October 19, 1982

Repro 10/22

Docket No. 50-155
LS05-82-10-056

Mr. David J. VandeWalle
Nuclear Licensing Administrator
Consumers Power Company
1945 W. Parnall Road
Jackson, Michigan 49201

Dear Mr. VandeWalle:

SUBJECT: SEP TOPICS III-6, SEISMIC DESIGN CONSIDERATIONS AND III-11, COMPONENT INTEGRITY - BIG ROCK POINT NUCLEAR POWER STATION

Enclosed is our draft safety evaluation for the seismic design of the Big Rock Point Plant. The staff's review is based on both preliminary and final analyses, and several working-level meetings between CPCo and NRC personnel. CPCo has not yet completed the seismic reevaluation of Big Rock Point. Therefore, the conclusions presented in the evaluation may be revised should new information be presented in later CPCo seismic reports.

Based upon the NRC staff and its consultants review of the analyses and criteria supplied by the licensee for structures, buried piping and portions of the reactor coolant loop piping, we cannot conclude that these analyses are adequate. Further, as indicated in the enclosure, significant analyses for piping, equipment and components are yet to be performed, and acceptable analysis criteria have not been established. Current licensee schedules for the completion of such analyses are about late 1983 to early 1984. No schedule has been provided for the implementation of any required modifications.

This evaluation will be a basic input to the integrated safety assessment for your facility unless you identify changes needed to reflect the as-built conditions at your facility. With respect to the potential modifications and open items outlined in this report, a determination of the need to actually implement these or other changes will be made during the same integrated assessment. This topic assessment may be revised in the future if your facility design is changed or if NRC criteria relating to this topic are modified before the integrated assessment is completed.

Add: Gary Staley

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Sincerely,

Original signed by:

Dennis M. Crutchfield, Chief
Operating Reactors Branch No. 5
Division of Licensing

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*See previous concurrence

Enclosure:
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in the future if your facility design is changed or if NRC criteria relating to this topic are modified before the integrated assessment is completed.

Sincerely,

Dennis M. Crutchfield, Chief
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SYSTEMATIC EVALUATION PROGRAM

Topics III-6 and III-11

Big Rock Point Nuclear Power Plant

TOPICS: III-6, SEISMIC DESIGN CONSIDERATIONS
III-11, COMPONENT INTEGRITY

I. INTRODUCTION

The eleven nuclear power plant facilities under review in the SEP received construction permits between 1956 and 1967. Seismic design procedures evolved significantly during and after this period. The Standard Review Plan (SRP), first issued in 1975; along with the Regulations 10 CFR Part 50, Appendix A; and 10 CFR Part 100, Appendix A constitute current licensing criteria for seismic design reviews. As a result, the original seismic design of the SEP facilities vary in degree from the Uniform Building Code up through and approaching current standards. Recognizing this evolution, the staff found that it is necessary to make a reassessment of the seismic safety of these plants.

Under the SEP seismic reevaluation, these eleven plants were categorized into two groups based upon the original seismic design and the availability of seismic design documentation. Different approaches were used to review the plant facilities in each group. The approaches were:

Group I: Detailed NRC review of existing seismic design documents with limited reevaluation of the existing facility to confirm judgments on the adequacy of the original design with respect to current requirements.

Group II: Licensees were required to reanalyze their facilities and upgrade, if necessary, the seismic capacity of their facility. The staff reviews the licensee's reanalysis methods, scope and results. Limited independent NRC analysis performed to confirm the adequacy of the licensee's method and results.

Based on the staff's assessment of the original seismic design, the Big Rock Point plant was placed in Group II for review.

The Big Rock Point Nuclear Power Plant is a 240 MW thermal boiling-water reactor. It is located about four miles northeast of Charlevoix, Michigan on the shore of Lake Michigan. Initial site investigations for the unit were conducted in May 1959 and in November 1959 through January 1960 by Raymond Concrete Pile Company. A geophysical cross-hole survey was conducted in 1978 and presented in January 1979 in a D'Appolonia Consulting

Engineers, Inc report. The nuclear steam supply system (NSSS), including the reactor vessel, was designed and furnished by the General Electric Company as subcontractor to Bechtel, Inc., the architect-engineer for the plant. Construction was begun in the spring of 1960 and the power plant became commerically operational in December 1962. The construction permit and operating license were granted in May 1960 (CPPR-9) and August 1962 (DPR-6), respectively.

The initial seismic criteria applied to Big Rock Point were significantly less stringent than would be required by current regulatory criteria. Based upon the low probability of earthquakes, it was concluded initially that special seismic design was not required. The design of the facility's major structures was based upon the static coefficient seismic requirements of the 1958 edition of the Uniform Building Code. The containment design was based upon a 0.05g horizontal static coefficient. The Reactor Building Internal Structure, Turbine Building, Concrete Stack, Intake Structure, Control Room and Waste Storage Building were designed based upon a 0.025g horizontal static coefficient.

The only piping and equipment described as being designed to resist seismic loads are the reactor vessel supports, NSSS piping and the Reactor Depressurization System (RDS). The reactor vessel supports were designed to resist a horizontal static coefficient of 0.05g while the NSSS piping was design to resist a horizontal static coefficient of 0.025g. The RDS was redesigned in 1974 in accordance with 1974 seismic requirements, which are comparable to current seismic criteria, assuming a 0.12g Regulatory Guide 1.60 Safe/Shutdown Earthquake (SSE). The 1974 structural analyses were performed to generate floor response spectra only, not to evaluate building seismic capacities. A more detailed description of the original seismic design bases for the Big Rock Point plant is contained in the draft summary report "Seismic Design Bases and Criteria for Big Rock Point Nuclear Generating Station," dated January 1979 (Attachment 1).

The SEP seismic review of the Big Rock Point facility addressed only the Safe Shutdown Earthquake (SSE), since it represents the most severe seismic event that must be considered in the plant design. The scope of the review included three major areas: (1) the integrity of the reactor coolant pressure boundary; (2) the integrity of fluid and electrical distribution systems related to safe shutdown; and (3) the integrity of mechanical and electrical equipment designed as engineered safety feature systems (including containment).

Via NRC 10 CFR 50.54(f) letters dated August 4, 1980 and April 24, 1981 (References 1 and 2), the licensee, Consumers Power Company (CPCo), was requested to seismically reevaluate and upgrade, if necessary, all safety-related structures, systems and components to a level of seismic resistance acceptable to the NRC staff.

II. REVIEW CRITERIA

Since the SEP Group II plants were not designed to current codes, standards, and NRC requirements, it was necessary to perform "more realistic" or "best estimate" assessments of the seismic capacity of the facility. A set of review criteria and guidelines was developed for the SEP plants. These review criteria and guidelines are described in the following documents:

1. NUREG/CR-0098, "Development of Criteria for Seismic Review of Selected Nuclear Power Plants", by N. M. Newmark and W. J. Hall, May 1978.

2. "SEP Guidelines for Soil-Structure Interaction Review", by SEP Senior Seismic Review Team, December 8, 1980.

3. Letter to D. J. VandeWalle (CPCo) from D. M. Crutchfield (NRC), "Systematic Evaluation Program Position Re: Consideration of Inelastic Response Using the NUREG/CR-0098 Ductility Factor Approach," dated June 23, 1982.

4. Letter to D. J. VandeWalle (CPCo) from D. M. Crutchfield (NRC), "SEP Topic III-6, Seismic Design Considerations, Staff Guidelines for Seismic Evaluation Criteria for the SEP Group II Plants," dated July 26, 1982.

5. Letter to D. J. VandeWalle (CPCo) from D. M. Crutchfield (NRC), "SEP Topic III-6, Seismic Design Considerations, Staff Guidelines for Seismic Evaluation Criteria for the SEP Group II Plants - Revision 1," dated September 20, 1982.

For the cases that are not covered by the criteria stated above, the following SRPs and Regulatory Guides were used for the review:

1. Standard Review Plan, Sections 2.5, 3.7, 3.8, 3.9 and 3.10.

2. Regulatory Guides 1.26, 1.29, 1.60, 1.61, 1.92, 1.100 and 1.122.

Any deviations from the criteria or guidelines were to be justified by the licensee on a case-by-case basis.

III. RELATED TOPICS AND INTERFACES

The SEP topics related to the review of Seismic Design Considerations and Component Integrity are II-4, II-4.A, II-4.B, and II-4.C. These topics relate to specification of seismic hazard at the site, namely, the site specific free-field ground response spectra for the Big Rock Point site.

IV. EVALUATION

A. General Approach

The seismic reevaluation of the Big Rock Point Nuclear Power Plant was initiated by conducting a detailed review of the docketed plant seismic related design criteria. The results of this review are summarized in Attachment 1. Based on the findings of this docket review, two NRC 10 CFR 50.54(f) letters were issued to require the licensee to complete a seismic reevaluation program. This program scope included: (1) providing a justification to demonstrate that the plant can continue to operate in the interim until the program is complete; (2) proposing a program plan which addresses the scope, criteria, and schedule for completion of the program; and (3) performing seismic analysis after staff acceptance of the proposed program, and providing the final results to the staff for review. The results of the staff review of this program would provide the basis for seismic safety assessment of the facility.

The licensee has not completed the requisite seismic analyses. As discussed later, several outstanding open issues remain to be resolved regarding the adequacy of the structural analyses presented in the August 1981 D'Appolonia reports on this subject. Evaluations of the safety-related masonry walls have not been provided for review. In addition, acceptable criteria has not been finalized regarding the piping, equipment and component evaluations which remain to be performed by the licensee.

The review approach which was followed on Big Rock Point was to first perform a general review of the summary volume (Volume 1) of the August 1981 D'Appolonia reports describing the analyses of the plant structures and portions of the NSSS. Selected structures were then subjected to a detailed audit review to determine the acceptability of the application of analytical techniques, and the corresponding results and conclusions. The structures were selected based upon 1) their importance to overall plant safety, and 2) the diversity of their seismic response, and applicable evaluation and acceptance criteria. The selected structures were the Reactor Building, the Turbine Building, the Ventilation Stack, the Intake Structure, and the Buried Intake and Fire Piping.

When structures are evaluated, they are judged to be adequately designed if:

1. The analyses are sufficient to adequately determine structural responses consisting of member loads, and floor response spectra for piping, equipment and component evaluations; and

2. The loads generated from the analyses are less than original loads; or

3. The seismic stresses from the analyses are low compared to reasonable estimates of the maximum strengths of the steel and concrete; or

4. The seismic stresses from the analyses exceed reasonable estimates of the steel or concrete maximum strengths, but estimated reserved capacity (or ductility) of the structure is such that inelastic deformation would be expected without structural failure or adverse impacts on piping, equipment or component responses.

If the above criteria are not satisfied, more comprehensive reanalyses are required to demonstrate design adequacy. The above Section II criteria 1 through 3 provide the basic guidelines for all evaluations, in conjunction with the previously referenced SRP and Regulatory Guide guidelines.

Certain major portions of the NSSS model were analyzed coupled with the reactor building and the results presented in the August 1981 D'Appolonia report on the subject. Due to certain inaccuracies (e.g., omission of the recirculation pump masses and misorientation of the steam drum supports, as stated by the licensee in meetings between the licensee and the staff) and to take advantage of the Site Specific Spectra, these analyses are being redone. However, the remaining piping systems have yet to be analyzed. Proposed criteria for the remaining piping evaluations was not provided until September 1, 1982 (Ref. 6). Given the incomplete nature of these evaluations and concerns resulting from the structures review, only a preliminary review of the piping criteria and evaluations was performed.

Piping is judged to be adequate if:

1. The analyses are sufficient to adequately determine piping system responses; and

2. The piping response stresses are in conformance with the criteria contained in References 4 and 5; or

3. The piping responses exceed the criteria referenced above, but estimated ductility is such that inelastic deformation is expected without loss of integrity or adverse impacts on the response of attached piping, equipment or components.

If the above criteria are not satisfied, more comprehensive reanalyses are required to demonstrate design adequacy. Criteria 1 through 5 in Section II above provide the basic guidelines for all evaluations, in conjunction with the previously referenced SRP and Regulatory Guide guidelines.

The licensee has indicated that certain mechanical components have been evaluated, but the details of these evaluations have not been provided. The proposed criteria for mechanical equipment was not provided by the licensee until September 1, 1982. Given the incomplete nature of these evaluations, only a preliminary review of the September 1, 1982, proposed mechanical equipment criteria could be performed. Although electrical equipment anchorage has been evaluated the effects of vertical slab flexibility, which will amplify the responses above those considered, remains to be addressed. (In meetings, the licensee has indicated that the Control Room floor, for example, has a first vertical mode frequency of approximately 6 to 8 Hz). No program has been described or implemented by the licensee for the evaluation of the structural integrity of electrical cabinets. The adequacy of mechanical and electrical equipment and components would be judged using criteria similar to that outlined above for the structures and piping.

B. Detailed Evaluation

1. Seismic Input

Per the results of the NRC Seismic Hazard Analysis Program (Ref. 7) conducted by the staff and its consultant, Lawrence Livermore National Laboratory (LLNL), the site specific ground response spectra, which are acceptable to the staff as input for the seismic reevaluation of the Big Rock Point plant, were recommended to the licensee through NRC letters dated August 4, 1980 and June 17, 1981 (Refs. 1 and 8). In addition, in May 1982, it was identified that possible anomalous soil conditions may exist at the Big Rock Point site which were not properly accounted for in the ground spectra specified in references 1 and 8. After further studies, as documented in Reference 9, the staff concluded that the site specific spectra specified in Reference 1 and 8 were still appropriate.

Although the analyses performed by the licensee prior to May 1982, were based upon more conservative Regulatory Guide 1.60 free-field ground spectra anchored at 0.12g horizontally, the primary coolant loop, piping, pipe supports, mechanical equipment, electrical equipment (including the effects of slab flexibility) and masonry walls will be evaluated using the Site Specific Spectra. Structures previously evaluated using the Regulatory Guide 1.60 free-field ground spectra anchored at 0.12g horizontally are not planned to be reevaluated. Either the Site Specific Spectra or the more conservative Regulatory Guide 1.60 free-field ground spectra anchored at 0.12g horizontally are appropriate for specification of free-field ground motion at the Big Rock Point site.

2. Justification for Continued Operation

Per the requirements of the NRC 50.54(f) letters (Refs. 1 and 2), the licensee provided information supporting continued operation as contained in its letters dated February 23 and April 25, 1979; February 13, March 31, and October 10, 1980; and July 27, 1981; and the meeting summaries dated August 7, 1979 and June 22, 1981. In addition, the staff and its consultant (Professor W. J. Hall of the University of Illinois) visited the site on June 30, 1981, to evaluate the seismic resistance of the facility. Based upon staff review of this material, the NRC Safety Evaluation Report (SER), to allow for continued operation of the Big Rock Point plant until completion of the seismic reevaluation program, was issued on September 29, 1981 (Ref. 10).

The conditions imposed in the September 29, 1981 SER were that:

"(1) results of seismic analysis are submitted for NRC review on the schedule specified in your July 27, 1981 letter; and

(2) in case of any modifications shown to be necessary as a result of the seismic analysis which are not implemented by January 1, 1983, the schedule for implementation and additional justification for continued operation over the period of this implementation are to be submitted and will be reviewed on a case by case basis."

In a letter dated August 26, 1981, a ten volume D'Appolonia report, Revision 1 dated August 1981, was submitted by the licensee which provided analyses results for the safety-related plant structures and the primary coolant loop piping. However, as discussed previously, the remaining analyses for safety-related masonry walls, piping, equipment and components have not been provided. By letter dated August 5, 1982 (Ref. 11), the licensee indicated that completion of piping analysis and the design of support modifications will not be completed until about 12 to 18 months following resolution of issues related to the August 1981 structural analyses. Evaluations of equipment nozzles would be completed following relevant piping analyses. The electrical equipment anchorage analyses will not be completed until about 4 months following resolution of the structural analyses concerns. In addition, as discussed previously, no program for the evaluation of electrical cabinet structural integrity has yet been planned, nor has their masonry wall evaluation been provided. The licensee in Reference 11 indicates that, although the above schedules are far in excess of those delineated in Reference 10, "the original bases for continued operation continue to be as valid today as they were when the staff SER (Ref. 10) was written. From the standpoint of overall plant safety, therefore, the seismic risk remains very low and operation continues to be justified."

