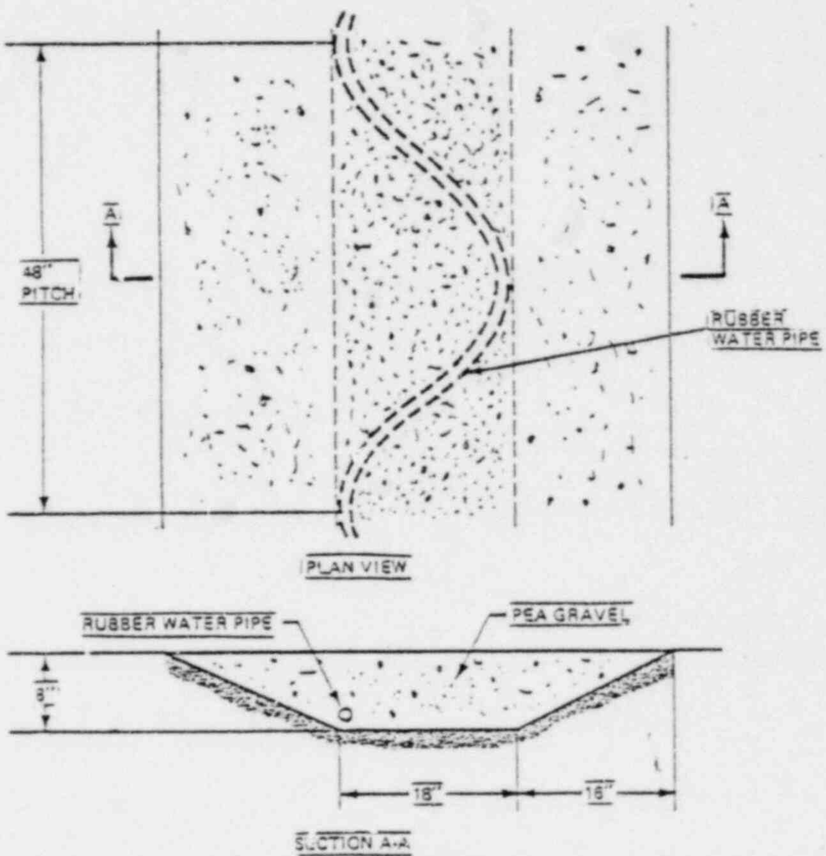


FIGURE D-8. WATER PIPE TRENCH

A test of this pipe was performed to demonstrate that a surface rupture offset underneath the trench may cause the hose to displace out of the ground, but failure would not occur.

In the yard surrounding the GETR, each rubber pipe is protected by a 4" diameter stainless steel pipe, and is buried 12" deep in a gravel-filled trench topped with asphalt or concrete (Figure D-9).

