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NUCLEAR REGULATORY COMMISSION ISSUANCES

January 1985

U.S. NUCLEAR REGULATORY COMMISSION

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NUCLEAR REGULATORY COMMISSION ISSUANCES

January 1985

This report includes the issuances received during the specified period from the Commission (CLI), the Atomic Safety and Licensing Appeal Boards (ALAB), the Atomic Safety and Licensing Boards (LBP), the Administrative Law Judge (ALJ), the Directors' Decisions (DD), and the Denials of Petitions for Rulemaking (DPRM).

The summaries and headnotes preceding the opinions reported herein are not to be deemed a part of those opinions or to have any independent legal significance.

U.S. NUCLEAR REGULATORY COMMISSION

Prepared by the Division of Technical Information and Document Control,
Office of Administration, U.S. Nuclear Regulatory Commission, Washington, D.C. 20555
(301/492-8925)

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Alan S. Rosenthal, Chairman
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Gary J. Edles
Dr. Reginald L. Gotchy
Howard A. Wilber

APPEAL BOARDS

Cite as 21 NRC 1 (1985) **ALAB-795**

UNITED STATES OF AMERICA
NUCLEAR REGULATORY COMMISSION

ATOMIC SAFETY AND LICENSING APPEAL BOARD

Administrative Judges:

Thomas S. Moore, Chairman
Dr. W. Reed Johnson
Dr. Reginald L. Gotchy

In the Matter of **Docket No. 50-155-OLA**
(Spent Fuel Pool Modification)

CONSUMERS POWER COMPANY
(Big Rock Point Plant) **January 9, 1985**

Finding no errors that require corrective action, the Appeal Board affirms on *sua sponte* review a series of Licensing Board decisions that ultimately authorized a license amendment permitting the expansion of the Big Rock Point Nuclear Power Plant spent fuel pool.

APPEAL BOARD: *SUA SPONTE* REVIEW

An appeal board's affirmance on *sua sponte* review of a licensing board's decision does not signify approval of everything said and done by the board below. Thus, an appeal board will not give *stare decisis* effect to licensing board conclusions on legal issues not brought to it by way of an appeal. *Duke Power Co.* (Cherokee Nuclear Station, Units 1, 2, and 3), ALAB-482, 7 NRC 979, 981 n.4 (1978). Such an affirmance only connotes agreement with the ultimate resolution of those issues crucial to the result reached. *See Portland General Electric Co.* (Trojan Nuclear Plant), ALAB-181, 7 AEC 207, 208 n.4 (1974).

MEMORANDUM AND ORDER

We have before us for our customary *sua sponte* review a series of seven "initial" decisions, supplemental initial decisions and addendum to initial decisions, issued over a two-year span by the Licensing Board in this spent fuel pool amendment proceeding.[1] We deferred our review of all decisions until after the Licensing Board issued the last one.[2] That decision was issued on September 25, 1984 and authorized a license amendment permitting the expansion of the Big Rock Point Nuclear Power Plant spent fuel pool.[3] No appeals have been filed from six of the Licensing Board's decisions and the appeal of joint intervenors, Christa-Maria, Mills, and Bier, from a seventh decision apparently was withdrawn.[4] In any event, that appeal was not perfected.

We have reviewed each of the Licensing Board's decisions on our own initiative and find no errors that demand corrective action. Accordingly, the Licensing Board's decisions are affirmed. We emphasize, however, that our affirmance on *sua sponte* review does not signify approval of everything said and done by a board below. For this reason, "we do not give *stare decisis* effect to licensing board conclusions on legal issues not brought to us by way of an appeal."[5] Indeed, our affirmance only connotes agreement with the ultimate resolution of those issues crucial to the result reached. In this particular instance, no inference should be drawn that we agree with the reasoning by which the Licensing Board admitted contentions to this proceeding or justified the

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[1] *See* LBP-82-60, 16 NRC 540 (1982); LBP-82-77, 16 NRC 1096 (1982); LBP-82-78, 16 NRC 1107 (1982); LBP-83-44, 18 NRC 201 (1983); LBP-83-44A, 18 NRC 211 (1983); LBP-84-32, 20 NRC 601 (1984); LBP-84-38, 20 NRC 1019 (1984).