Reference 10 indicated that the criteria and analytical techniques described in the referenced information up until July 27, 1981, stated criteria and analytical procedures for structures that were generally acceptable to the staff. Further, the inadequacies associated with the junction of the 4" cross-tie on the reactor coolant loop and overstressing of steel bracing and column bases in the service building complex, as identified by the licensee, were determined to not require immediate resolution because of design conservatisms and conservatism in the 0.12g Guide 1.60 ground spectra used as input in the analyses, and the low probability of an earthquake during the period of continuing analysis coupled with the small consequences of an accident.

3. Review of the Seismic Reevaluation Program Plan

A program plan, including descriptions of criteria, scope, analytical procedures, modeling techniques and, detailed schedules for completion of all seismic analysis and implementation of any required modifications, has not been provided by the licensee. Volume I of the ten volume August 1981 D'Appolonia report regarding the analyses of certain portions of the NSSS and the safety-related plant structures was reviewed in lieu of a formal program plan. The The results of this review were documented in a January 19, 1982 NRC letter to the licensee. In addition, several meetings were held with the licensee in the subsequent months to discuss the August 1981 D'Appolonia analyses. Licensee responses to issues raised in January 19 and July 27, 1982 NRC letters and the subsequent meetings were provided by letter dated September 21, 1982. This letter indicated that two key issues would not be addressed until later. Preliminary criteria and vague schedules for the licensee's evaluations of piping and mechanical equipment are described in Refs. 6 and 11, respectively. Electrical equipment anchorage evaluations are yet to be finalized, and details of the electrical equipment structural integrity evaluations have not been provided. Therefore, the staff concludes that the program plant is incomplete. Ref. 11 indicates that the currently anticipated completion date for analyses is late 1983 to early 1984. Schedules for the implementation of any modifications have not been provided.

4. Review Scope

The scope of the reevaluations was specified in the August 4, 1980 and April 24, 1981 NRC 10 CFR 50.54(f) letters (Refs. 1 and 2) to include those structures, systems and components necessary to assure, both during and after a postulated seismic event:

1. The integrity of the reactor coolant pressure boundary.

2. The integrity of fluid and electrical distribution systems related to safe shutdown and engineered safety features, and

3. The integrity and functionability of mechanical and electrical equipment and engineered safety feature systems (including containment).

The resolution of issues related to the functionability of mechanical and electrical equipment was later deferred to the Unresolved Safety Issue (USI) A-46.

5. Review of Reevaluation Criteria and Scope Proposed by the Licensee

As discussed in previous sections, the licensee has not provided complete information regarding the criteria and the scope of the evaluations planned for piping, and electrical and mechanical equipment and components. The broad scope of systems defined in the June 27, 1981 licensee letter are generally acceptable to the staff, however, details of the items of equipment and system boundaries to be included have not been provided. The scope of the August 1981 D'Appolonia analyses of structures, and buried piping are sufficient to include those necessary to provide for the integrity of the reactor coolant pressure boundary, and safe shutdown and accident mitigation systems.

The Reactor and Turbine Buildings, the Ventilation Stack, the Intake Structure and the Buried Fire and Intake Piping analyses were reviewed on an audit basis. Our evaluations of these are described below. The criteria for the majority of the evaluations of piping and mechanical equipment were provided by letter dated September 1, 1982 (Ref.6). (Criteria for portions of the main reactor coolant loop piping were contained in the August 1981 D'Appolonia reports.) Our evaluation of the criteria for these items is provided below. Issues related specifically to the integrity of the spent fuel pool and associated systems are being addressed as a part of the ASLB ASLB hearings on the proposed storage capacity expansion.

6. Review of Structural Evaluations

a. Reactor Building

NRC concerns regarding the August 1981 D'Appolonia analyses of the Reactor Building (Volume II) are identified in letters dated January 19 and July 27, 1982. An additional concern regarding the adequacy of the Reactor Building model to represent the construction joint separating the spent fuel pool from the internal concrete structure was identified in August 5 and 24, 1982 NRC meetings with the licensee and its consulants.

Responses to these concerns were provided by the licensee's letter of September 21, 1982 (Ref. 12), which stated that two major concerns, including the adequacy of the reactor building model, would not be addressed until later. Based upon our review of the above information, we have determined that the major outstanding issues discuss below exist. Therefore, we cannot conclude that the licensee's Reactor Building structural analyses are adequate to predict loads on highly stressed structural members (e.g., the spent fuel pool structure), and the responses of attached or supported piping, equipment and components.

discussed

(i) Inappropriate Consideration of Soil Structure Interaction and Structural Damping.

Reference 2 indicates that variations in soil springs must be considered in the development of structural responses. This variation is necessary to account for both uncertainties in the knowledge of soil properties (shear modulus, G) and the uncertainty associated with the soil spring stiffnesses themselves. Based upon a review of the geotechnical information, the staff concludes in Attachment 2 that (1) the uncertainty associated with the in-situ shear modulus for the glacial till alone is ±33% from the best estimate of G at SSE earthquake levels, and (2) the best estimate of the SSE strain level G is 75% of the low strain G (G max) rather than 90% as assumed by the licensee. Considering the geotechnical data alone, G for the till at SSE strain levels may vary from about .5G max to 1.0G max. Further variation in the soil spring must be accounted for considering the springs themselves. Therefore, a variation of ±50% of the licensee's best estimate springs (associated with 90% Gmax) should be used. This would imply a consideration of a range of 0.45 Gmax to 1.35G max. The lower bound of 0.45 Gmax is only 5% lower than the variation in the till alone. There should be less uncertainty associated with the underlying limestone than the till for the low soil stiffness. Therefore, it is the staff's judgement that soil springs based upon reducing the licensee's best estimate G by 50% for the entire soil/rock column should be adequate to cover both uncertainty in the soil properties and soil springs.

An independent confirmatory analysis of the containment shell was performed by the staff consultants (LLNL) and is presented as Attachment 3. The above range of soil spring variation is also supported in this analysis. In addition, based upon the overall stress levels predicted in the containment shell and Reactor Building concrete internal structure from those members reported by the licensee as being the most highly stressed, structural dampings of about, at most, one-half of those assumed by the licensee are appropriate. Given the high damping introduced in certain modes by the soil springs, uncertainties associated with structural damping, and the licensee's analytical results considering 0.12g Regulatory Guide 1.60 horizontal free-field spectra, damping of this lower order would also be appropriate when using the Site Specific Spectra.

The licensee performed a limited sensitivity study for the Reactor Building to address the above concerns and presented results in an April 1982 D'Appolonia Report entitled "Parametric Study-Soil Structure Interaction". Only floor spectra were generated at nodes 650, 652 and 661 of the model shown in Figure 1. These are points at which torsional effects may be maximized and may not be representative of motions on the internal structure or at containment penetrations. No stresses or member forces were addressed.

The licensee's parametric study presented results for varying soil springs and damping separately. For the study of varying structural damping, the licensee also used higher soil damping then in their initial analyses which tended to compensate for the decreased structural damping. In the range of 2 to 3% damping (representative for much attached piping and equipment), the spectra presented by the licensee indicate that in the amplified regions of the floor spectra, exceedances of their broadened spectra of typically 10% are possible for variations of both the soil springs and the damping levels, independently. However, this study is too limited to draw general conclusions, especially considering that Attachment 3 indicates that about a -40% and +100% variation in soil shear modulus and spring constant leads to consistent increases in peak shell stresses of about 40 to 85% above those predicted for the best estimate case. Attachment 3 concludes that the containment is still structurally adequate for each case due to the low magnitudes of the stresses. However, this conclusion is not generally applicable, especially to the spent fuel pool structure, the foundation stability analyses, and the analyses of attached piping, equipment and components.

(ii) Inadequate Consideration of Construction Joints

The "stick" model used for the analysis of the Reactor Building concrete structure was derived assuming that the spent fuel pool structure would behave monolithically with the concrete internal structure around the reactor and the steam drum enclosure, as indicated in Figure 1.

Figure 2 indicates the general arrangement of these items and illustrates the expansion joints between the steam drum and reactor enclosures and the spent fuel pool. These were not evaluated in the initial D'Appolonia Analyses. As indicated in Figures 3 through 5, these details were clearly illustrated on at least 4 of the construction drawings. Figure 3 illustrates that the only connection between the steam drum enclosure and spent fuel pool structure is a vertically acting bearing connection between the steam drum enclosure wall near the top of the spent fuel pool south wall and a thin slab connection at the floor of the spent fuel pool. Figure 4 illustrates that the only connection between the reactor enclosure wall and the east spent fuel pool wall is through a predominantly vertical acting shear key. By inspection, the behavior assumed in the Reactor Building model is at least questionable. In addition, such detailing is not unreasonable for a facility which was initially designed with minimal lateral load consideration. In response to

NRC concerns, these connections now are being evaluated. The September 21, 1982 licensee submittal (Ref. 12) indicates that justification of their assumption is being prepared and will be submitted later. It is not mentioned whether or not similar situations exist elsewhere in the Reactor Building or other structures. No conclusions can be finalized until this later submittal is reviewed.

(iii) Lack of Consideration of Member Connection Capacities

The evaluations of structural member capacities in the August 1981 D'Appolonia analyses (except for evaluations of certain column base connections) were based on the assumption that the member connection capacities were sufficient to develop the ultimate member strength. Connection failures are the notorious failure modes for structures and would be suspect for a structure initially designed with minimal horizontal load considerations. (For example, the vertical rebar in the concrete walls connected to the bottom of the spent fuel pool slab extend only about 4 to 5 inches into the slab and the predominant concentration was placed on vertical load transfer at the construction joints discussed in Item (iii) above). In response to NRC concerns connection evaluations are now being performed. The September 21, 1982 licensee submittal (Ref. 12) indicates that this issue is to be addressed in a later submittal. The scope of these connection evaluations is not discussed but in meetings the licensee has indicated that a sampling approach was being considered. No conclusions can be finalized until the connection program and associated evaluations are reviewed.

(iv) Inadequate Consideration of Uncertainties in the Coupled Reactor Building/Reactor Coolant Loop Analyses

The licensee has analyzed certain portions of the NSSS integrally coupled with the Reactor Building structural model using a single three component ground time history as input (See Volume II of the August 1981 D'Appolonia Report). The results of such a coupled model analysis have associated with them the implicit assumption that the relative stiffnesses of the building and NSSS are precisely modeled. As discussed in items (i) and (ii) above, many uncertainties are associated with the structural response. In addition, there are general uncertainties associated with the modeling of structures, piping, equipment and components. The effects of uncertainties on the response of attached piping, equipment and components are normally accounted for through the use of broadened response spectra in their analyses. Alternatively, numerous coupled analyses could be performed varying the input time histories, and building and NSSS properties to determine maximum responses. The potential for the variations in response which can occur due to changes in frequencies of the building, NSSS or both can be easily ascertained by noting the jaggedness of the unbroadened floor response spectra present in the April 1982 D'Appolonia parametric soil structure interaction study. Small changes in frequency can lead to large increases in response. This has not been considered in the licensee's coupled analysis and could lead to the predicted responses at certain locations in the NSSS increasing up to several times that obtained from the single coupled analysis.

(v) Use of the Guyan Reduction Technique Without Adequate Substantiation

Ref. 12 states that the Guyan Reduction Technique was used in the Reactor Building analysis. This technique is used to reduce the number of degrees-of-freedom in a dynamic analysis in order to economize on analysis costs. NUREG/CR-1938, entitled "Reduction of Structural Degrees of Freedom" indicates that this technique may predict frequencies and mode shapes adequately, yet still lead to large errors in the determination of the magnitudes of structural response. Differences are most severe when a model contains elements with large differences in stiffnesses and or masses such as would be the case for the coupled Reactor Building/NSSS models. NUREG/CR-1938 was based upon limited study of relatively simple models. Current, soon to be published research work based upon an extension of this study indicates that errors for complex nuclear plant type structures can be much more severe than those described in NUREG/CR-1938. Therefore, the staff cannot conclude that the licensee's analyses incorporating this technique are adequate.

(vi) Development of "Stick" Model Member Properties

In Ref. 12 and at August 5 and 24, 1981 meetings with the licensee and its consultants, the licensee indicated that their "stick" model member properties were derived without adequate consideration of:

a) Eccentricities between shear centers and centers of mass unless there were obviously large eccentric masses;

b) Shear center locations accounting for the nonuniform shear distribution for wall sections forming an "open section;"

c) Overall bending stiffesses assuming that walls may not be fully effective in bending such as for flanges of an I-beam; and

d) Connectivity at certain elevations of the structure to support the "rigid diaphram" assumption (e.g., at the top and bottom of the spent fuel pool).

Improper consideration of the above can lead to variations in structural response as discussed in NUREG/CR-2015, Volume 5, entitled "Seismic Safety Margins Research Proram - Phase I Final Report - Major Structure Response (Project IV)." The licensee has not substantiated that consideration of the above would lead to an insignificant nonconservative changes in predicted responses.

(vii) Interaction of Biaxial Shear in Concrete Members.

In Ref. 12, the licensee contends that ACI 349-76 recommends that biaxial shears are to be considered independently. This is not the case. Section 1.1 of the ACI 349-80 Code Commentary (as in ACI 349-76) indicates that "Some special structures involve unique problems not covered by the Code."

The ACI Code shear allowables are based on essentially 2-D testing (not 3-D testing). Consideration of biaxial shear is a 3 dimensional problem not explicitly addressed in the Code. Also, the Code does address that in-plane membrane stresses must be considered simultaneously with punching shear and does acknowledge that biaxial moments on beams and columns must be evaluated simultaneously. The Code addresses these cases explicitly, since these conditions are normally encountered. Since large biaxial shears are not generally encountered in structural elements, it can be understood that they they are not explicitly addressed in the Code. Ref. 12 indicates that shear stresses are low in what is contended to be the critical element of the Reactor Building (the east spent fuel pool wall) when biaxial shear is considered simultaneously. However, they do not support that this element is critical considering all members in all structures, nor is the situation addressed for connections. It is the staff's position that the simultaneous actions of bixial shear must be shown to be acceptable for all concrete members and connections.

(viii) Inappropriate Calculation of Factors of Safety for Overturning Resistance

The calculation of the factors of safety against overturning assumes that the overturning resistance is provided by the couple consisting of the vertical forces acting through the center of gravity of the structure and an equal and opposite force acting at the outermost point of the foundation. The lever arm is taken as the distance between the two forces, assuming rigid body behavior for the structure. This calculation ignores that: (1) the compliance of the soil will result in a pressure distribution which will have a resultant lever arm less than that assumed; and 2) the soil or the foundation structure may fail before the forces of the magnitude implied by this factor of safety could be applied. Therefore, the factors of safety quoted from this static calculation are too high. Given that: 1) soil pressure distributions were investigated for the calculated SSE loads and indicated substantial margins exist with respect to the licensee's specified allowable soil bearing pressures; and 2) the licensee's calculated factors of safety against overturning are substantially in excess of the required minimum of 1.1; overturning should not be a problem as long as the licensee verifies that the foundation structural elements are adequate to resist forces in excess of those applied for the SSE condition, and demonstrates the adequacy of previous analyses.

6. Turbine Building

The Turbine Building consists of a steel frame structure housing the generator and safety related plant equipment, a large concrete pedestal near its center supporting the generator, and a concrete steam pipe tunnel. This building is structurally connected to the Service Building and the liquid radwaste vault structures. The analyses of this complex is presented in the August 1981 D'Appolonia Report, Volume III. NRC concerns regarding these analyses were formally identified in letters dated January 19 and July 27, 1982, and in NRC meetings with the licensee and its consultants on August 5 and 24, 1982. Most of the concerns applicable to the Reactor Building analyses are also applicable to the Turbine Building analyses and are described below.

(i) Inappropriate Consideration of Soil Structure Interaction and Structural Damping

The uncertainties associated with the soil springs are discussed for the Reactor Building (SER Section IV.B.6.a.i) and are applicable here. The licensee has not demonstrated that the variation of soil spring constants assumed in the Turbine Building Complex analyses would not adversely affect the structural responses (e.g., floor response spectra, and forces in members and connections between those parts of the structure where soil structure interaction is significant).