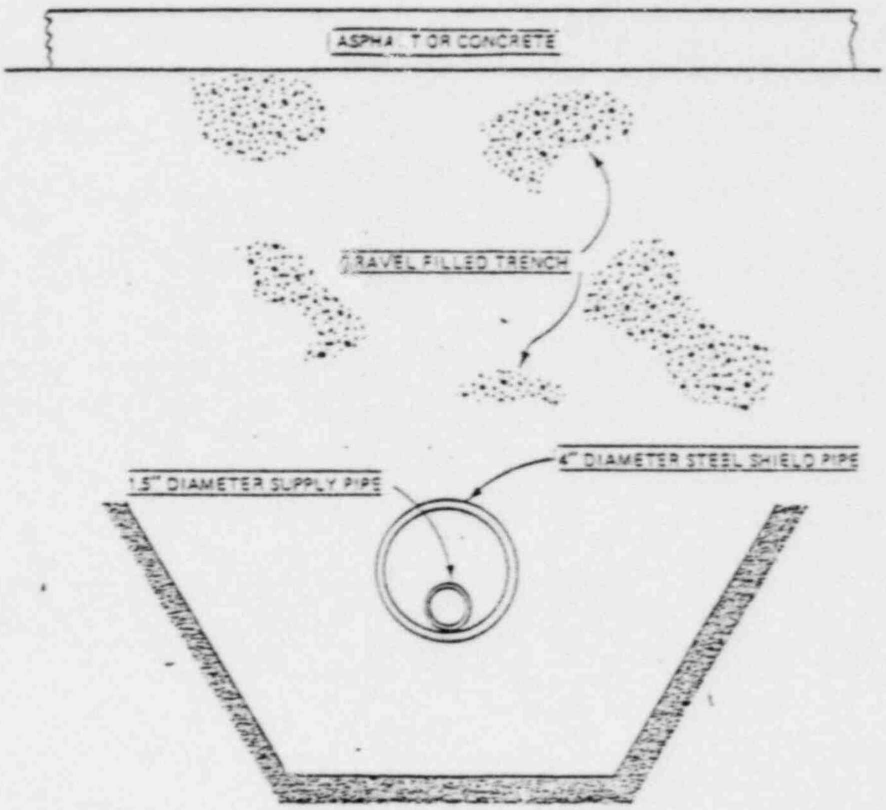


FIGURE D-9. SHIELD PIPE

The shield pipe and ground provides protection for the rubber pipe against vehicular traffic. An analysis was conducted to verify that the design basis surface rupture offset will not cause the shield pipe to pinch or squeeze the contained rubber pipe and thus shut off the flow of water.

At locations where the rubber pipe in the trench enters the shield pipe (e.g. at the end of the yard area, Figure D-4) and leaves the shield pipe (e.g., to enter the Reactor Building), reinforced concrete transition pads are provided to protect the rubber pipe (Figure D-10).

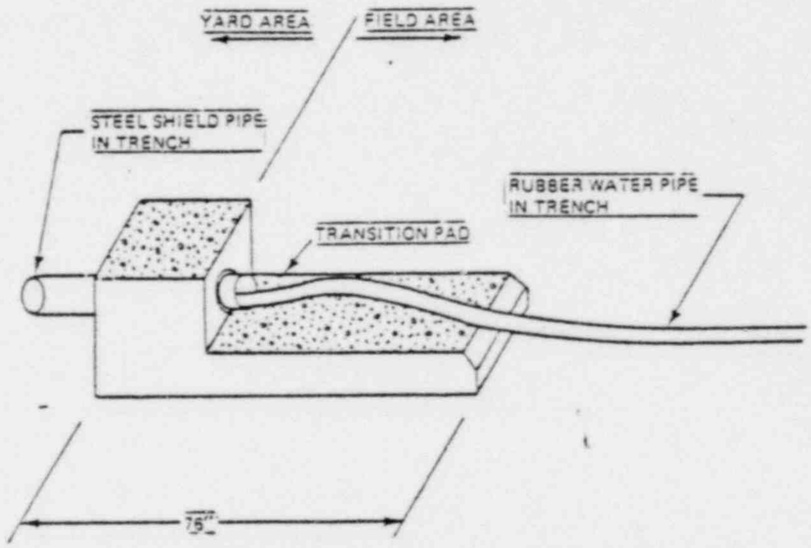


FIGURE D-10. TRANSITION PAD

In the event that the rubber pipe is pushed or pulled out of the ground due to surface rupture offset, the concrete transition pad will prevent the end of the shield pipe from kinking the rubber pipe.

Two steel penetration valve well structures which support and protect the FFS valves are located on the north and south sides of the containment shell between the first and second floors of the Reactor Building (Figure D-11).