An additional "initial" decision was previously before us on directed certification. LBP-82-97, 16 NRC 1439 (1982), *rev'd and remanded*, ALAB-725, 17 NRC 562 (1983).

[2] *See* Order of August 31, 1982 (unpublished); Order of October 4, 1982 (unpublished). Because our October 4, 1982 Order was issued after the Board already had handed down its third "initial" decision, we cautioned that "[i]n the future, the Licensing Board should, if possible, confine its issuances to a minimum number of partial initial decisions." Order at 2. Apparently, the Board overlooked our admonition.

[3] LBP-84-38, *supra*.

[4] *See* Letter of October 2, 1984, from Christa-Maria to all parties. *See also* Order of October 24, 1984 (unpublished).

[5] *Duke Power Co.* (Cherokee Nuclear Station, Units 1, 2, and 3), ALAB-482, 7 NRC 979, 981 n.4 (1978).

result reached.[6] Nor do we necessarily agree with the Board's discussion of matters which do not have a direct bearing on the outcome.[7]

The Licensing Board's decisions are *affirmed*.

It is so ORDERED.

FOR THE APPEAL BOARD

C. Jean Shoemaker
Secretary to the
Appeal Board

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[6] For example, the Licensing Board permitted the litigation of a number of issues pertaining to the Big Rock Point emergency plan. Putting to one side the procedural machinations surrounding the admission of these issues (*see* LBP-82-32, 15 NRC 874 (1982); LBP-80-4, 11 NRC 117 (1980)), it is difficult to see how the expansion of a fuel pool could ever properly implicate the facility emergency plan. Any additional spent fuel placed in the expanded pool would make an entirely negligible contribution to the plant's radioisotopic inventory and to its potential for radiological r[illegible]ses.

[7] *See Portland General Electric Co.* (Trojan Nuclear Plant), ALAB-[illegible], 7 AEC 207, 208 n.4 (1974).

UNITED STATES OF AMERICA
NUCLEAR REGULATORY COMMISSION

ATOMIC SAFETY AND LICENSING APPEAL BOARD

Administrative Judges:

Thomas S. Moore, Chairman
Gary J. Edles
Dr. Reginald L. Gotchy

In the Matter of **Docket No. 50-344-OLA**

PORTLAND GENERAL ELECTRIC COMPANY, *et al.*
(Trojan Nuclear Plant) **January 10, 1985**

The Appeal Board in this operating license amendment proceeding declines to undertake *sua sponte* review of a Licensing Board's decision that was based on the proposed findings of fact and conclusions of law stipulated by the parties and adopted by the Licensing Board.

MEMORANDUM AND ORDER

On November 28, 1984, the Licensing Board in this spent fuel pool amendment proceeding issued an initial decision permitting Amendment No. 88 to License No. NPF-1 for the Trojan Nuclear Plant to remain in full force and effect without modification. The license amendment had previously been issued by the Director of the Office of Nuclear Reactor Regulation pursuant to 10 C.F.R. 50.92. That provision allows the issuance of an amendment without a prior hearing when the Director finds that the amendment involves no significant hazard to the public health and safety. *See also* 42 U.S.C. 2239(a); 10 C.F.R. 50.91. No appeals from the initial decision were filed.