As with the Reactor Building Analyses, the licensee has not justified the assumed levels of damping in the structure. High stresses and overstresses (from the licensee's sampling of critical members) are somewhat localized. Per the staff criteria, stress levels in the overall structure must be sufficient to justify the use of any damping level. Modifications discussed in Ref. 12 to alleviate overstresses may further reduce stress levels. In addition, the reduced stress levels associated with the use of the Site Specific Spectra would imply a further reduction in overall stress levels.

The licensee's argument (See Ref. 12) that the neglect of soil damping would compensate for the assumed structural damping has not been substantiated. It is especially questionable since the August 5, 1982 NRC/licensee meeting, the licensee indicated a feeling that soil structure interacton was not significant for the Turbine Building Complex overall response.

(ii) Lack of Consideration of Member Connection Capacities

The analyses presented in the August 1981 D'Appolonia report evaluated only certain column base connection capacities. As was described for the Reactor Building (SER Section IV.B.6.a.iii) all other member connections were assumed to be capable of developing the ultimate member (both concrete and steel) capacities. The staff has identified this concern to the licensee. As discussed in SER Section IV.B.6.a.iii, this has not yet been addressed by the licensee; therefore, final conclusions cannot be reached.

(iii) Use of the Guyan Reduction Technique Without Adequate Substantiation

The discussions presented previously for the Reactor Building (SER Section IV.B.6.a.v) are equally applicable to the Turbine Building Complex analyses. The technique was employed in the analysis of this complex and large variations in the various member stiffnesses and masses are present for this structure. Therefore, the staff cannot conclude that the licensee's analyses of the Turbine Building Complex are appropriate.

(iv) Interaction of Biaxial Shear In Concrete Members

The discussions presented previously for the Reactor Building (SER Section IV.B.6.a.vii) are also applicable to the Turbine Building analyses and are not repeated here.

(v) Inappropriate Calculation of Factors of Safety for Overturning Resistance

The discussions presented previously for the Reactor Building (SER Section IV.B.6.a.viii) are also applicable to the Turbine Building analyses and are not repeated here.

c. Ventilation Stack

The Big Rock Point ventilation stack is a 240' reinforced concrete structure. Its evaluation is described in the August 1981 D'Appolonia analyses, Volume IV. The acceptance criteria for the concrete and rebar stresses are appropriate. However, certain areas of concern with respect to the licensee's analyses have been identified.

(i) Inappropriate consideration of Soil Structure Interaction and Structural Damping

As was discussed for the Reactor Building (SER Section IV.B.6.a.i), the licensee has not considered the effects of soil spring constant variation. Therefore, for the stack the licensee must vertify that a 50% reduction in the soil spring constants does not change the conclusion that soil structure interaction is negligible for the stack.

The damping assumed by the licensee of 10% of critical is not justified by the overall stress levels in the structure, as presented by the licensee. In addition, the licensee has not stated the percentage of the reported stresses that are contributed by the assumed full thermal gradient to verify that the thermal stresses are not contributing substantially to the total stresses presented. Given that the 10% damped 0.12g Regulatory Guide 1.60 horizontal ground spectrum envelopes the 5% damped Site Specific Spectrum (as shown in Figure 6) and the vector application of the forces from the assumed two horizontal directions of ground motion is about 40% more conservative than necessary for such a circular structure, it is judged that the general structural integrity of the structure is reasonably assured if thermal stresses do not contribute substantially to the overall stresses. However, the foundation stablility, the integrity of construction joint at the base of the stack, and the integrity of areas where openings are present at the base of the stack must be verified as adequate at assumed damping levels consistent with those asssociated with overall stress levels resulting from application of the Site Specific Spectra.

(ii) Inadequate Consideration of Construction Joints

The construction joint between the base of the stack and the concrete foundation has not been evaluated considering both shear and moment.

(iii) Inadequate Consideration of Shear at the Base of the Stack

In Ref. 12 the licensee indicated that maximum shear stress based upon an elastic shear stress distribution was acceptable when compared to a monolithic concrete shear stress allowable. The effects of cracking due to applied moment were not considered but must be.

(iv) Apparent Inconsistentency Between Resultant Stresses Due to Seismic and Tornado Wind Loadings

The August 1981 D'Apolonia report, Volume IV indicates concrete and steel stresses due to combined seismic, thermal and dead weight loads at elevation 650.2 ft. for the stack. The August 3, 1982 licensee submittal for Big Rock Point regarding SEP Topic III-2, Wind and Tornado Loadings, provides concrete and steel stresses due to combined wind and dead weight loads at about elevation 595.5' + 56.33', or 651.83 ft. For a given applied set of loads, resultant concrete and steel stresses should be essentially the same at each of these points given the very small difference in elevation. A comparison of applied forces and resulting concrete and steel stresses at these points for the two different cases is presented in Table 1.

Thermal stress contributions are not given for the seismic load combination. Figure 7 shows the vertical steel to be located outside the circumferential steel. Given that the stack varies in thickness between 6" and 8", the vertical steel should not be located any further from the outside surface than one-half the thickness, and is probably closer to the outside face. Per equation 16 on page E1-8 of the August 1981 D'Appolonia report Volume IV, the vertical steel stress due to thermal should be computed based upon:

$$\text{steel stress} = L\ (z-k)\ T_x E_s$$

The quantities, L, T_x, and E_s are positive for the licensee's assumed temperature of -30°F outside and 70°F inside. Assuming the steel to be located at one-half the thickness, z is one-half. Equation 14 on page E1-7 of the August 1981 D'Appolonia report Volume IV indicates that:

$$k=-pn + [pn(pn+2z)]^{1/2}$$

The SEP Topic III-2 licensee submittal indicates that pn=.0456 at El. 651.83'.

Therefore,

$$k=-.0456+[.0456\ (1.0456)]^{1/2} = .173$$

Since (z-k) is positive, thermal stresses would be additive to the steel stress from seismic and deadweight. If the steel were nearer the outer face, the steel thermal stress would increase.

Based upon Table 1, the seismic moment is about 20% less than the wind moment. The reduced deadweight load considering vertical seismic effects is about 10% less than for the full deadweight assumed in the wind analysis. Also, the wind analysis did not include any additional steel stress contribution due to temperature effects. Consideration of these factors does not appear to support the licensee's conclusion that steel stresses due to combined deadweight, seismic and thermal loads is about 60% less than that due to combined deadweight and wind loads. This difference would increase further if thermal effects were excluded. The adequacy of the stack seismic analysis must be verified in light of the above in order to draw conclusions concerning its seismic adequacy.

d. Buried Fire and Intake Piping

As discussed previously, this piping is relied upon for safe shutdown of the plant. The analysis of this piping is presented in the August 1981 D'Appolonia report, Volume IX. The intake line is a buried reinforced concrete pipe which provides the water from Lake Michigan to the fire system. The buried fire piping is of cast iron construction and buried at an average depth of 6 ft. Via phone conversations with the licensee, the cast iron is the brittle, gray cast iron, centrifugally cast, and not ductile cast iron. Joints are of the mechanical type with rubber gaskets within the joint between segments. Several concerns described below were identified in our audit review of the analysis.

(i) Inappropriate Consideration of Seismic Wave Velocity Relationships

The licensee's analysis assume that the shear wave velocity is approximately equal to one-half of the compression wave velocity. This implies a Poisson's Ratio of 0.33 for the soil. Based upon the licensee's cross hole survey, the licensee indicated that a Poisson's Ratio of about 0.45 is appropriate for the soil. This value would imply that the shear wave velocity is about .30 of the compression wave velocity. In addition, the cross hole measurements of shear wave and compression wave velocities imply ratios of shear to compression wave velocities ranging from about .5 to .3 near the surface, and decreasing with depth. It must be verified that the .5 is a conservative analysis assumption, or an appropriate range determined and evaluated. In addition, the analysis indicates that since the compressional component of the Rayleigh wave velocity is about 0.45 times the compression wave velocity, it will produce substantially higher stresses than the compression wave. No indication is given whether the compression wave was neglected, and if so, no basis is given for neglecting its contribution. This must be addressed in the licensee's evaluation.

(ii) Lack of Connection Capacity Evaluation

The adequacy of the connection capacities for both the fire and intake piping were not evaluated, considering any potential degradation over time. Both bolts and flanges must be evaluated. Also, aging effects on the gaskets must be considered.

(iii) Inadequate Substantiation of Stress Reduction Due to Gasket Flexibiltiy

The licensee has not provided detailed substantiation that the assumed decrease in stress due to joint flexibility is reliable. The effects of aging on the gaskets and backfill in the joints has not been addressed.

(iv) Inappropriate Use of ASME Code Stress Limits

The licensee's analyses of the gray cast iron pipe uses Level C allowable stresses per the criteria of the 1977 ASME Code, Section III, Subsection NC for Class 2 piping. Thermal stresses are treated separately from other induced stresses, assuming them secondary in nature. The ASME Code is formulated assuming ductile materials and does <u>not</u> endorse the use of cast iron. Therefore, it was necessary for the licensee to obtain the allowable stress from the 1967 version of B31.1. This methodology is unacceptable for cast iron pipe due to its propensity for brittle failure.

The licensee's stress limits for the reinforced concrete are stated to be based upon the 1977 ASME Code, Section III, Subsection NC. This reference to Section III of the ASME Code is erroneous since <u>no</u> criteria for reinforced concrete pipe are provided. In addition, this material in this application is not covered explicitly by the normal ACI 318 or 349 Codes.

(v) Lack of Justification for Assumed Temperature Changes

The maximum temperature change of 20°F and 5°F for the fire and intake piping, respectively, assumed in the licensees analyses has not been justified.

(vi) Inadequate Consideration of Building Differential Movement

The licensee's evaluations considered building motions derived from their seismic analyes of the structures. Normal settlements which have occurred and may continue to occur over time have not been considered. These additional loads on the buried piping must be included.

(vii) General Performance of Gray Cast Iron Pipe

Gray cast iron pipe has seen decreased usage since the early 1950's According to the Ninth Edition of the Metals Handbook and the engineering department of the Los Angeles Department of Water and Power, it currently has limited availability. The Metals Handbook indicates that its manufacture is expected to cease altogether. Its past use was dictated by its low cost and its elimination is dictated by its propensity for brittle failure in transportation, installation and service.

The Washington Suburban Sanitary Commission (WSSC) engineering department indicated extreme outdoor hot or cold temperatures as being the predominant cause of failures they experience. These temperature changes induce expansion and contractions in the ground. Failures most frequently occur either at joints or where rocks contact the pipe and lead to loss of function due to brittle failure analagous to the shattering of a clay vessel.

The engineering department of the Los Angeles Department of Water and Power indicated that the past experience of cast iron pipe subject to earthquakes of Richter magnitudes of 4 to 4.5 has been good. However, it was felt that at earthquake levels associated with a Richter magnitude of 5 to 5.5, the performance may be questionable. The predominant failure mode was described as failure of the connection, leading to the axial movements of the pipe segments failing the bell of the pipe through a "stabbing" action. This in turn leads to loss of pipe function.

Given the past experience with gray cast iron pipe, and the licensee's inadequate evaluation of this piping and the associated uncertainties discussed above, it is the staff's position that the fire loop will not serve as a reliable water source.

e. <u>Intake Structure</u>

The review of this structure, as evaluated in the August 1981 D'Appolonia report, Volume VIII, was initiated. However, the review was not completed due to the nature of the deficiencies identified in the review of the structures and buried piping discussed above.

Given the generic nature of many of the issues identified in these other reviews, based upon an audit review, the staff cannot conclude that evaluation of the intake structure is appropriate.

f. <u>Masonry Wall Evaluations</u>

The licensee has not submitted the criteria and evaluations for the Big Rock Point safety related masonry walls.

g. Criteria for the Evaluation of Piping and Mechanical Components

The licensee provided a preliminary copy of proposed criteria for the evaluation of piping and mechanical equipment via letter dated September 1, 1982 (Ref. 6). Analyses of portions of the NSSS piping were contained in the August 1981 D'Appolonia report, Volume II. Given the preliminary nature of the majority of the criteria and the substantitive issues identified in the review of the licensee's structural evaluations, only a preliminary review of the piping and mechanical component criteria was conducted. Based upon the review of the staff and its consultants, we have determined that there are significant concerns with the licensee's proposed criteria and that the criteria are not in conformance with the SEP acceptance criteria, as referenced previously. Details of the preliminary evaluation of the proposed criteria are contained in Attachment 4, entitled "Technical Evaluation Report - Big Rock Point Plant Seismic Design." In addition, the methodology for consideration of local structural flexibility is yet to be provided, as indicated in Reference 12. Further discussions with the licensee are required to arrive at a final criteria and to complete our review.

h. Electrical Equipment

The licensee has not submitted the criteria and evaluations for electrical equipment anchorages. As discussed previously, the licensee indicated that previous evaluations had not considered vertical slab fexibility which is significant in certain areas. These effects are now being evaluated by the licensee. In addition, a program for the evaluation of the entire electrical cabinet structural integrity has not yet been formulated by the licensee.

V.. SUMMARY AND CONCLUSIONS

The licensee has not provided a complete definition of the scope of the necessary piping, equipment, and component evaluations. Final criteria for these evaluations has not been provided by the licensee. Previous evaluations of certain portions of the NSSS piping are still to be redone and resubmitted to the NRC. Electrical equipment anchorages must be reevaluated considering local slab flexibility, as appropriate, before details of these evaluations are submitted. Many piping, equipment and component evaluations are yet to be performed. Finally, the staff review of the preliminary criteria proposed by the licensee for these evaluations has identified several areas of concern.

The licensee's current estimates for the earliest date of completion of the requisite piping, equipment and component evaluations is between late 1983 and early 1984. No schedule has been provided for the implementation of any required modifications. This schedule has been justified by the licensee based upon the low seismicity of the Big Rock Point site, coupled with the small consequences of an accident at the facility.

The staff in its review of the licensee's structural analyses and analyses of certain portions of the NSSS piping has identified several areas of concern with these analyses. The major concerns can be lumped into the following broad categories:

1. Failure to appropriately consider analytical uncertainties in a) the ability of models to depict response, b) the knowledge of assumed mass, stiffness and damping parameters, and c) the analytical techniques themselves;

2. Failure to evaluate weak links such as connection capacities; and

3. Failure to assure that the analytical models adequately represent as-built details which could substantially alter predicted structural response.

The licensee's evaluations of the Big Rock Point plant structures has indicated the following deficiencies with respect to the structures capability to withstand an earthquake having a zero period horizontal free-field ground acceleration of 0.12g and matching the USNRC Regulatory Guide 1.60 response spectrum:

- Two steel columns (Column Nos. E_A-8 and D_A-7) have been judged to have marginal buckling capacities in the passegeway area.

- A stress concentration probably exists in the Primary Coolant Loop (PCL) at the junctions of 4-inch diamter crows ties and the 24 inch diameter downcomer. Possible undesirable displacements are also present in this area.

- The potential for uplift at some of the steel column bases exists between the steel column and its base plate. This uplift is small and can generally be resisted by the existing footing and base plate connections. However, for Column Nos. J-1 and H-1 in the service building, the uplift force is significant and under the stipulated loading conditions may lead to failure at their column bases.

- Overstressing of some of the bracing in the northeast corner of the service building has been predicted.

The licensee has indicated that the above structural deficiencies will be corrected. However, the licensee proposed to modify the portion of the reactor coolant loop only if warranted after reanalysis using the site specific spectra and the previous analytical methodologies. While the staff agrees that these appear to be the "worst" areas, these modifications may not be sufficient, due to the concerns discussed above, to provide assurance of seismic qualification for free-field ground motion defined by the Site Specific Spectra.

Based upon the review of the analyses of the buried cast iron fire loop and reinforced concrete intake piping, we have concluded that the licensee's analyses are not adequate to demonstrate the adequacy of this piping. Further, this review indicates that such piping does not provide for reliable, seismically qualified water sources.

In discussions with the licensee, the licensee has indicated that it is economically unfeasible to perform the analyses required to quantify analytical uncertainty. The results of this audit review suggest that considerable detailed review is required.