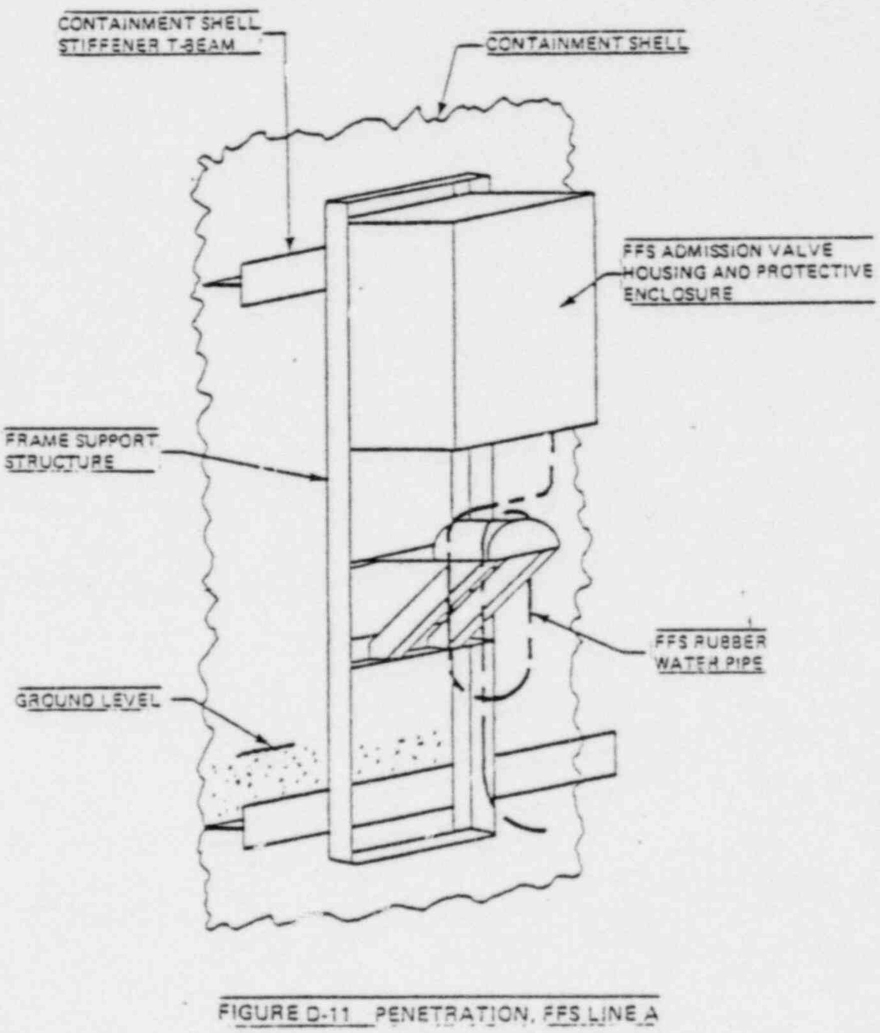


FIGURE D-11 PENETRATION, FFS LINE A

The penetration valve well structures were analyzed for the postulated vibratory motions. It was determined that the computed stresses in the supporting frame structures due to the design basis seismic events were less than the allowable stresses.

All valves in the FFS have been seismically qualified as described previously in Section B of this testimony.

2. Potential Impinging Structures

A systematic evaluation of all structures and objects w[illegible] could possibly fall and affect the FFS lines was pe[illegible]ed. All potential impinging structures and objects loca[illegible] within the GETR yard area and Reactor Building which could conceivably damage the FFS were investigated. Figure D-12 shows the two FFS line Routes A and B through the yard area to the Reactor Building and identifies the structures and objects which were evaluated.

The structures and objects included the fire house, cooling tower and equipment, ventilation isolation valves, liquid nitrogen tanks, stack, operations storage shed, miscellaneous objects on exterior of Reactor Building, electrical terminal box, penetration nozzles, miscellaneous objects on the interior of Reactor Building, and the Reactor Building floor slabs. For each potential impingement, the FFS was demonstrated to be adequate because (1) the path of the potentially i[illegible]nging component was shown not to intersect the path of the FFS, (2) the component was