In the absence of an appeal, our customary practice is to review *sua sponte* the authorization of licensing action. *See, e.g., Consumers Power Co.* (Big Rock Point Plant), ALAB-795, 21 NRC 1 (1985). In this instance, however, we eschew that practice. After a brief hearing on the admitted contentions, the applicant filed proposed findings of fact and conclusions of law that the intervenor (the State of Oregon) and the NRC staff then adopted. At that point there was, in effect, a stipulated resolution or a settlement of the contested issues and thus no need for the Board below to do anything more than dismiss the proceeding.[1] In an amendment proceeding where the Board has raised no significant safety or environmental issues on its own motion, as in an operating license proceeding, the only issues to be decided by a licensing board are those contested by the parties. *See* 10 C.F.R. 2.760a. Once those issues are no longer in dispute, whether before or after the hearing, the proceeding should be dismissed. *See* 10 C.F.R. 2.761. Because we do not review proceedings that are dismissed when the parties settle the issues, we shall not conduct a *sua sponte* appellate review here.

It is so ORDERED.

FOR THE APPEAL BOARD

C. Jean Shoemaker
Secretary to the
Appeal Board

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[1] In fact, all the Licensing Board did was adopt the agreed upon findings of the parties.

UNITED STATES OF AMERICA
NUCLEAR REGULATORY COMMISSION

ATOMIC SAFETY AND LICENSING APPEAL BOARD

Administrative Judges:

Christine N. Kohl, Chairman
Dr. W. Reed Johnson
Howard A. Wilber

In the Matter of **Docket No. 50-382-OL**

LOUISIANA POWER & LIGHT COMPANY
(Waterford Steam Electric Station, Unit 3) **January 17, 1985**

The Appeal Board grants a motion by the NRC staff for clarification and/or reconsideration of an earlier Appeal Board decision, ALAB-792, 20 NRC 1585 (1984), that held that the Board has jurisdiction to rule on intervenors' motion to reopen the record in this operating license proceeding.

APPEAL BOARD: JURISDICTION

When an appeal board has finally determined some issues in a proceeding and others are still pending before it, the board has jurisdiction over new matters raised by a party if there is a "reasonable nexus" or "a rational and direct link" between the new issues and those pending. A total identity or commonality of issues is not required. *See, e.g., Virginia Electric and Power Co.* (North Anna Nuclear Power Station, Units 1 and 2), ALAB-551, 9 NRC 704, 707 (1979); *Florida Power and Light Co.* (St. Lucie Nuclear Power Plant, Unit No. 2), ALAB-579, 11 NRC 223, 226 (1980).

APPEAL BOARD: JURISDICTION

A party cannot properly import wholly unrelated, discrete issues into a closed proceeding by combining them, in a single motion to reopen, with another issue that is related to a matter pending before an appeal board. In such a case the appeal board could sever the unrelated material from the matter over which it had retained jurisdiction.

ADJUDICATORY BOARDS: JURISDICTION

Jurisdictional disputes in NRC proceedings do not have Constitutional dimensions.

ADJUDICATORY BOARDS: JURISDICTION

In determining jurisdictional disputes in NRC proceedings, an adjudicatory board may take into account practical considerations, like efficiency in the disposition of the matter at hand and fairness to the parties. *See Philadelphia Electric Co.* (Limerick Generating Station, Units 1 and 2), ALAB-726, 17 NRC 755 (1983).

APPEARANCES

Sherwin E. Turk for the Nuclear Regulatory Commission staff.

Bruce W. Churchill, Washington, D.C., for applicant Louisiana Power & Light Company.

MEMORANDUM AND ORDER

The NRC staff has moved for clarification and/or reconsideration of ALAB-792, 20 NRC 1585 (1984). In that memorandum decision, we determined that we have jurisdiction to rule on Joint Intervenors' November 8, 1984, motion to reopen the record in this operating license proceeding. We concluded that there is a reasonable nexus between that motion and another motion to reopen concerning the adequacy of the concrete basemat on which the Waterford facility rests, filed earlier by Joint Intervenors and still pending before us. The staff essentially agrees with our analysis but asks that we clarify that our jurisdiction extends to only that *part* of the November 8 motion that specifically relates to matters raised by the basemat motion.[1] We grant the staff's motion and clarify our decision as explained below.[2]