The staff does believe that the facility possesses some inherent seismic resistance. However, insufficent analyses have been performed along with the implementation of corresponding required modifications, to allow the staff to conclude that there is assurance that the facility can safely withstand the occurrence of an SSE as defined by the site specific spectra. Based upon the low probability of an earthquake with significant ground motion at the site and the low radiological consequences of an accident (Refs. 10 and 11), the staff concludes that continued operation is acceptable pending completion of the integrated assessment. It is the staff's position that the licensee address the issues identified in this draft evaluation and provide other information addressing alternative approaches (e.g., use of dedicated portable emergency pumps for safe hot shutdown similar to that proposed by Yankee Atomic Electric Company for Yankee or Dairyland Power Cooperative for LaCrosse) which should be considered during the Big Rock Point Integrated Assessment.

VI. REFERENCES

1. Letter to D.P. Hoffman (CPCo), from D.G. Eisenhut (NRC), "RE: Big Rock Point," dated August 4, 1980)

2. Letter to D.P. Hoffman (CPCo), from D.M. Crutchfield (NRC) "SEP Topic III-6, Seismic Design Considerations - Big Rock Point," dated April 24, 1981.

3. Letter to D.J. VandeWalle (CPCo), from D.M. Crutchfield, (NRC), "Systematic Evaluation Program Position Re: Consideration of Inelastic Response Using the NUREG/CR-0098 Ductility Factor Approach," dated June 23, 1982.

4. Letter to D.J. VandeWalle (CPCo), from D.M. Crutchfield, (NRC), "SEP Topic III-6, Seismic Design Considerations, Staff Guidelines for Seismic Evaluation Criteria for the SEP Group II Plants," dated July 26, 1982.

5. Letter to D.J. VandeWalle (CPCo), from D.M. Crutchfield (NRC), "SEP Topic III-6, Seismic Design Considerations, Staff Guidelines for Seismic Evaluation Criteria for the SEP Group II Plants - Revision 1," dated September 20, 1982.

6. Letter to D.M. Crutchfield (NRC), from D.J. VandeWalle (CPCo), "SEP Topic III-6, Seismic Design Considerations," dated September 1, 1982.

7. NUREG/CR-1582, "Seismic Hazard Analysis," Volumes 2 through 5, dated October 1981.

8. Letter to all SEP Owners (Except San Onofre Unit 1) from D.M. Crutchfield (NRC), "Site Specific Ground Spectra for SEP Plants Located in the Eastern United States," dated June 17, 1981.

9. Letter to D.J. VandeWalle (CPCo), from D.M. Crutchfield (NRC), "Assessment of Possible Soil Amplification at Big Rock Point Site," dated June 30, 1982.

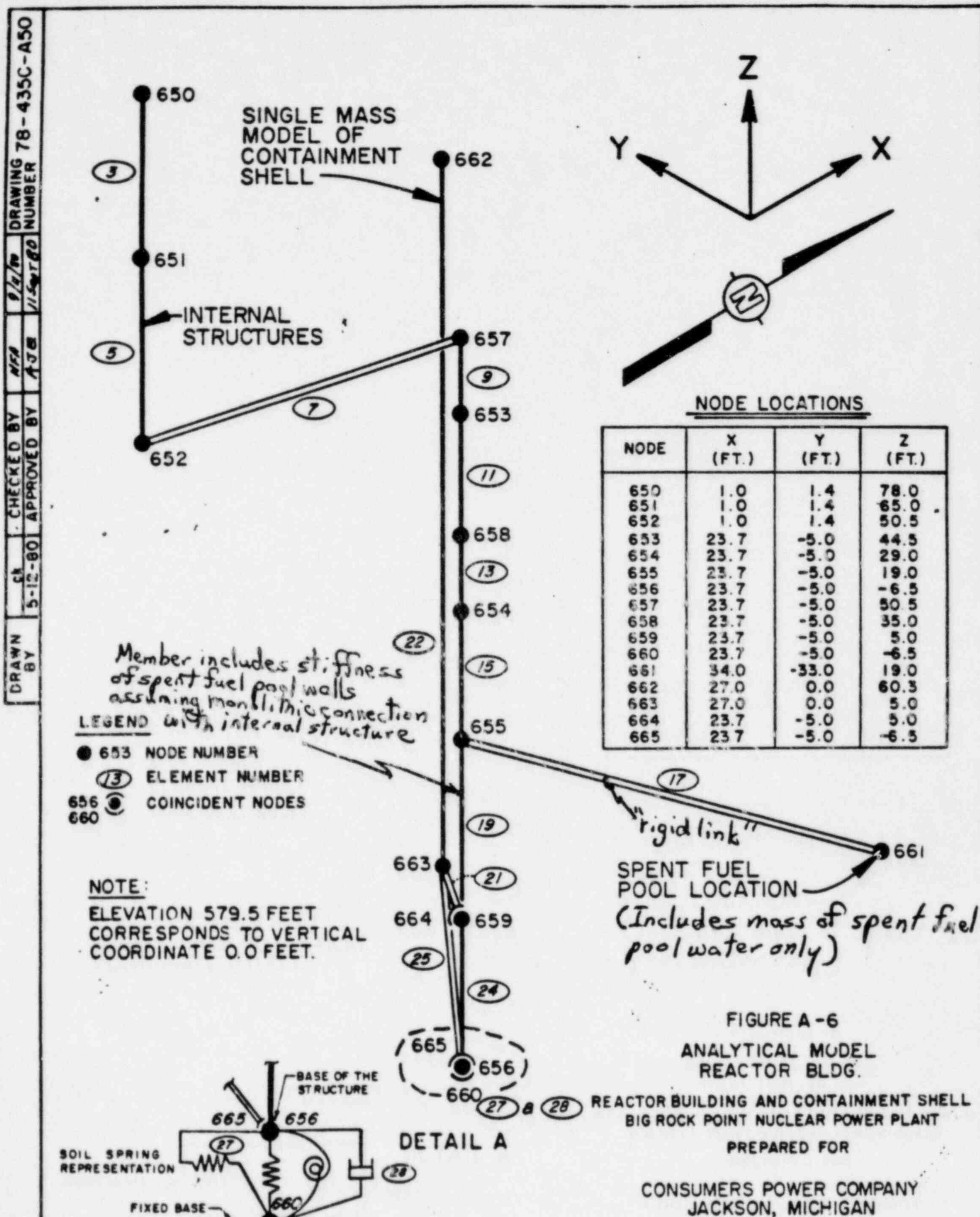
10. Letter to D.P. Hoffman (CPCo), from D.M. Crutchfield (NRC), "SEP Topic III-6, Seismic Design Considerations - Big Rock Point Nuclear Power Station," dated September 29, 1981.

11. Letter to D.M. Crutchfield (NRC), from R.A. Vincent (CPCo), "Big Rock Point Plant - SEP Topic III-6, Seismic Design Considerations," dated August 5, 1982.

12. Letter to D.M. Crutchfield (NRC), from D.J. VandeWalle (CPCo), "Big Rock Point Plant - SEP Topic III-6, Seismic Design Considerations," dated September 21, 1982.

TABLE 1: SEP Topics III-6 and III-2, Load and Stress Comparisons for the Big Rock Point Ventilation Stack

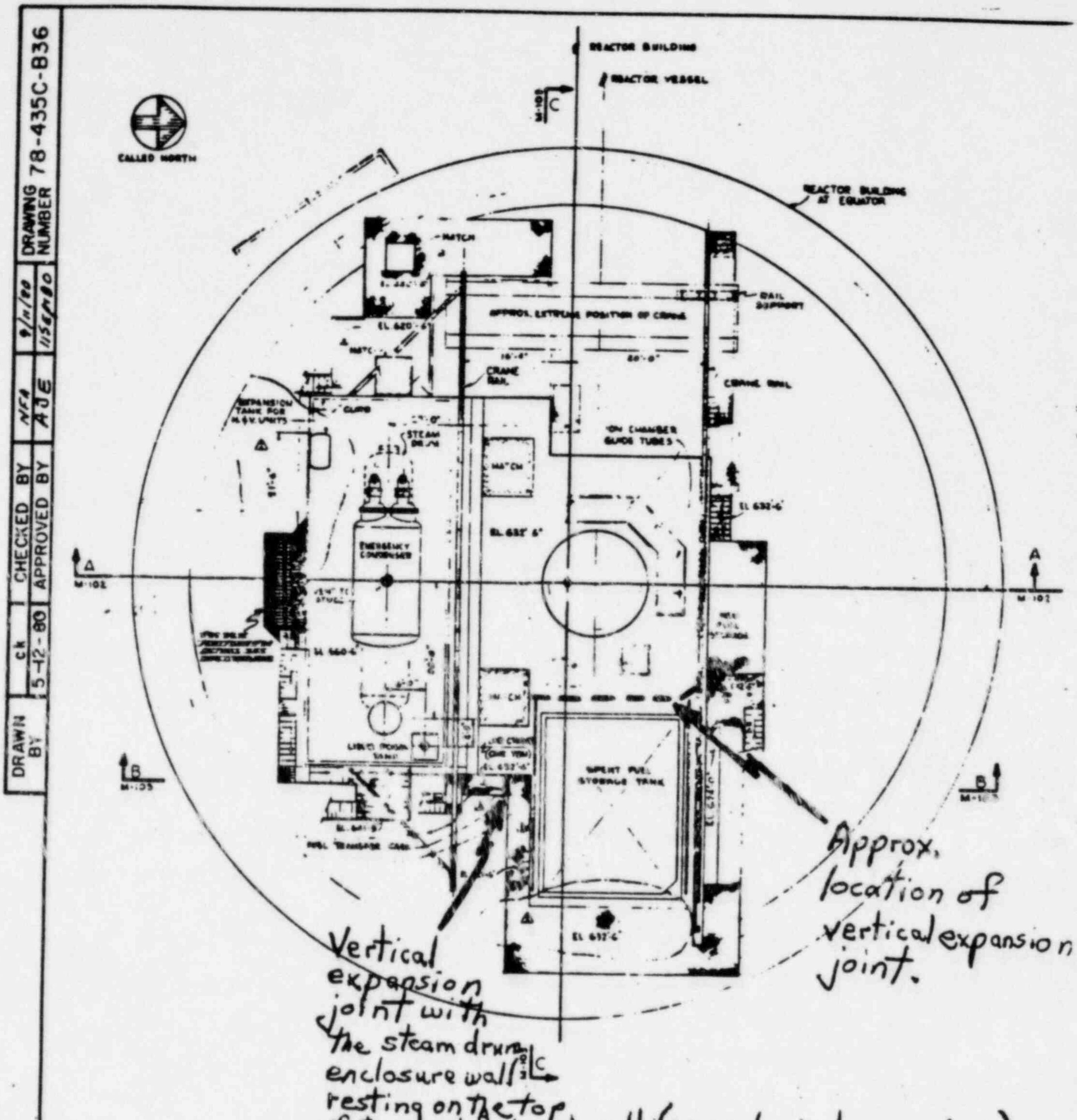
LOAD COMBINATIONS	TOTAL MOVEMENT	TOTAL AXIAL LOAD	CONCRETE COMPRESSION STRESS	STEEL STRESS	STEEL THERMAL STRESS CONTRIBUTION
Deadweight and Seismic and Thermal	1.4 (2830)= 3962 ft-kips	364-.4(82.4)= 331 kips	950 psi	9210 psi	Additive to the total from seismic and deadweight
Deadweight and Wind	5,011 ft-kips	364 kips	911 psi	24,500 psi	0 psi



NODE LOCATIONS

NODE	X (FT.)	Y (FT.)	Z (FT.)
650	1.0	1.4	78.0
651	1.0	1.4	65.0
652	1.0	1.4	50.5
653	23.7	-5.0	44.5
654	23.7	-5.0	29.0
655	23.7	-5.0	19.0
656	23.7	-5.0	-6.5
657	23.7	-5.0	50.5
658	23.7	-5.0	35.0
659	23.7	-5.0	5.0
660	23.7	-5.0	-6.5
661	34.0	-33.0	19.0
662	27.0	0.0	60.3
663	27.0	0.0	5.0
664	23.7	-5.0	5.0
665	23.7	-5.0	-6.5

Figure 1: Reactor Building "stick" model.



KEY PLAN

BELOW EL. 674'-0"

REFERENCE:

BECHTEL DWGS. 3159 M-102 REV. C (NEW DWG. NO. 0740G40102) AND 3159 M-104 REV. B (NEW DWG. NO. 0740G40104)

Figure 2: General relationship of spent fuel pool with the rest of the Reactor Building.

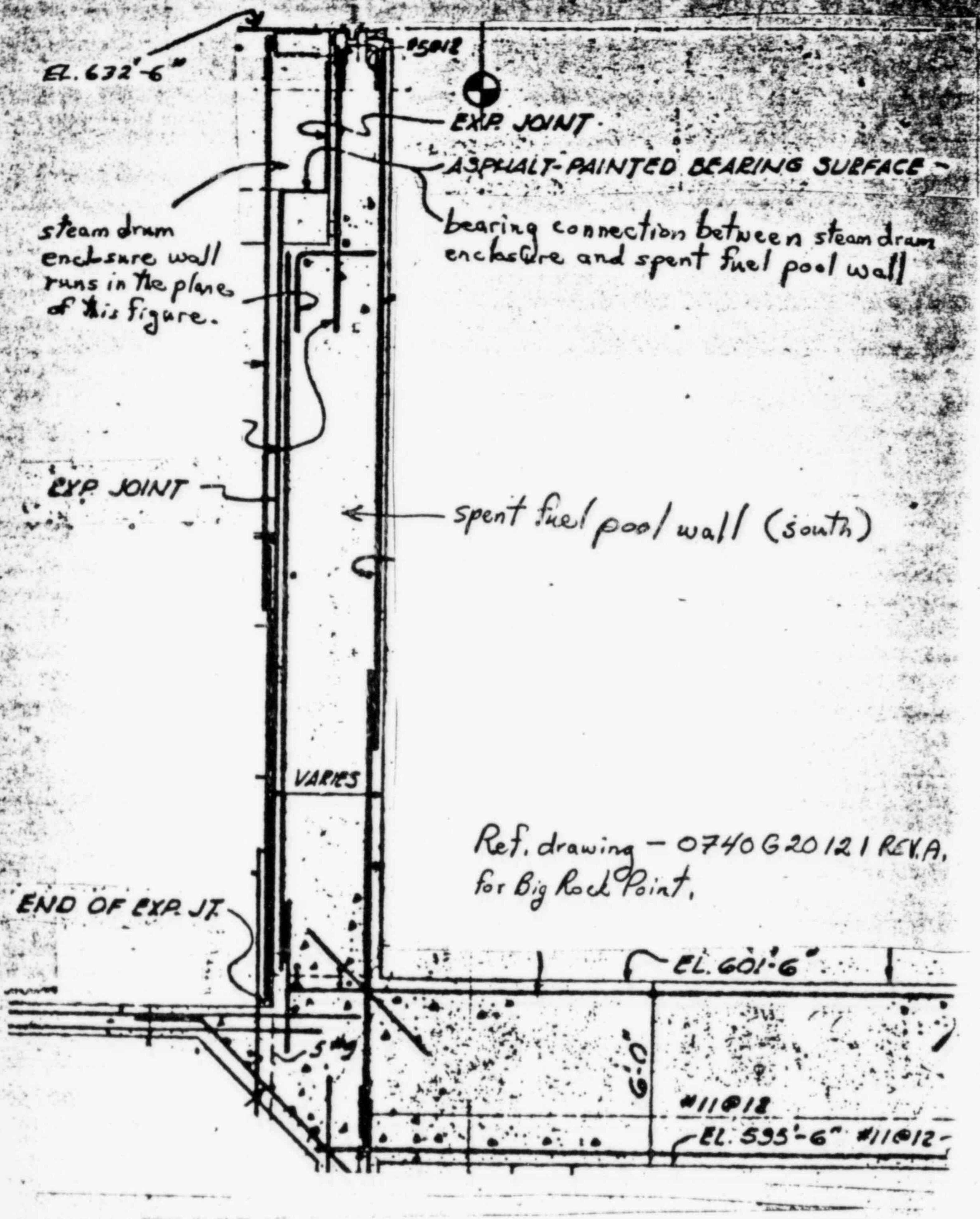


Figure 3: Reactor Building internal structure construction details.