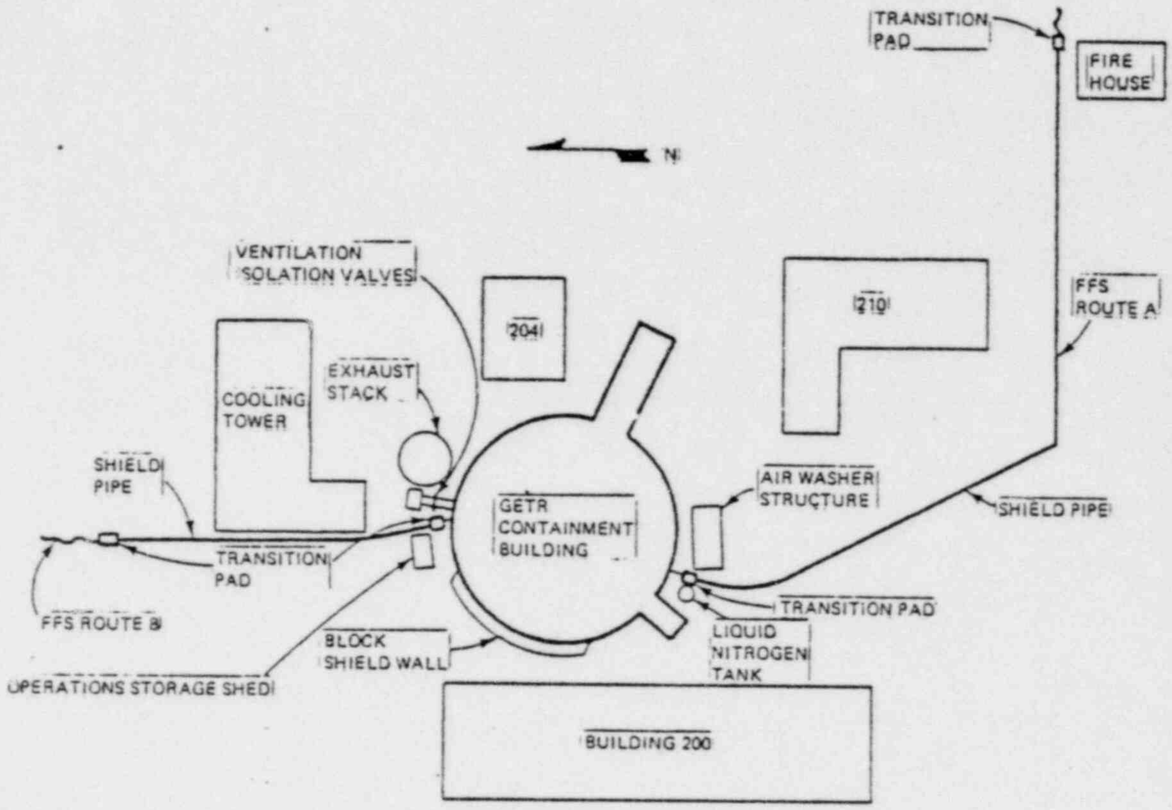


FIGURE D-12. FFS LINES

strengthened such that it will not fail, or (3) the FFS line was adequately protected by a structural shield.

3. Conclusions

Detailed seismic evaluations of the structures and components which form the FFS were analyzed for the effects of earthquake-induced forces due to the design basis seismic events. In addition, analyses were performed for structures which could possibly fail and affect the FFS. It was found that the FFS structures are protected and that the FFS system can withstand the effects of the postulated seismic events.

Thus, the Fuel Flooding System (FFS) will meet the system requirement of keeping the fuel elements in the canal storage tanks and RPV covered by replacing water in these containers due to evaporation and boil-off.

IV. CONCLUSIONS

The structural and mechanical analyses described in this testimony were performed to show that the GETR safety-related structures and equipment meet the following requirements:

1. Assure integrity of Reactor Building concrete core structure which supports other systems and components important to safety
2. Assure integrity of reactor pressure vessel
3. Assure integrity of canal fuel storage tanks
4. Assure capability of providing make-up water to spent fuel storage tanks and reactor vessel.

The above requirements are met as follows:

A. Integrity of Reactor Building Concrete Core Structure

The structural investigations demonstrated that the rigid massive concrete core structure of the GETR Reactor Building, which supports other systems and components important to safety, is assured.

An extensive program of investigations was undertaken to demonstrate the adequacy of the concrete core structure to withstand seismic effects postulated for the site. The investigations focused on two basic earthquake phenomena:

- o Ground shaking due to an earthquake on the Calaveras or Verona faults.
- o A ground displacement, denoted "surface rupture offset," at the site due to an earthquake on the Verona fault.

The investigations were divided into three main parts consistent with the seismic criteria for the site:

1. Analysis for effects of vibratory ground motions caused by an earthquake on the Calaveras fault.
2. Analyses for effects of vibratory ground motions combined with a surface rupture offset caused by an earthquake on the Verona fault.
3. Analyses for effects of vibratory ground motions caused by an aftershock.

In addition, the numerous conservatisms in the seismic evaluations of the GETR Reactor Building were qualitatively examined. This included characterization of earthquakes, the Verona fault, analytical models, and strength and capacity. It was concluded that, if all individual safety margins in each main step or parameter in the evaluations were quantified, the result would be a total margin of safety significantly above that determined to be required based on the seismic evaluations of the GETR Reactor Building.