Joint Intervenors' November 8 motion seeks to raise three new contentions that allege (1) a breakdown in applicant's construction quality assurance program, (2) a lack of integrity and competence on the part of applicant's management, and (3) a lack of confidence in the NRC staff's inspection and investigation efforts at the Waterford facility. The contentions contain numerous, more specific subissues as well. As we stated in ALAB-792, "[a]lthough [this] motion is substantially broader, there is a clear overlap insofar as Joint Intervenors allege [in their earlier motion to reopen] quality assurance deficiencies in connection with the construction of the basemat." 20 NRC at 1589. Acknowledging that it would require "a careful examination," the staff would have us parse through the motion and excise from our consideration any allegations not specifically related to the concrete basemat. NRC Staff's Motion (Dec. 24, 1984) at 7.

The cases on which we relied for guidance in ALAB-792 refer to a "*reasonable* nexus" and "a rational and direct *link*" — not a total identity or commonality of issues. *See, e.g., Virginia Electric and Power Co.* (North Anna Nuclear Power Station, Units 1 and 2), ALAB-551, 9 NRC 704, 707 (1979) (emphasis added); *Florida Power and Light Co.* (St. Lucie Nuclear Power Plant, Unit No. 2), ALAB-579, 11 NRC 223, 226 (1980) (emphasis added). That is not to say that a party could properly import wholly unrelated, discrete issues into a closed proceeding by combining them, in a single motion to reopen, with another issue that is related to a matter already pending before us. In such a case, we could and would sever the unrelated material from the matter over which we have retained jurisdiction. But contrary to the staff's assertion, the particular issues raised by Joint Intervenors' November 8 motion are not so easily separated. That is, whether many specific matters raised in that motion have a reasonable nexus to the basemat motion will not be ap-

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[1] Applicant agrees with the staff and us that we would have jurisdiction if there is a reasonable nexus between the two motions. Applicant argues, however, that there is no such link between any of the matters raised in the motions here. Joint Intervenors did not file a reply to the staff's motion.

[2] We deny the staff's curious request, in note 3 of its motion, for a stay of ALAB-792. We fail to understand exactly what the staff wants us to stay and why. ALAB-792 "ordered" nothing. It simply expressed the view, in advance of our merits ruling on the motion, that we have jurisdiction over the November 1984 motion and intend to entertain it. Both the staff and applicant have already addressed the entire motion to reopen, on its merits and at considerable length. Further, we have not yet ordered the "litigation" of any matters raised by the motion to reopen, and, indeed, it remains to be seen whether any such litigation will be ordered. Thus, we do not understand the staff's assertion that, without a stay now, irreparable injury may result from the "litigation" of matters unrelated to the basemat.

parent, in our view, until those matters have been considered on the merits.

For example, management integrity — as discussed in Joint Intervenors' motion — cannot be given reasonable or fai. *consideration* by reference to only one part of the plant (the basemat) and in isolation from the arguments raised concerning other aspects of plant management. Similarly, inquiry into quality assurance in one area (e.g., basemat inspector certification) *may* necessarily spill over into other areas of quality assurance performance. Perhaps after our merits review of Joint Intervenors' motion is completed, the various issues raised by both motions will appear more distinct and severable. We may then decide to terminate our consideration of matters genuinely unrelated to the basemat motion and possibly refer them to the Director of Nuclear Reactor Regulation for resolution.[3] For the sole present purpose of determining whether we should even entertain the motion, however, we cannot now draw such clear distinctions.[4]

We have previously noted, albeit in a somewhat different context, that jurisdictional disputes in NRC proceedings do not have Constitutional dimensions. It is therefore proper to take into account practical considerations, like efficiency in the disposition of the matter at hand and fairness to the parties. *See Philadelphia Electric Co.* (Limerick Generating Station, Units 1 and 2), ALAB-726, 17 NRC 755 (1983). With that in mind and subject to the reservation noted above, we again conclude that we have jurisdiction to consider the entirety of Joint Intervenors' November 8 motion to reopen.[5]

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[3] In this connection, we stress that the comments made here concerning jurisdiction are *not* to be construed as reflecting any judgment whatsoever on the merits of Joint Intervenors' motion.