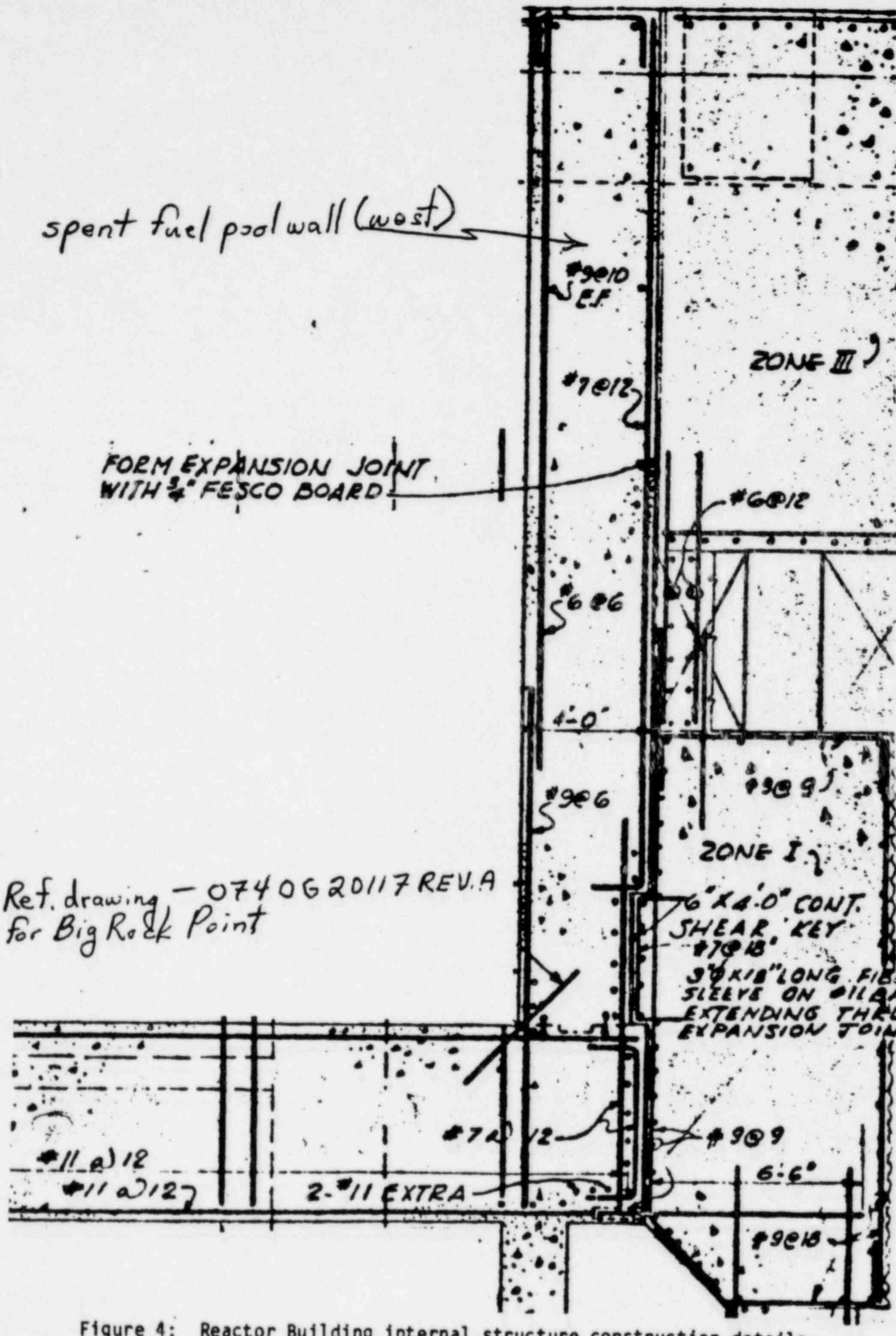
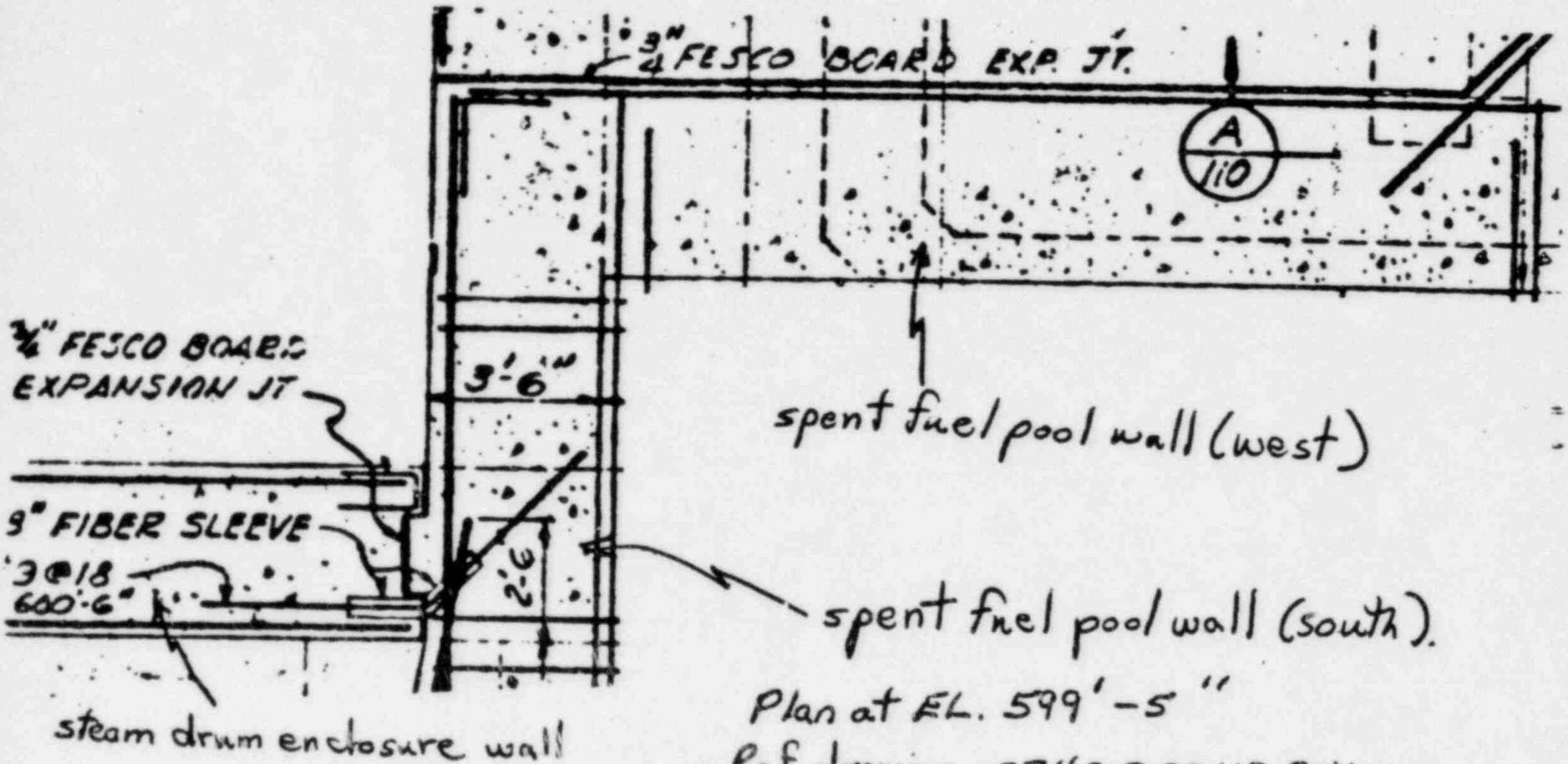
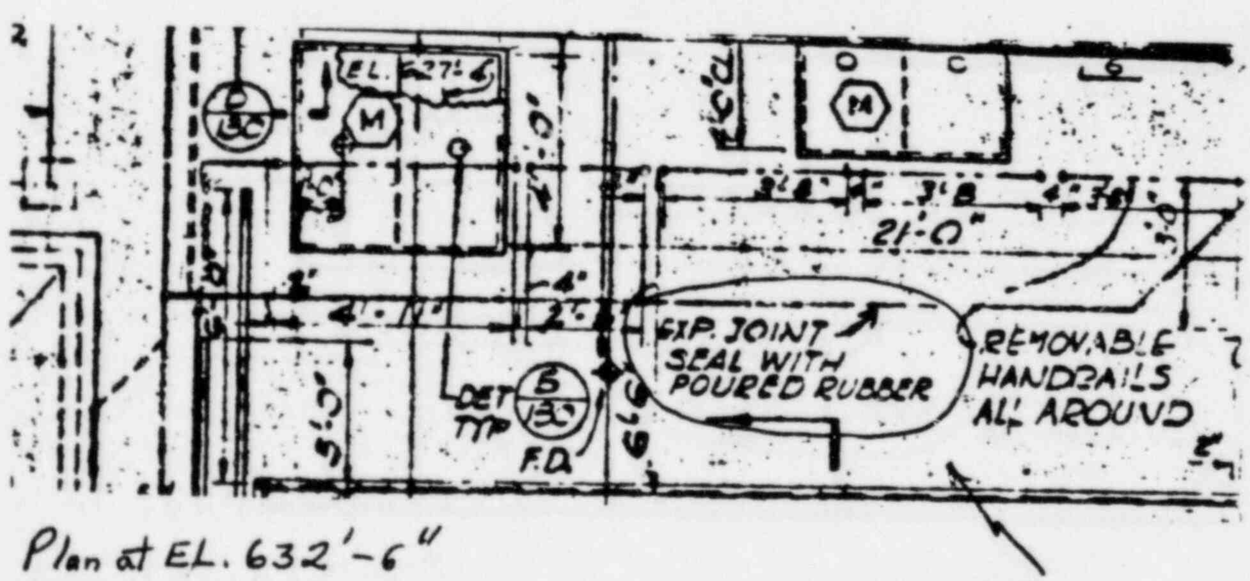


Figure 4: Reactor Building internal structure construction details.



Plan at EL. 599'-5"
Ref. drawing. 0740 G 20113 REV. A
Big Rock Point



Plan at EL. 632'-6"
Ref. drawing 0740 G 20115 REV. A
Big Rock Point

Figure 5: Reactor Building internal structure construction details.

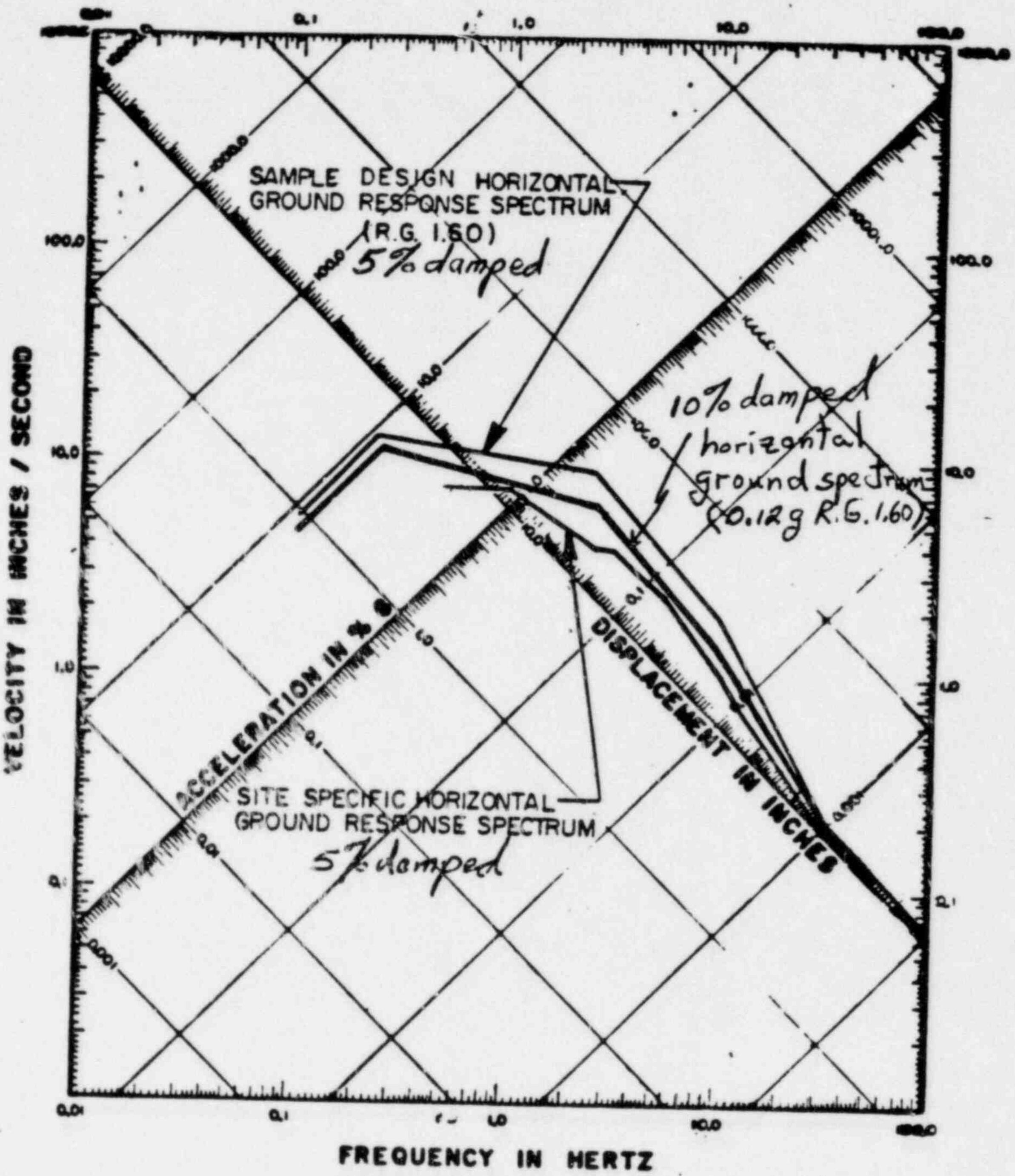


Figure 6: Comparison of 10% damped 0.12g R.G. 1.60 horizontal free-field ground spectrum and 5% damped Site Specific Spectrum.

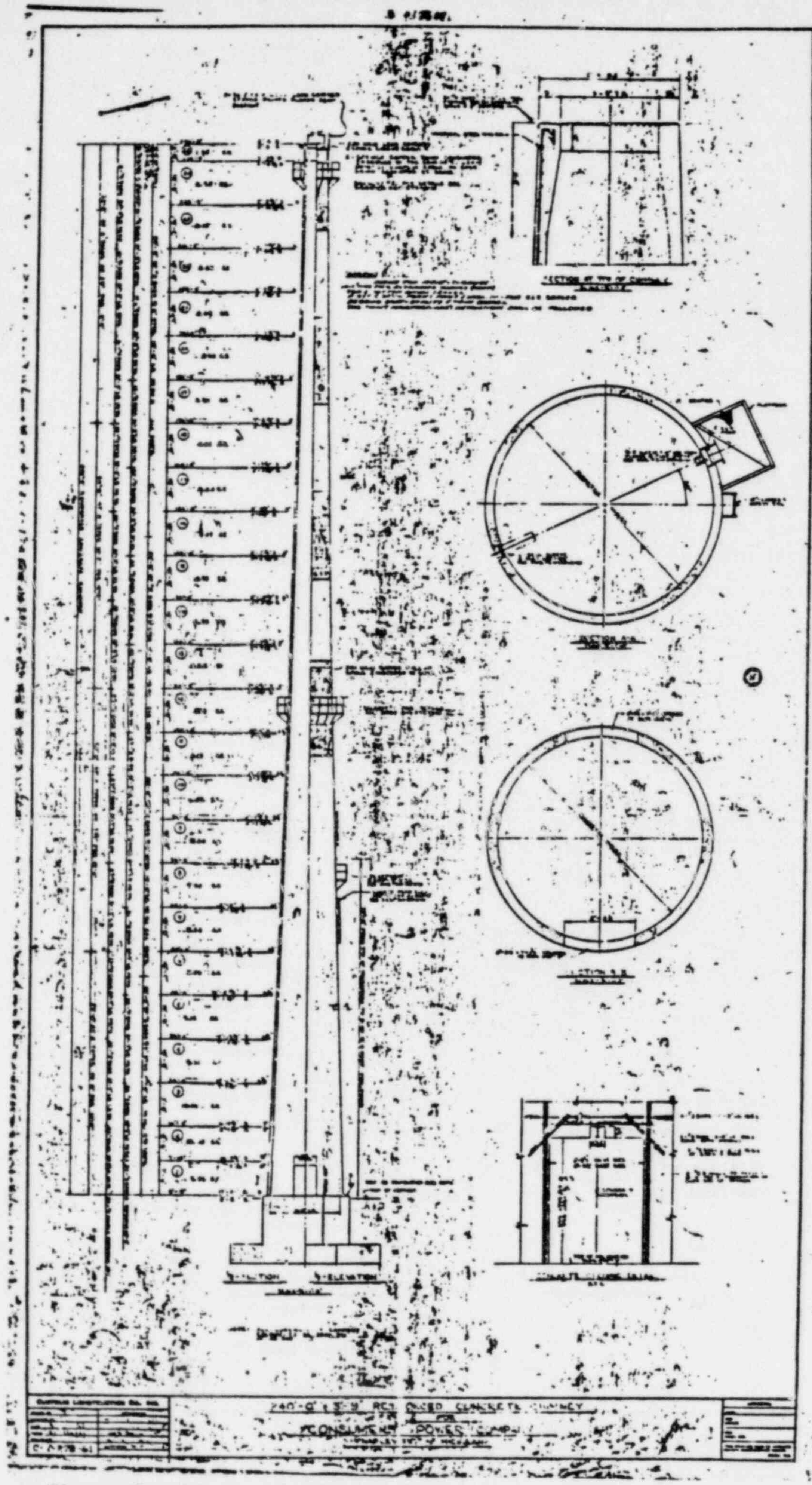


Figure 7: Big Rock Point concrete ventilation stack details.