In these investigations, mathematical computer models were used to represent the physical characteristics of the concrete core structure and its behavior when subjected to earthquakes.

The following summarizes the conclusions resulting from these investigations:

1. <u>Evaluations for an Earthquake on the Calaveras Fault</u>

The analyses showed that the stresses in the concrete core structure induced by an earthquake on the Calaveras fault were smaller than the capacity stresses. These stresses were based on the forces obtained from linear elastic dynamic analyses. Nonlinear analyses, including the nonlinearities due to potential uplift and sliding of the Reactor Building at the foundation slab-soil interface, were also performed. The results confirmed the conservatism of the linear analyses. The nonlinear analyses also demonstrated that the Reactor Building is stable against potential uplift and sliding and that the soil pressures remain below the soil capacity. The analyses showed that, although some cracking of slabs exterior to the safety-related concrete core structure may occur, the deformations of these slabs will be small, resulting only in minor, insignificant, non-safety-related cracking.

It was therefore concluded that integrity of the Reactor Building concrete core structure is assured for an earthquake on the Calaveras fault.

2. <u>Evaluation for an Earthquake on the Verona Fault</u>

Even though (1) the surface rupture offset failure zone was shown to bypass the Reactor Building foundation, and (2) the probability of a surface rupture offset under the foundation at the site is low, the surface rupture offset was assumed to occur for the purposes of the structural investigations. The resulting loadings produced on the Reactor Build' g depended upon the point at which the surface rupture offset was assume o intersect the building. Thorough investigations identified two basic cases which needed to be considered. Case 1 primarily involved the production of soil pressures on the exterior ring wall between the basement and first floor levels. In Case 2, the surface rupture offset was assumed to intersect the Reactor Building concrete foundation mat. This case primarily involved the potential for loss of foundation soil support of the structure in certain regions. Detailed structural analyses demonstrated that the concrete core structure is adequate to withstand the loadings on the structure produced in both of these cases, w 'ch led to the conclusion that the concrete core structure is adequate to withstand the combined vibratory ground motion and a surface rupture offset due to a postulated earthquake on the Verona fault.

3. Post-Offset Analyses

An analysis was performed to demonstrate that the concrete core structure could resist aftershock ground motions. It was determined that the concrete core structure of the Reactor Building would be stable and the stresses in the structure would be within acceptable limits; this led to the conclusion that the concrete core structure is adequate to withstand aftershock ground motions.

4. Summary for All Evaluations of the Concrete Core Structure

Finally, for all cases, it was concluded that the detailed analyses performed for the vibratory ground motions and surface rupture offset demonstrate that the concrete core structure which surrounds the pool and storage canal will not be damaged in the event that major earthquake motions and/or surface rupture occur at the GETR site. Thus, the structural and mechanical requirement to assure the integrity of the concrete core structure (which supports other systems and components important to safety) is met.

B. Integrity of Reactor Pressure Vessel

All safety-related piping and equipment, as modified, have been shown to be adequate to meet the specified seismic criteria. The piping and equipment described in this section of testimony were those items which are required to meet the functional requirement of keeping the fuel elements covered with water. This was accomplished by meeting the structural and mechanical requirement of keeping the fuel element containers intact. These containers consist of the reactor pressure vessel and associated piping and components, and the canal storage tanks and associated appurtenances. Section B, Part III of the testimony addressed the reactor pressure vessel and associated piping and components. The canel storage tanks were discussed in Section C, Part III of the testimony.

The basic approach was to ensure that the fuel elements will remain covered by verifying the adequacy of or modifying any component which is required to maintain the water in the reactor pressure vessel and pool. Modifications were in the form of adding seismic restraints (i.e., braces) to the piping or component which restricted its movement during seismic events. As was the case with the Reactor Building concrete core structure, mathematical computer models were used to represent the physical characteristics of the safety-related piping and equipment, and their behavior when subjected to earthquakes. In addition, physical tests were performed to demonstrate the adequacy of safety-related valves.