[4] Apparently, the staff cannot either. Other than listing some examples of general matters it considers unrelated to the basemat, the staff has not gone through Joint Intervenors' 62-page motion and identified the *specific* pages and arguments that are assertedly beyond our jurisdiction. *See* NRC Staff's Motion at 4.

[5] The staff suggests that the Commission itself may have jurisdiction to consider the matters raised in Joint Intervenors' motion to reopen that are not related to the basemat, and that, pursuant to a "remand order," the Commission could then direct us to consider such matters anyway. *Id.* at 9 n.7. We previously considered that possibility and concluded that, if this is so, the Commission has already delegated us that authority in the Rules of Practice. *See* 10 C.F.R. § 2.785(b)(1) ("Appeal Board will also exercise the authority and perform the functions which would otherwise have been exercised and performed by the Commission under . . . [10 C.F.R. §] 2.730" (disposing of motions)). Under that view, there is additional cause for us to consider the entirety of Joint Intervenors' motion.

The staff's motion for clarification and/or reconsideration of ALAB-792 is *granted*, and ALAB-792 is *clarified* in accordance with the discussion above.

It is so ORDERED.

FOR THE APPEAL BOARD

C. Jean Shoemaker
Secretary to the
Appeal Board

Atomic Safety and Licensing Boards Issuances

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LICENSING BOARDS

UNITED STATES OF AMERICA
NUCLEAR REGULATORY COMMISSION

ATOMIC SAFETY AND LICENSING BOARD

Before Administrative Judges:

John H Frye, III, Chairman
Dr. James H. Carpenter
Dr. Peter A. Morris

In the Matter of	**Docket No. 40-2061-ML** **(ASLBP No. 83-495-01-ML)**
KERR-McGEE CHEMICAL CORPORATION **(West Chicago Rare Earths Facility)**	**January 9, 1985**

Licensing Board rules that, in permitting document inspection after having screened its files to remove privileged documents, Applicant waived its right to subsequently assert attorney-client or work product privileges. Licensing Board also rules that only parties must respond to requests for documents and that State agencies which are not parties to a proceeding need not so respond. However, such State agencies may be subject to subpoenas seeking documents.

RULES OF PRACTICE: DISCOVERY (PRIVILEGED MATTER)

In determining whether an inadvertent disclosure of a privileged document operates to waive the privilege, Licensing Board considers the precautions taken to prevent disclosure, the effectiveness of those precautions, whether the documents were produced under the compulsion of a rigorous schedule, and the promptness of the disclosing party's objection on discovering the disclosure.

RULES OF PRACTICE: DISCOVERY

Under 10 C.F.R. § 2.741, only parties must respond to document requests.

RULES OF PRACTICE: DISCOVERY

Subpoenas may be issued to State agencies which are not parties to a proceeding in order to obtain documents.

MEMORANDUM AND ORDER
(Ruling on Discovery Disputes)

Discovery disputes currently exist between Kerr-McGee and the People of the State of Illinois.[1] These disputes concern document requests filed by each party on the other and the schedule for further discovery. Briefly, we are now asked to decide whether Kerr-McGee has waived its claim of privilege as to ninety-two documents which counsel for the People has inspected and wishes copies, whether counsel for the People must produce relevant documents for inspection by Kerr-McGee's counsel from any State agency possessing them or whether counsel's search for such documents may be limited to counsel's client agencies, and whether further discovery in this proceeding should be staged pending our ruling on the above two matters and the pending motions for reconsideration of our Memorandum and Order ruling on contentions (LBP-84-42, 20 NRC 1296 (1984)).