Attachment 1

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DRAFT

SEISMIC DESIGN BASES AND CRITERIA FOR BIG ROCK POINT NUCLEAR GENERATING STATION

SYSTEMATIC EVALUATION PROGRAM

prepared for

Nuclear Test Engineering Division
Lawrence Livermore Laboratory
Livermore, California

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EDAC

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1. INTRODUCTION

This report presents the results of an evaluation of the seismic design bases used in the design and analysis of the Big Rock Point Nuclear Generating Station. The plant is located at Charlevoix, Michigan, and is operated by Consumers Powers Company of Michigan. The evaluation was focused on ascertaining the seismic analysis methodologies and code requirements. The Final Hazards Summary Report (Reference 1) and plant related communication dockets were the principal source of information.

The site is located on the northern shore of Lake Michigan in Charlevoix County, Michigan (Figure 1-1). Near the shore, a belt of limestone bedrock is either at the ground surface or covered with thin unconsolidated glacial and lacustrine deposits. Big Rock Point (BRP) is a Boiling Water Reactor of 70 MWe capacity. The plant went into commercial power production in December, 1962. The Nuclear Steam Supply System (NSSS) was designed and supplied by the General Electric Company, and the Bechtel Corporation was the architect-engineer.

The principal structures include:

The 130-foot dia spherical containment vessel

The turbine building

The water intake facility

The 240-foot stack

Waste storage vaults

The containment houses the reactor, recirculation network, pumps, steam drum, fuel pool, and systems for heat removal. The turbine-generator and other components are located in the turbine building.

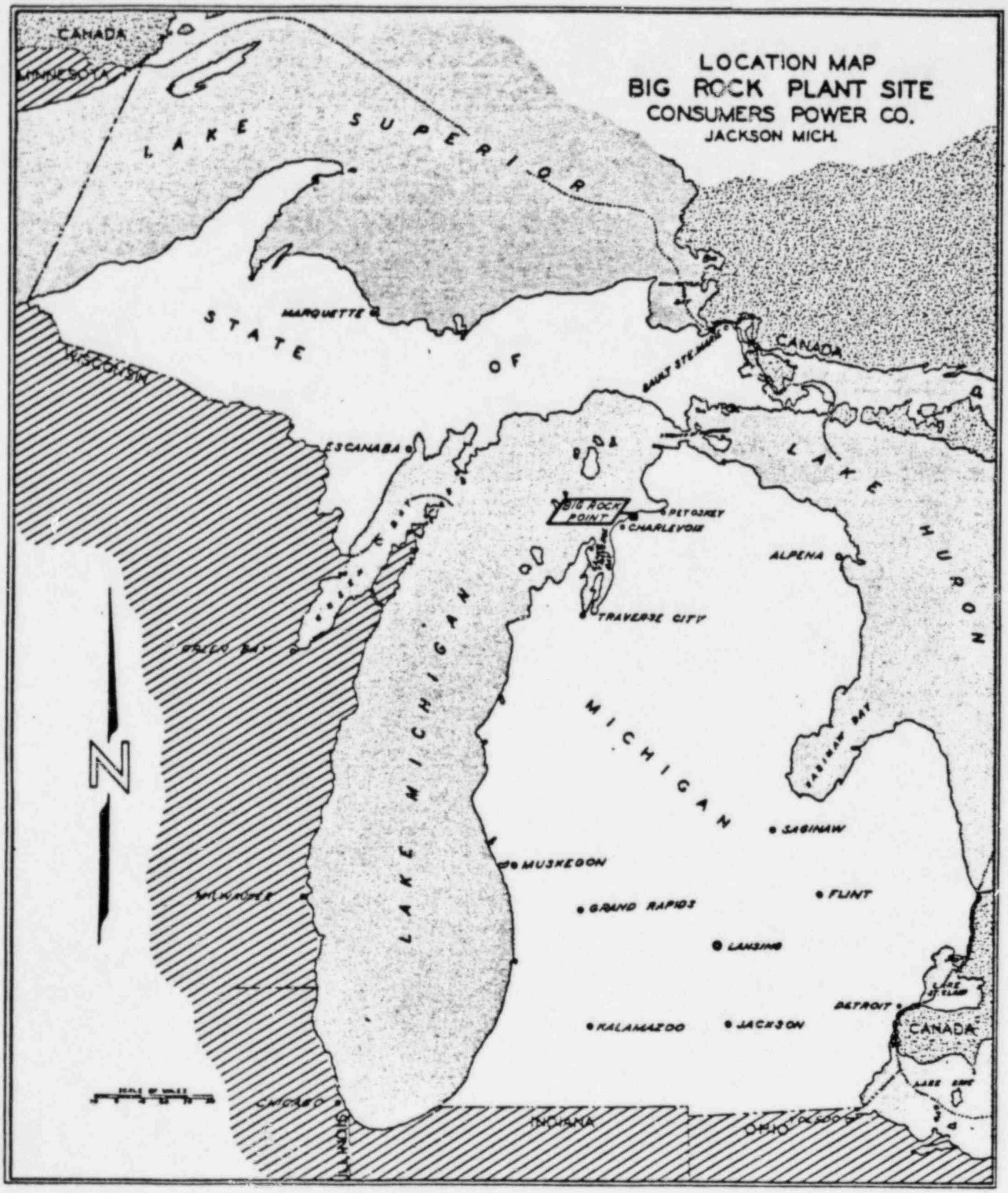

FIGURE 1-1: LOCATION MAP

2. GEOTECHNICAL DATA

2.1 GEOLOGY

Big Rock Point is located on the northern shore of Lake Michigan in Charlevoix County. The lake shore consists of scattered limestone outcrops alternating with short stretches of beach. Topographically, the region within a five-mile radius of the site location can be divided into two categories: (i) a zone of low relief which was once submerged beneath the water, (ii) upland surface which rises 40 to 60 feet above the general plain surface.

The site lies in a belt of limestone of lower Paleozoic age. The exposed rocks along the lake shore are part of the Transverse Group of Devonian age. Near the shore, the bedrock surface is either at the ground or covered by thin, unconsolidated glacial deposits. Farther inland, the bedrock is more deeply buried beneath the till plain.

Borings at the site were made to a depth of 40 feet. The limestone bedding planes are nearly horizontal and the regional dip is to the southeast toward the center of the Michigan Basin. Generally, the soils in the site area have weakly developed podzol profiles and are well drained.

2.2 SEISMOLOGY

Included in this section are the historical seismicity records and seismicity evaluation for the site at the time of the FHSR (Reference 1). Table 2-1 is a list of historical earthquake records (References 2 and 3) and Figure 2-1 reproduces that part of the map from Reference 2 to show the epicenters and intensities of historical earthquakes.

The importance of earthquakes to plant design was independently assessed by the Bechtel Corporation, and their summary statement was:

An investigation of the seismic history indicates that this is a region of low seismic activity. The Coast and Geodetic Survey Publication, Serial 609, Earthquake History of the United States, lists earthquakes in the Michigan area as shown below. All of these are classified as intermediate or minor. The nearest recorded earthquake was the one centered near Menominee, approximately 110 miles from the plant site.

Based upon the low occurrence of earthqukes, it was concluded that special seismic design features were not necessary. However, all structures were designed to resist seismic loading. Structural design of the plant complied with the Uniform Building Code (Reference 4), with the horizontal force based upon the Zone 1 factor or higher.

TABLE 2-1
HISTORICAL EARTHQUAKES

1804, August 20. Near Chicago. Felt over an area of 30,000 square miles.

1872, February 6. Three small shocks near Wenona, Michigan (east of Saginaw).

1877, August 17. Small shock in southeastern Michigan.

1883, February 4. Small shock near Kalamazoo, Michigan. Felt over 8,000 square miles.

1905, March 13. MM intensity 5 at Menominee, Michigan.

1905, July 26. The Earthquake History of the U. S. reports, "An earthquake which was apparently associated in some way with the peculiarly unstable conditions brought about by mining operation was felt all over the Keweenaw Peninsula, Michigan. It was heaviest at Calumet. There was a terrific explosion, chimneys fell with a crash, and plate glass windows broke. The explosion was heard far down in the mine. Felt at Marquette, Michigan."

1906, May 26. The Earthquake History of the U.S. reports, "At the Atlantic Mine on Keweenaw Peninsula the effects were such as might be produced by a great earthquake. Rails were twisted and there was notable sinking of the earth above the workings. Such effects were noted nowhere else, though at Madison, Wisconsin, there were three distinct shocks, at Lansing furniture swayed and at Muskegon along the lake shore dishes were upset and windows shaken. The area affected was about 1,000 miles in diameter."

1909, January 22. Small shock at Houghton, Michigan.

1909, May 26. MM intensity 7 in northern Illinois. Felt over an area 800 miles across.

1935, November 1. Timiskaming, Ontario. This is just out of the area, but it is listed as it is probably the largest shock felt in Michigan in historic times. Some slight damage in the Detroit area.

1943, March 8. Small shock in central Lake Erie.

TABLE 2-1 (continued)

1947, August 9. South-central Michigan. MM intensity 6 north of Coldwater. Felt over an area of 50,000 square miles.

The May 26, 1906, shock centering on the Keweenaw Peninsula was probably felt more strongly at Charlevoix than any other, but there seems to have been no damage except in the mining area of the Keweenaw Peninsula and this seems to have been related to the mining operation.

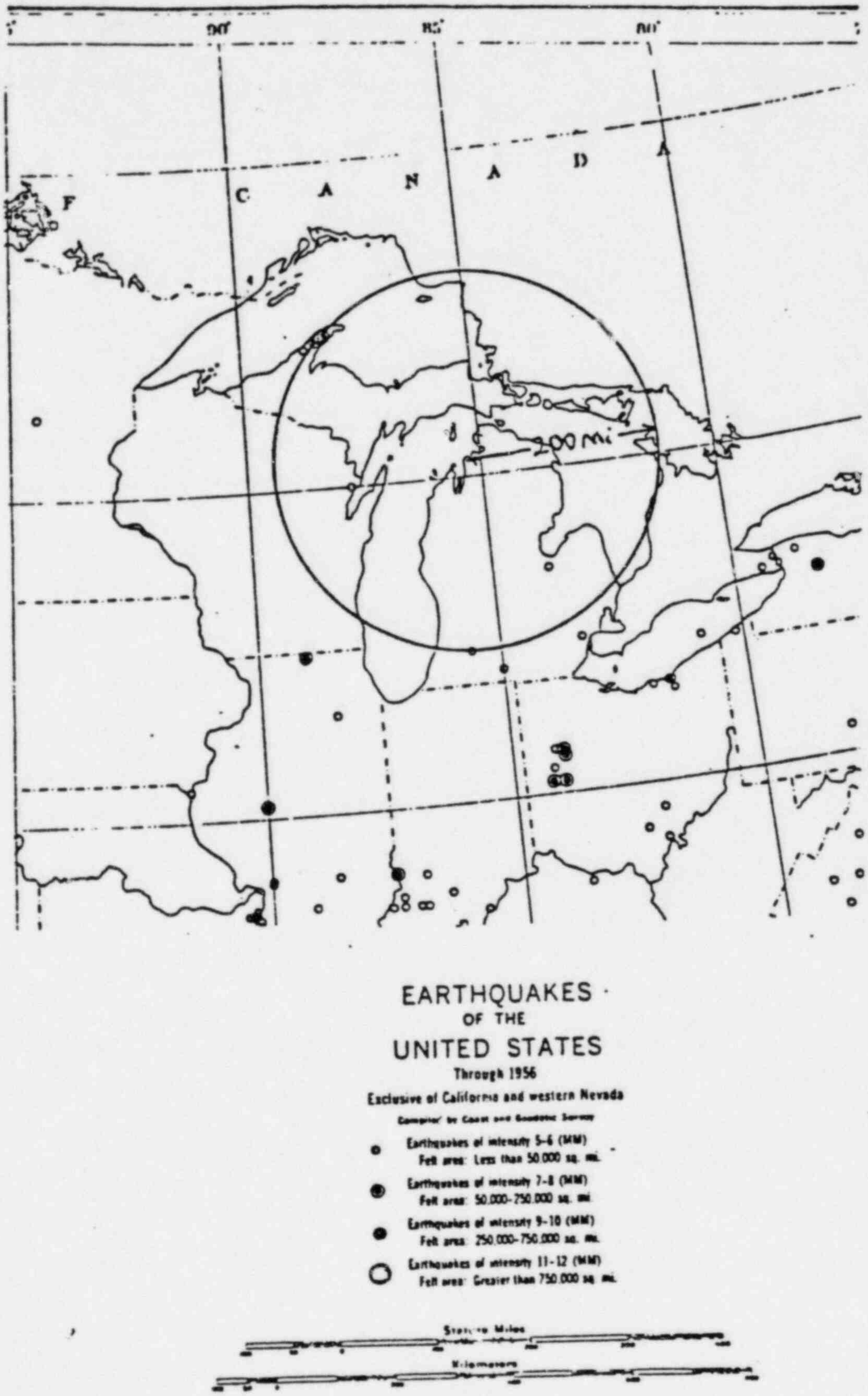

FIGURE 2-1: HISTORICAL EARTHQUAKES

3. SEISMIC CRITERIA

Big Rock Point (BRP) plant was not designed in accordance with today's criteria and standards. The grouping of the systems and structures was based upon their functional requirements with regard to safety of the plant as defined in ANS Guides and the Code of Federal Regulations (References 5 and 6). Therefore, the systems were not grouped into Seismic Category I or equivalent. The design of most of the facility was based upon the seismic requirements of UBC (Reference 4), and static analyses were performed. The seismic design bases of major structures are discussed in the following sections. The equipment and piping are presented in a subsequent section.

3.1 REACTOR CONTAINMENT VESSEL

The containment vessel is a spherical steel vessel of 130-foot diameter, which houses the NSSS, spent fuel storage pool, and emergency condenser, as shown in Figures 3-1 through 3-5. The lower segment of the steel is embedded in concrete, and the structure extends 27 feet below grade. The foundation consists of a combination of a 3-foot thick concrete mat and reinforced concrete footings from 38 feet to 8 feet below grade. The design parameters of the vessel are summarized in Table 3-1 and Table 3-2 lists the codes and standards used in the BRP design and evaluation.