The major elements evaluated included the:

1. Reactor Pressure Vessel and Primary Cooling System
2. Primary Heat Exchanger
3. Reactor Pressure Vessel and Pool Drain Lines and Poison Injection Line
4. Safety-Related Valves
5. Heat Exchanger HE-102
6. Control Rod and Incore Shuttle Assemblies

In conclusion, it was demonstrated by analysis, modification and analysis, or testing that all safety-related piping and equipment are adequate to resist the motions induced by the postulated seismic events for the GETR site. These components, therefore, meet the structural and mechanical requirement that the fuel element containers remain intact, and functional requirement that the fuel elements in the reactor pressure vessel remain covered with water.

C. Third Floor Missile Impact System and New Fuel Storage System

The third floor missile impact system consists of a series of structural bents and honeycomb impact pads which prevent damage due to postulated collapse of the polar crane trolley assembly. The structural bents were analyzed under maximum postulated seismic design basis and aftershock loading, and it was shown that the system will remain functional under those conditions.

The new fuel storage tanks consist of three separate inner tanks within an outer tank located in the fuel canal pool. These tanks were analyzed under maximum postulated seismic loading conditions, and it was shown that the tanks would remain functional under those conditions.

D. Integrity of Fuel Flooding System

Section D, Part III of the testimony summarized the results of the structural design and analysis of the GETR Fuel Flooding System (FFS) for the effects of earthquake-induced forces due to postulated vibratory ground motions and surface-rupture offset. The FFS is designed to meet the system requirement of keeping the fuel elements covered by replacing water in the containers (reactor pressure vessel and canal fuel storage tanks) lost due to evaporation or boil-off. The FFS is designed to automatically provide the water to the RPV and the canal fuel storage tanks located in the Reactor Building. The system will activate at a low-level motion induced by an earthquake and will assure that the fuel located in the Reactor Building will be covered by water for an extended period of time without assistance from personnel at GETR.

Each necessary component of the FFS was identified, designed, and evaluated. The structures and systems which were included in the FFS structural analysis were the 50,000 gallon flexible nylon-reinforced reservoir tanks, FFS reservoir retaining structures, and reservoir valve wells at the reservoir sites. Also included were the water pipes in trenches between the reservoirs and the Reactor Building; and the steel shield pipes, water pipe/shield pipe transition pads, and penetration valve wells in the GETR yard area. Also, a systematic evaluation of all structures and objects which could possibly fall and affect the FFS lines was performed. All potential impinging structures and objects located within the GETR yard area and Reactor Building which could conceivably damage the FFS were investigated. The FFS was demonstrated to be adequate because (1) the path of any potentially impinging component was shown not to intersect the path of the FFS, (2) the component was strengthened such that it will not fail, or (3) the FFS line was adequately protected by a structural shield.

In summary, detailed seismic evaluations of the structures and components which form the FFS were analyzed for the effects of earthquake-induced forces due to the design basis seismic events. In addition, analyses were performed for structures which could possibly fail and affect the FFS. It was found that the FFS structures are protected and that the FFS system can withstand the effects of the postulated seismic events.

Thus, the Fuel Flooding System (FFS) will meet the system requirement of keeping the fuel elements in the canal storage tanks and RPV covered by replacing water in these containers lost due to evaporation or boil-off.

E. Overall Conclusions of Structural and Mechanical Analyses

The structural and mechanical analyses described in the testimony demonstrated that the GETR safety-related structures and equipment meet the following requirements:

1. The integrity of the Reactor Building concrete core structure which supports other systems and components important to safety is assured;

2. The integrity of the reactor pressure vessel is assured;

3. The integrity of the canal fuel storage tanks is assured; and

4. The capability of providing make-up water to the spent fuel storage tanks and reactor pressure vessel is assured.

((

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Education

B.S. - Civil Engineering, Stanford University (Magna Cum Laude), 1961
M.S. - Structural Engineering, Stanford University, 1962
Degree of Engineer - Structural Engineering, Stanford University, 1963
Ph.D. - Structural Engineering, Stanford University, 1972

Registration

Civil Engineer, California (16842); Structural Engineer, California (1659)

Professional Experience

Dr. Kost has extensive experience in the analysis and design of structures, piping systems, and mechanical and electrical equipment for the effects of natural and man-made hazards. He has been responsible for seismic investigations of numerous types of facilities with emphasis on nuclear power plants and other installations in the nuclear field, such as test reactors, nuclear fuel fabrication and reprocessing facilities, and research facilities. He has extensive experience in the representation of soil-structure interaction effects and finite-element techniques, and has been responsible for the analysis of above-ground, partly embedded and buried structures. By means of research for his dissertation and his experience with these analyses, he also has a solid background in nonlinear analysis techniques and the effects of nonstructural components on building response.