WAIVER OF PRIVILEGE

In our Prehearing Conference Order of February 24, 1984 (unpublished), we established a schedule for discovery. This schedule was extended twice at the request of the parties.[2] On August 3, 1984, Kerr-

[1] The People of the State of Illinois and the Illinois Department of Nuclear Safety are intervening parties in this proceeding. They are collectively referred to as "the People."

[2] Motions for extensions of time were granted on April 3 and July 6, 1984. No party objected to these requests.

McGee requested an extension of time to September 15 for responses to requests for admissions and documents, interrogatories, and for objections to the same. No party objected and this request was granted on August 6.

Pursuant to this schedule and to agreements between counsel for Kerr-McGee and the People, counsel for the People inspected some one million pages of documents and marked approximately 30,000 pages for copying at Kerr-McGee's headquarters in Oklahoma City on September 18 through 21. At a subsequent inspection held on October 9 and 10 in West Chicago, counsel for the People reviewed about half the number of documents produced in Oklahoma City and marked about 4000 for copying. At both inspections, documents which Kerr-McGee deemed privileged had been removed and replaced with an indicator card.

Counsel for the People expected that documents which she had marked would be copied and forwarded to her. The first indication that counsel for Kerr-McGee did not intend to follow this course was communicated to her on Friday, September 21, the last day of her inspection of the Oklahoma City documents. We quote from the affidavit of Peter J. Nickles, counsel for Kerr-McGee, which accompanied Kerr-McGee's November 30 motion:

> On September 21, 1984, I met with Ms. Anne Rapkin from the Office of the Attorney General of the State of Illinois (the "State") in the offices of Kerr-McGee in Oklahoma City. Ms. Rapkin was present in the Kerr-McGee offices in order to inspect Kerr-McGee's files in connection with discovery in the above-captioned matter. I explained that Kerr-McGee had assembled all of its files relating to West Chicago for inspection by the State and had undertaken efforts to remove privileged documents from the files. Because the files were voluminous and the time available for review was short, I was not confident that all privileged materials had been removed. I therefore informed Ms. Rapkin that copies of the documents marked by the State for production would be forwarded to Covington & Burling's offices in Washington for further examination to identify privileged documents that should not be produced. Covington & Burling would then forward the copies that were determined not to be privileged to the State. Ms. Rapkin expressed no disagreement with this procedure.

Although she did not respond to Mr. Nickles' statement at the time, on Monday, September 24, Ms. Rapkin wrote Mr. Nickles stating in part:

> Before Jim and I came to Oklahoma, Mead [Mead Hedglon, in-house attorney for Kerr-McGee] reviewed the documents to be produced and withdrew a number of them on grounds of privilege. Last Friday you informed me that before the company xeroxes and mails us those documents we marked for copying, you personally will re-review them to determine whether any are privileged. It is the People's position

that whether or not any of the documents you produced might have been privileged, any privilege was waived when you produced them last week. Therefore I expect a xerox of each and every document which Jim and I marked for copying, together with any notes which may be affixed there.

The documents marked by Ms. Rapkin in Oklahoma City and in West Chicago were copied and forwarded to Mr. Nickles' office. In a November 9 letter to Ms. Rapkin, Mr. Nickles stated in part:

> As you know, while you were in Oklahoma City and in West Chicago you were given unrestricted access to every file in any way related to the West Chicago matter. This included memoranda which reflected the development of Kerr-McGee's approach to the matter from the beginning right up to the time that you were making your inspection. Many of the documents put forth proposals or set out tentative conclusions that were never adopted or perhaps even given serious consideration by Kerr-McGee. Moreover, many of the documents discuss sensitive matters and some may contain information that may be deemed to be proprietary or to reflect trade secrets.
>
> . . . [W]e believe that our internal consideration of policies and procedures is entitled to confidential treatment. We have therefore prepared the enclosed Protective Order which will afford the documents confidential treatment without delaying the proceeding. If you will sign and return the Order to us, we will then forward the nonprivileged documents that you have identified for copying.