The containment vessel was designed in accordance with UBC requirements (Reference 4) using a lateral, static load coefficient of 5 per cent of gravity. The siting and strength of the containment was controlled by the design wind load of 100 miles/hour, however, and not the earthquake forces. Design and construction of the vessel was in accordance with Reference 7, Sections II, VIII and IX, as modified by the applicable nuclear code cases. The shell is constructed of SA-201 Grade B, firebox quality steel produced to SA-300 specification.

3.2 THE INTERNAL CONCRETE STRUCTURES

The massive concrete structures were designed to serve as: (i) supports for the major equipment, (ii) attenuate the radiation from the reactor system to an acceptable level, and (iii) protect the system from postulated missiles. The lower segment of the concrete carries the spherical steel containment, and functions as a mat foundation as well. The concrete structure is coupled to the turbine building by the pipe tunnel. Figures 3-1 through 3-5 shown plan and sectional views of these structures.

The lateral concrete loads were determined from UBC (Reference 4) requirements and element design details and sizes were based upon ACI 318 (Reference 8). A seismic factor of 0.025 was used for the equivalent lateral coefficient. Attenuation of radiation was a controlling factor in determining the wall thicknesses.

3.3 OTHER STRUCTURES

Other major structures include the turbine building, the 240-foot high stack, water intake structure, control room and waste storage building.

All these buildings and structures were designed in accordance with Reference 4, again with a static coefficient of 0.025g, and constructed to comply with Reference 8. Snow, wind and seismic lateral loads were used in the static design analysis.

3.4 NUCLEAR STEAM SUPPLY SYSTEM

The reactor vessel was designed, fabricated and tested in accordance with Reference 7 and applicable code cases. Where the code was not applicable, the design was evaluated based upon Reference 9. The piping design criteria and material selection were in conformance with the code for Pressure Piping, B31.1 of Reference 10, and Nuclear Code Cases N-1, N-7, N-9 and N-10. Fabrication and inspection of the pipe conformed to Section I of Reference 7. Piping supports were designed in accordance with Section VI, B31.1 of Reference 10. The steam drum was also designed to meet Sections I and VIII of Reference 7.

The reactor vessel supports were designed for seismic loads by Combustion Engineering, Inc. (Tennessee) (Reference 13). A static, seismic load analysis was performed. A horizontal load of 0.05g was applied at the computed center of gravity of the reactor vessel system.

In the early 1970's, modifications were made to provide redundancy to the post-accident systems (Reference 14). The 1970 seismic equipment specifications were given as a per cent of acceleration of gravity, and the instruments were qualified to vibration tests in accordance with References 15 and 16. A comparison of specifications of Reference 15 and Reference 17 was made by NUS Corporation (Reference 21) which concluded these tests met the guidelines of IEEE 344 (1971) provided the instruments were rigidly attached to floors or walls, or to rigid ($f > 33$ hz) equipment.

The Reactor Depressurization System (RDS) design was modified in 1974 at which time the RDS was redesigned in accordance with 1974 seismic requirements (References 11 and 12). Based upon further study of seismicity data and analysis, the Safe Shutdown Earthquake (SSE in 1974) peak ground acceleration level was set at 0.12g (Reference 12).

Floor response spectra were generated by the Kapur method (Reference 18), where the building model response was based on the SRSS response from Regulatory Guide 1.60 ground response spectra (Reference 19). The modal damping for the containment structure was assumed as 5% of critical rather than 4% for welded structures, in conformance with Regulatory Guide 1.61 on the basis of the steel-concrete interface. The floor response spectra were broadened ±10% near the peaks. The RDS components were analyzed by the modal response technique based upon floor response spectra for both vertical and horizontal input motion using damping values from Regulatory Guide 1.61. The two horizontal excitations were assumed to act simultaneously with the vertical component, but remain statistically independent. The square root of the sum of the squares (SRSS) technique was used to combine modes and directions of excitation. Stresses resulting from seismic excitation were combined with the stresses from other loads such that the computed stresses did not exceed the limits set by the codes and standards (Reference 7, 1974 edition).

TABLE 3-1

BIG ROCK POINT CONTAINMENT DESIGN PARAMETERS

Design Pressure, Internal	27 psig
Design Pressure, External (Coincident with dead load only)	0.5 psig*
Design Temperature Rise (Coincident with design internal pressure)	190 degrees F**
Design Maximum Temperature	235 degrees F
Wind Load	ASA Std A58.1
Without Snow Load	(Basic wind pressure= 30 psf)
With Snow Load	60 mph
Snow Load	ASA Std A58.1 (max = 40 psf at top)
Lateral Seismic Force (Coincident with dead load and snow load only)	5 per cent of gravity
Maximum Leakage Rate at 27 psig	0.5 per cent per day

*External pressure does not govern; with shell thickness designed to withstand 27 psig internal pressure, safe external pressure coincident with dead load only is 1.22 psig.

**This value assumes a rise from an initial shell temperature of 45 degrees F, and is structurally more severe than a rise from 100 to 235 degrees F, which is assumed in determining the design maximum temperature. The maximum temperature <u>rise</u> is used in determining secondary stresses due to the structural discontinuity where the vessel shell emerges from the foundation. These stresses, when combined with primary stresses, are required to be no greater than 1.5 times the allowable primary stresses.

TABLE 3-2

BIG ROCK POINT DESIGN CODE SUMMARY

System	Code
Containment Vessel	UBC, 1958 ACI 318, 1956 ASA Std A58.1 ASME B&PV Code, Sections II, VIII & IX, 1958
Containment Concrete Internal Structures	UBC, 1958 ACI 318, 1956
NSSS	UBC, 1958 ASME B&PV Code Section VIII, 1958
NSSS Piping	ASME B&PV Code Section VIII, 1958 ASA B31.1, 1955
Turbine and Other Buildings	UBC, 1958 ACI 318, 1956
Reactor Depressurization System (RDS)	ASME B&PV Code Section III, 1974
RDS Analog Instruments	ASME B&PV Code Section III, 1974 MIL-STD-167-1
Post-Incident Systems	MIL-STD-167-1 MIL-S-901C

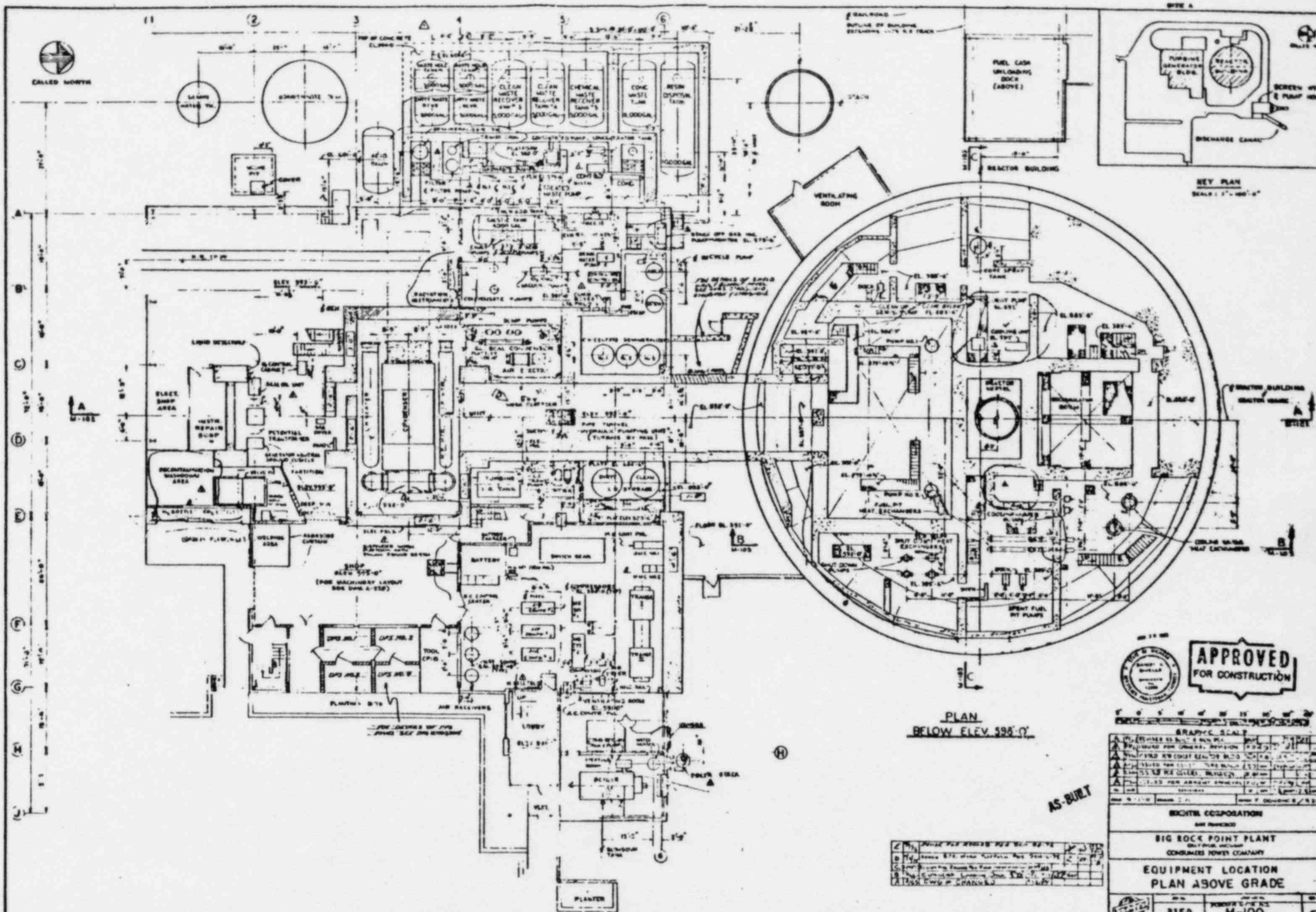

FIGURE 3-1: BIG ROCK POINT PLAN ABOVE GRADE

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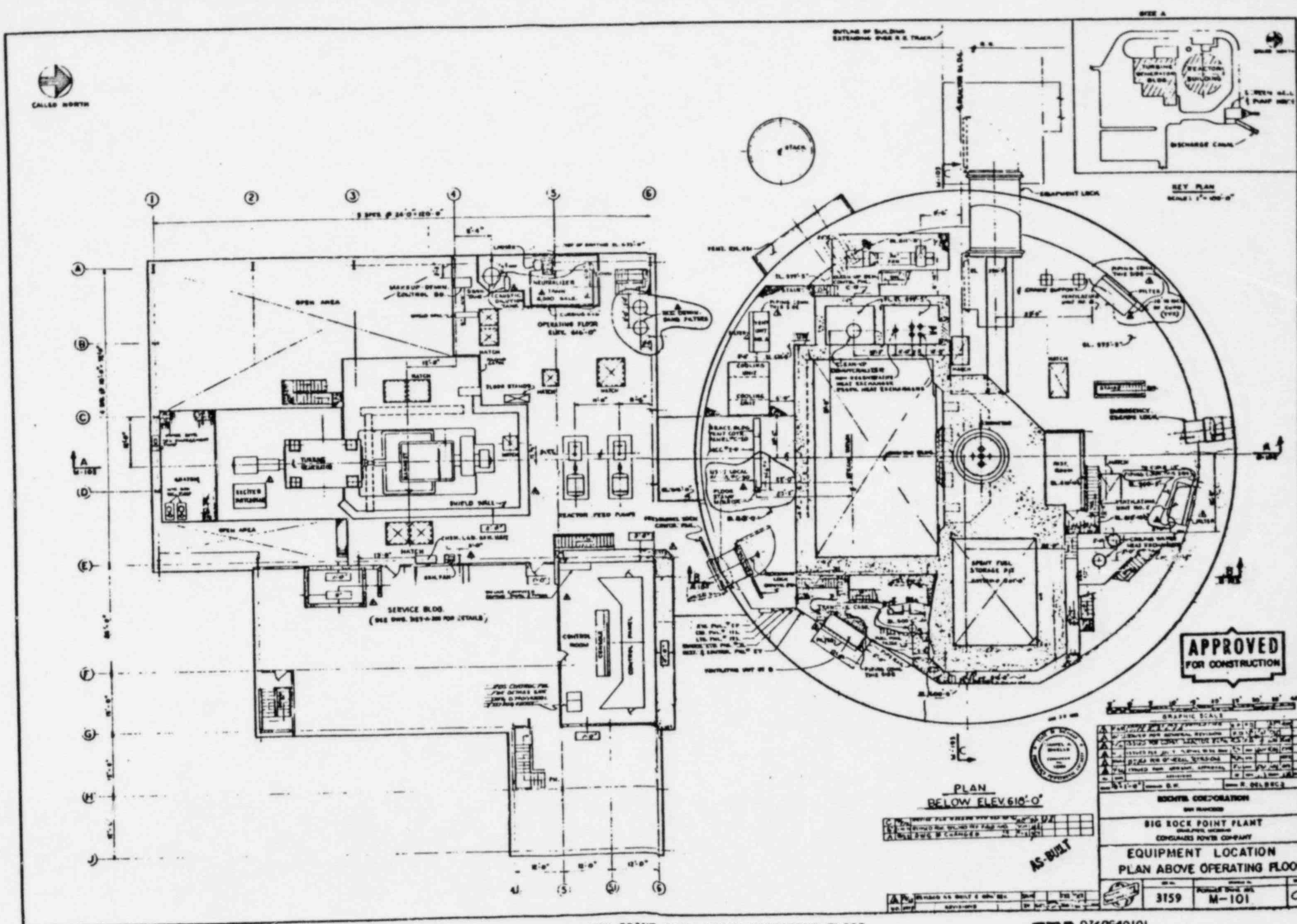