He has been responsible for analyses for explosion, hydrogen burn, and other pressure wave phenomena, as well as hydrodynamic effects. In addition, he as been responsible for the independent review of the analyses and designs of numerous nuclear and related facilities, including review of the adequacy of seismic designs and seismic design criteria of 19 nuclear power plants and 1 fuel reprocessing plant for the USAEC Directorate of Licensing. Dr. Kost has supervised thermal stress and seismic analyses of piping systems and components, and was responsible for the development of a comprehensive computer program for the static and dynamic analysis of piping systems which is now marketed worldwide.

He has conducted evaluations of structures for effects of ground motions induced by underground nuclear explosions, and for aircraft crash, cask drops and other missile impact effects. Included in the above investigations have been analyses as well as development of design modifications to resist extreme loadings. In addition, he was responsible for development of structural analysis programs and evaluation of structural response data to determine the dynamic response of structures to sonic boom shock waves. Dr. Kost has also designed public and commercial structures including schools, hospitals, office buildings, parking garages, warehouses, and shopping centers. These designs consisted of steel and concrete frame, shear wall, precast and prestressed concrete, and wood single- and multistory buildings.

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SUNNYVALE, CA 94086

RESUME

Dr. Harold Durlofsky, Associate

EDUCATION: Drexel U., Phila., Penna., B.S. in C.E., 1957
San Jose State U., San Jose, Calif., M.S. in C.E., 1965
Stanford U., Stanford, Calif., Ph.D. in Ae.E., 1970

EXPERIENCE: Seventeen years experience in the areas of stress analysis, structural analysis, and structural dynamics. This experience includes development of both analytical and finite element analysis techniques, and applications in the civil, aeronautical and mechanical engineering fields.

Areas of extensive experience include:

- Development of computer codes based on Finite Element and Rayleigh-Ritz methods.
- Static and dynamic analysis of piping systems and power plant structures.
- Development and application of stress analysis techniques for fiber-reinforced, composite structures.
- Analysis of aircraft and missile structures.
- Analysis of gears and mechanisms.
- Application of ASME Pressure Vessel code.

REGISTRATION: Registered Civil Engineer, State of California

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DWIGHT L. GILLILAND

General Electric Company

EDUCATION:

BS Electrical Engineering, Kansas State University, 1952

PROFESSIONAL:

Professional Engineer Certificate of Registration
No. NU 1665 California, 1977

EXPERIENCE:

Mr. Gilliland has been employed by General Electric Company since 1953, serving in several engineering and operations management positions. His first assignment was in the Aircraft Nuclear Propulsion Department where he worked in shielding experimentation and instrumentation at the Oak Ridge National Laboratory and later supervised the installation and checkout of nuclear power plant instrumentation at the Idaho Test Station. He also managed the Shield Test Facility there. It utilized a pool reactor and advanced instrumentation for the testing of experimental reactor shield configurations.

From 1961 to 1965, he managed in succession, two re-entry vehicle projects at Vandenburg Air Force Base for the Re-entry Systems Department. In 1963, Mr. Gilliland joined the Nuclear Thermionic Power Operations where he was the responsible for testing thermionic devices and for the design of the thermionic reactor experiment.

From 1970 to 1973, he managed the development, design and requisition engineering and process development of nuclear sensors, penetration seals, radiation monitoring and source products in the Nuclear Instrumentation Department. Since 1973, Mr. Gilliland has managed the operations and plant engineering for the General Electric Test Reactor and the Nuclear Test Reactor. He also manages the activities related to GETR License Renewal.

UNITED STATES OF AMERICA

NUCLEAR REGULATORY COMMISSION

In the Matter of GENERAL ELECTRIC COMPANY (Vallecitos Nuclear Center - General Electric Test Reactor)	Docket No. 50-70 Operating License No. TR-1 (Show Cause)

CERTIFICATE OF SERVICE

I hereby certify that the foregoing has been served as of this date by personal delivery or first class mail, postage prepaid, to the following:

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George L. Edgar

George L. Edgar
Attorney for
General Electric Company

Dated: May 1, 1981