Thus, on further examination, counsel raised not only claims of privilege, but claims of confidentiality as well. The protective order enclosed by Mr. Nickles would have accorded confidential treatment to all the documents in question and prevented their use or disclosure, absent Kerr-McGee's consent, other than for purposes of this proceeding. On November 15, Ms. Rapkin wrote Mr. Nickles rejecting the latter's protective order but offering to consider a protective order for specified documents. Ms. Rapkin noted her expectation of receiving the documents by November 23. When the documents were not furnished on that date, Ms. Rapkin moved to compel production on November 26 asserting that Kerr-McGee had waived any privilege and on November 30 Kerr-McGee moved for an order that its claims of privilege were preserved. Additionally, Kerr-McGee sought an order implementing its proposed protective order or a protective order limited to specifically identified documents. If the latter order were to be adopted, Kerr-McGee sought an additional 30 days to identify documents which contain trade secrets or other proprietary or confidential information to be protected.[3] Thus two issues had crystallized at that point:

> First, had Kerr-McGee waived its claim of privilege with respect to the documents inspected by the People; and
>
> Second, was Kerr-McGee entitled to a protective order with respect to trade secrets or other proprietary or confidential information contained in those documents.[4]

On December 10, 11, and 17, respectively, Kerr-McGee, Staff,[5] and the People replied to the two motions. The People resisted Kerr-McGee's requests for relief and argued that any privileges pertaining to the inspected documents had been waived. Similarly, Kerr-McGee resisted the People's waiver argument.

Also on December 17, the People filed a motion for an emergency ruling on the pending discovery disputes. Reciting the fact that, pending a resolution of these motions, they have voluntarily refrained from publicizing the contents of the Kerr-McGee documents, the People alleged that their constitutional rights were infringed "so long as they are constrained from informing the public about information within their knowledge . . . ." The People supported their motion with a confidential submittal summarizing the content of some of the Kerr-McGee documents. This document recites evidence that Kerr-McGee has sought to influence public opinion and elected officials with respect to its West Chicago site.[6]

Noting that early resolution of these disputes would speed the progress of this proceeding, on December 19 we scheduled a prehearing conference for December 26. Then, on December 21, Kerr-McGee responded to the emergency motion by turning over all documents with the exception of ninety-two which it claims to be privileged under the attorney-client or work product doctrines. Kerr-McGee abandoned any claim for

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[3] During the same time that the above dispute was developing, a second dispute arose concerning the People's obligation to produce documents from various State agencies that are not parties to this proceeding. Kerr-McGee's November 30 motion also sought relief with respect to that dispute. We discuss that dispute *infra*.

[4] On November 27, the People submitted a third set of interrogatories and requests for documents to Kerr-McGee. This led Kerr-McGee to seek a stay in further discovery. This motion is also dealt with, *infra*.

[5] Staff supported Kerr-McGee's motion insofar as it seeks a response to its document production request from all State agencies; Staff took no position on the other disputes.

[6] This document was not filed with the Secretary but was served on counsel under instructions not to disclose its contents. Because we saw nothing in this document which demanded that it be withheld from the public, we indicated on December 24 that, in the absence of objection received by January 4, we would transmit a copy to the Secretary for inclusion in the Commission's public files. No objections having been received, we have taken that action.

protection of trade secret or other proprietary or confidential information. As a result of this development and with the People's agreement, on December 24 we cancelled the prehearing conference. On December 26, the People commented on this dispute, noting that they did not intend to abandon their position that any privilege claims had been waived by Kerr-McGee. On December 27, Kerr-McGee moved for permission to reply to the People's response to its motion, attaching that reply. In that reply, Kerr-McGee maintains that the People acquiesced in its two-stage review procedure and argues that the case law supports the proposition that it did not waive its claim of privilege.