FIGURE 3-2: BIG ROCK POINT PLAN ABOVE OPERATING FLOOR

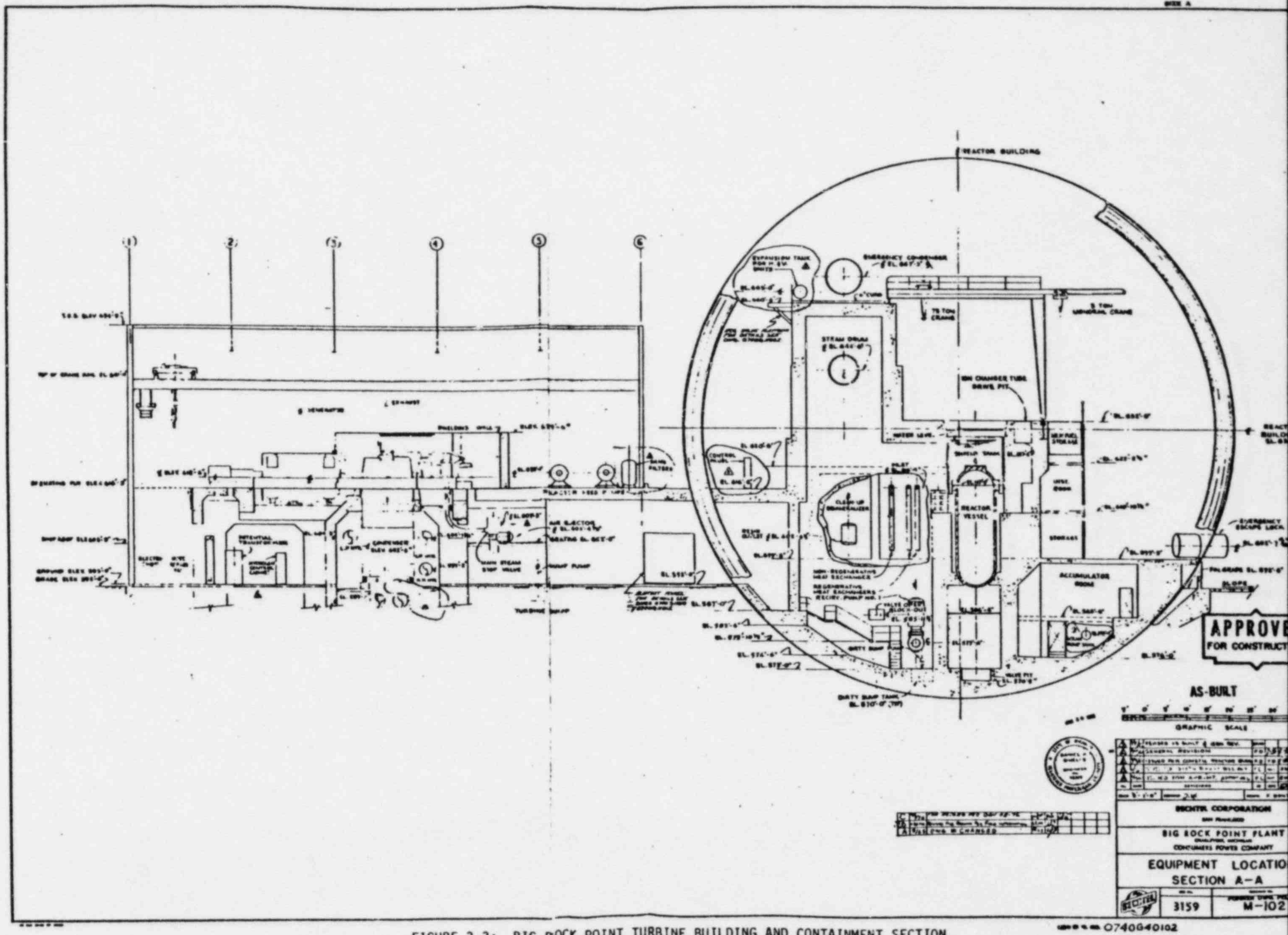

FIGURE 3-3: BIG ROCK POINT TURBINE BUILDING AND CONTAINMENT SECTION

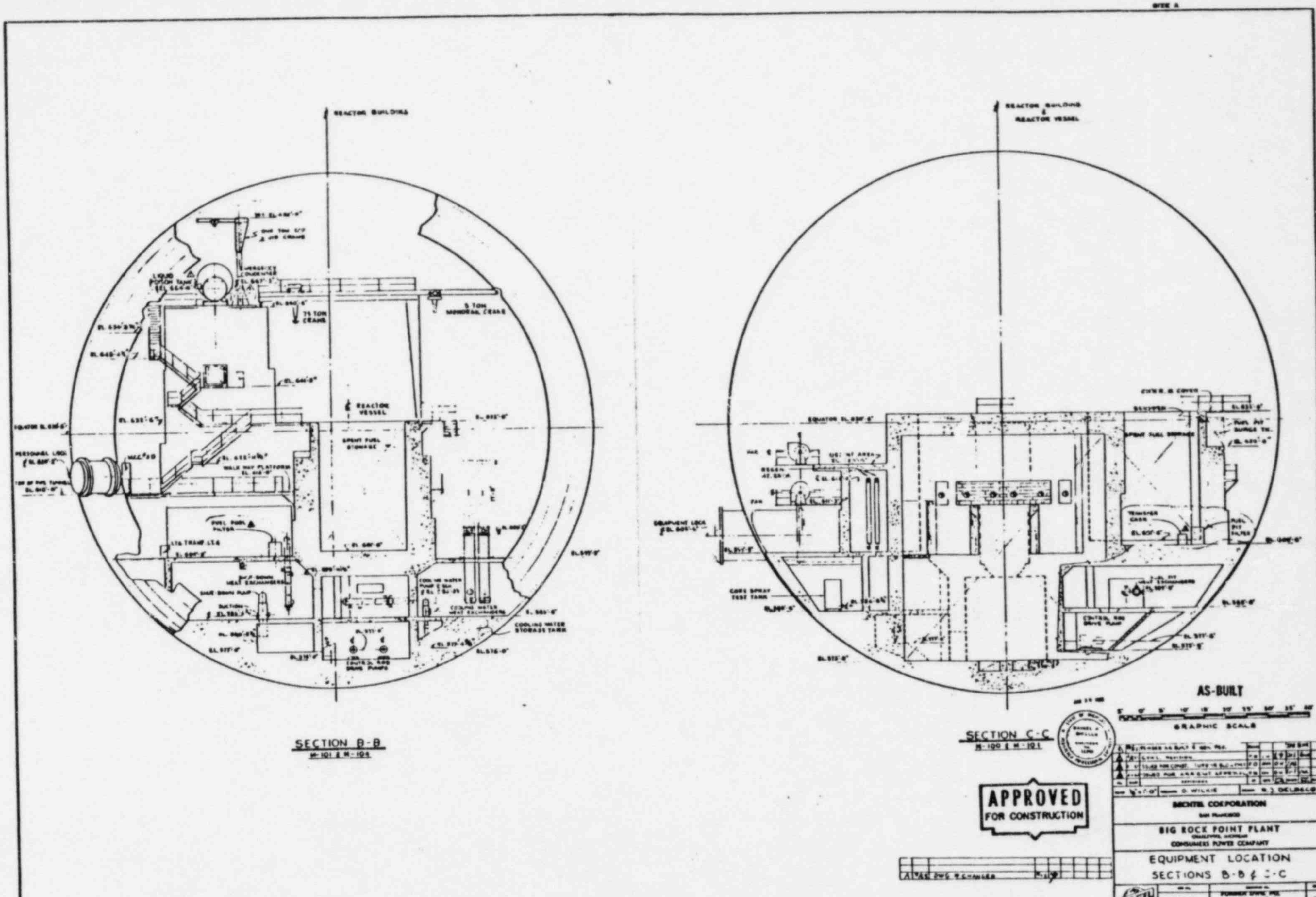

FIGURE 3-4: BIG ROCK POINT CONTAINMENT SECTIONS

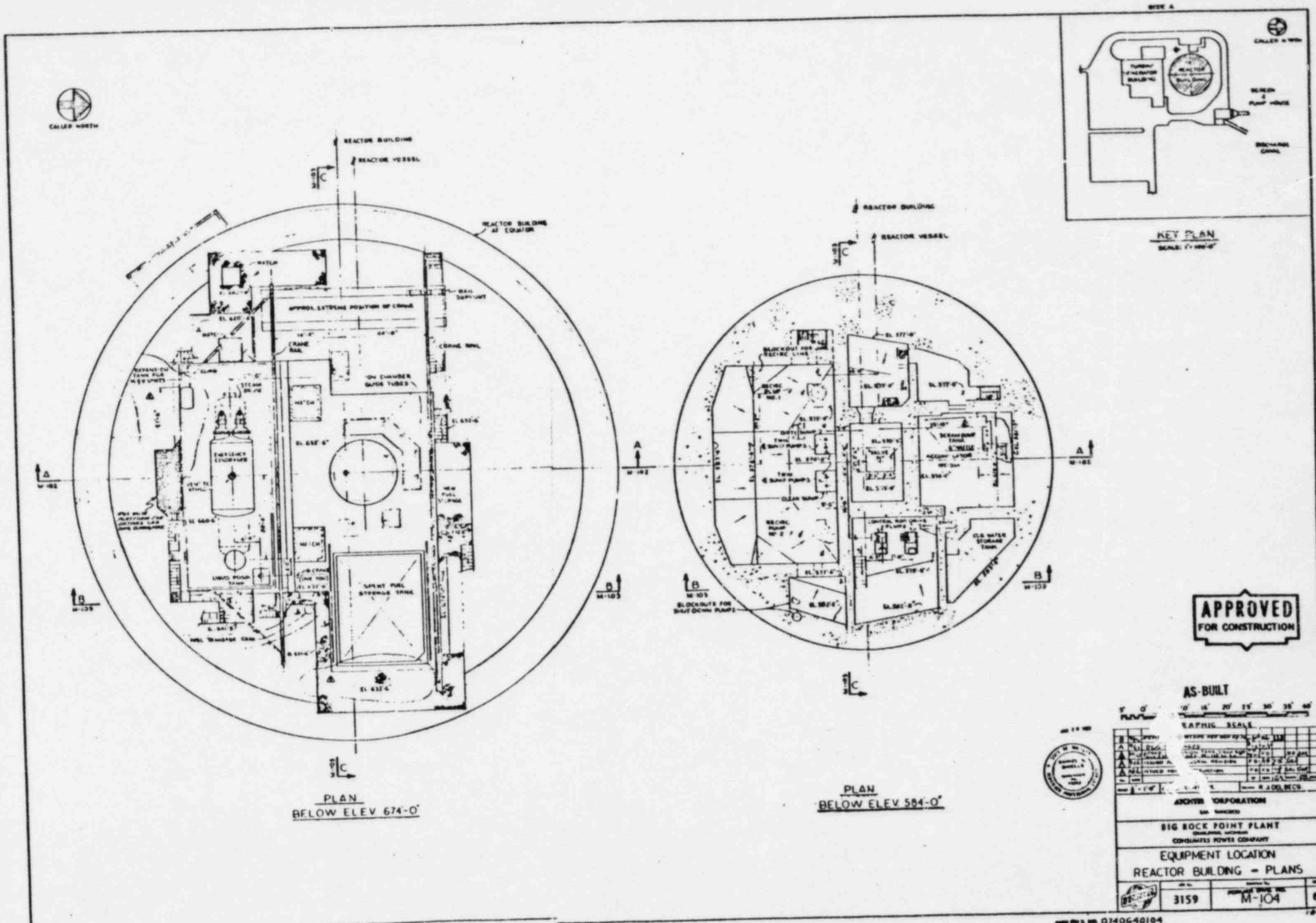

FIGURE 3-5: BIG ROCK POINT CONTAINMENT PLANS

4. SUMMARY

As a part of the Systematic Evaluation Program (SEP), a review of the seismic design bases and code requirements used in the design of the Big Rock Point (BRP) Nuclear Power Plant was performed. The principal objective was to investigate the seismic design analysis techniques and the design criteria.

Big Rock Point was designed and constructed prior to the advent of seismic design methodology in use today. Therefore, the plant was designed and constructed in accordance with the requirements of the Uniform Building Code of 1958. A simplified, static analysis was used in the design to withstand a horizontal force of 0.025g. The containment was designed to a higher static load of 0.05g.

The BRP plant has been modified and several systems upgraded to higher seismic standards. Among these were the Reactor Depressurization System which was analyzed by means of floor response spectra and stresses obtained in conformance with current criteria. Table 4-1 summarizes the design procedure and requirements used in seismic analysis of BRP structures and equipment.

TABLE 4-1

BIG ROCK POINT SEISMIC DESIGN INFORMATION

ITEM	BIG ROCK POINT	CURRENT LICENSING CRITERIA
1. Type of Plant	BWR	---
2. Plant Capacity (MWe)	70	---
3. Architect/Engineer	Bechtel	---
4. Foundation	Soil	---
5. Systems Important for Plant Safety (Equiv. Seismic Category I)	Reactor Depressurization System (RDS)	Systems necessary to: 1) Maintain Coolant System Pressure Boundary 2) Shutdown Reactor & Maintain Safe Condition 3) Prevent or Mitigate Offsite Exposure. Ref. USNRC Reg. Guide 1.29 and S.R.P. 3.2.1
6. OBE (or Design E)	Not Used	Ref. 10 CFR 100, Appendix A
7. SSE (or Max. E)	0.12g (RDS)a*	Ref. 10 CFR 100, Appendix A, S.R.P. 3.7.1
8. Response Spectra	Not Used	USNRC Reg. Guide 1.60 or Site Dependent Spectra, S.R.P. 3.7.1
9. Type of Analysis	UBC (0.05g Containment 0.025g Other Structures) Response Spectrum (RDS)	Finite Element or Lumped Mass
10. Predominant Frequencies	Not Available	---
11. Material Damping	Containment 4% (SSE)(a) RDS - Reg. Guide 1.61	Ref. USNRC Reg. Guide 1.61 S.R.P. 3.7.1

* See Notes

TABLE 4-1 (continued)

ITEM	BIG ROCK POINT	CURRENT LICENSING CRITERIA
12. Modal Combinations	SRSS (a)	SRSS or Modification, USNRC Reg. Guide 1.92, S.R.P. 3.7.2
13. Directional Combinations	3-Direct. Concurrently (SRSS) (a)	3-Direct. Concurrently (SRSS) Ref. USNRC Reg. Guide 1.92, S.R.P. 3.7.2
14. Time History Analysis	Synthetic Earthquake (a)	S.R.P. 3.7.1
15. Floor Response Spectra	Kapur (a)	Ref. USNRC Reg. Guide 1.122, S.R.P. 3.7.2
16. Testing of Equipment	MIL-STD-167-1 (b)	Ref. IEEE 344
17. Design Load Combinations	Containment: Seismic (0.05g) + dead load + snow (c)	ASME B&PV Code Section III, Division 2
	Internal Concrete Structures: Seismic (0.05g) + dead load + equipment NSSS: Seismic (0.05g) + dead load + pressure NSSS Piping: Seismic (0.025g) + pressure + equipment Turbine Building: Seismic (0.025g) + dead load + equipment	USNRC Reg. Guides 1.10, 1.15, 1.18, 1.19, 1.48, 1.55, S.R.P. 3.8.1, 3.8.3, 3.8.4, 3.8.5
18. Simplified Design Methods	Not Used	Floor Spectra Required S.R.P. 3.7.2 Peak of Floor Spectrum S.R.P. 3.7.2, 3.7.3

TABLE 4-1 (continued)

NOTES

(a) The only dynamic analysis was for the RDS for a 0.12g SSE. A dynamic analysis of the structure was done in order to develop floor response spectra. Presumably an artificial earthquake, whose response spectra enveloped those of Reg. Guide 1.60, was used, but no details of the structure model or time history are available in the docket other than 4% of critical damping was used for the building.

(b) The analog instruments required for the RDS were tested in accordance with MIL-STD-167-1 (SHIPS), "Mechanical Vibrations of Shipboard Equipment," May 1, 1974. A subsequent evaluation by NUS Corporation (Reference 21) concluded that these tests met the seismic requirements of the BRP RDS in accordance with IEEE-344 (1971) guidelines provided the instruments were rigidly attached to floors or walls or to rigid ($f > 33$ hz) equipment.

(c) Non-seismic loads also included internal pressure + wind + snow + dead weight. However, internal pressure was not combined with seismic. Wind load was the controlling factor.

REFERENCES

1. Final Hazards Summary Report of the Big Rock Point Nuclear Generating Station, Docket 50155-2.

2. "Earthquake History of the United States," 1956, U. S. Department of Commerce Publication 41-1.

3. "Earthquake History of the United States," Coast and Geodetic Survey Publication 609.

4. Uniform Building Code, 1958 edition.

5. 10 Code of Federal Regulations, Part 100, Design Basis for Nuclear Generating Stations.

6. American Nuclear Standards, ANS.

7. ASME Boiler and Pressure Vessel Code, 1958.

8. American Concrete Institute, ACI 318-1956.

9. "Tentative Structural Design Basis for Reactor Pressure Vessels and Directly Associated Components," April 1, 1958, Navy Bureau of Ships Publication.

10. ASA, 1955 edition, latest revision.

11. Docket 50155-317, October 3, 1974, Request for Additional Information About RDS Design.

12. Docket 50155-344, December 17, 1974, Reply for the Above Request.

13. Docket 50155-50, Amendment 10, Addenda to FHSR, May 1, 1962.

14. Docket 50155-86, Special Report No. 21: Investigation of and Correlation of Deficiencies Associated with Equipment Required to Operate During a Postulated LOCA," May 2, 1975.

15. MIL-STD-167, Mechanical Vibrations of Shipboard Equipment; and MIL-STD-901C, Shock Tests; Requirements for.

16. SUNTAC Nuclear Corporation Report, "Seismic Qualification of RDS for BRP Plant," Part of Amendment 8, Docket 50155-50, or NUS Report No. 1354.

17. IEEE-STD-344, "Recommended Procedures for Seismic Qualification Class I Electric Equipment for Nuclear Generating Stations," 1971.

18. Kapur, K. and L. C. Shao, "Generation of Floor Response Spectra for Equipment Design," Proceedings of ASCE Specialty Conference on Structural Design of Nuclear Plant Facilities, December, 1973.

19. U.S. AEC Regulatory Guide 1.60, "Design Response Spectra for Nuclear Power Plants," 1973.

20. U.S. AEC Regulatory Guide 1.61, "Damping Values for Seismic Design of Nuclear Power Plants," 1973.

21. NUS Corporation Technical Report No. 1354, "Seismic Qualification of Reactor Depressurization System Analog Instruments for the Big Rock Point Plant," February 28, 1975.