In considering this issue, we assume that the disclosure of the ninety-two documents was inadvertent. Thus the issue is whether Kerr-McGee's inadvertent disclosure of these documents operated to waive its right to withhold them.

The Federal case law concerning inadvertent disclosure of privileged documents is not uniform. *Suburban Sew'n Sweep Inc. v. Swiss Bernina, Inc.*, 91 F.R.D. 254, 257 (N.D. Ill. 1981 There does not appear to be a basic rule of law concerning waiver which is consistently adhered to by a majority of the Federal courts.

We begin our consideration with the traditional view of waiver recited in the Wigmore treatise on evidence which apparently serves as the foundation for the reasoning in many of the waiver-related decisions.[7] While Wigmore's text does not directly address inadvertent waiver, an explication may be found under the section on *Indirect Disclosure by the Attorney*. 8 Wigmore, Evidence §§ 2325-2327 (McNaughton 1961). There Wigmore adopts the traditional view that even an involuntary disclosure results in a waiver of the attorney-client or work product privileges.[8]

Under the Wigmore analysis, the privilege is lost when documents are disclosed, even when that disclosure is through loss or theft from the attorney,

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[7] *United States v. Kelsey-Hayes Wheel Co.*, 15 F.R.D. 461 (E.D. Mich. 1954); *United States v. Cole*, 456 F.2d 142, 144 (8th Cir. 1972); *Duplan Corp. v. Deering Milliken, Inc.*, 397 F. Supp. 1146 (D.C.S.C. 1974); *In Re Grand Jury Investigation of Ocean Transportation*, 604 F.2d 672 (D.C. Cir. 1979); *In Re Sealed Case*, 676 F.2d 793, 807 (D.C. Cir. 1982).

[8] Because of the conclusion which we reach on this subject, we find it unnecessary to consider this issue in terms of the particular privilege which is deemed waived. Further, the section of the Wigmore treatise cited herein has been used by several courts in their analysis of waiver as it applies to both the attorney-client and work product privileges. *See In Re Subpoenas Duces Tecum Fulbright and Jaworski, Vinson and Elkins, Tesoro Petroleum Corp.*, 738 F.2d 1367 (D.C. Cir. 1984), *Permian Corp. and Occidental Petroleum Corp. v. United States*, 665 F.2d 1214, 1219 (D.C. Cir. 1981).

> on the principle (§ 2326 *infra*) that, since the law has granted secrecy so far as its own process goes, it leaves to the client and attorney to take measures of caution sufficient to prevent being overheard by third persons. The risk of insufficient precautions is upon the client. This principle applies equally to documents.

Id. at 632.

We first begin with an analysis of the cases which hold that though disclosure was inadvertent or accidental, waiver of the privilege is nonetheless the result. These cases discount the element of intent and instead apply Wigmore's strict responsibility doctrine as enunciated in the often quoted case *Underwater Storage, Inc. v. U.S. Rubber Co.*, 314 F. Supp. 546, 549 (D.D.C. 1970). There the court decided it would not look behind the objective fact that the client turned over documents to his attorney for production to inquire whether the client intended that the documents be produced. The court explained that once the document was produced it was in the public domain, that is, the existence of its contents was within the knowledge of the opposing counsel and the element of confidentiality, so crucial to the privilege, was destroyed. The court in *Underwater Storage, supra,* reasoned that when confidentiality is no longer present, the basis for the privilege has been abrogated.

In *Kelsey-Hayes, supra,* an earlier case relied upon by the court in *Underwater Storage, supra,* one of the defendant corporations permitted attorneys for the Government to review its files consistent with a discovery request by the Government. The files contained twenty-nine documents which may have been subject to work product or attorney-client privilege. The court declined to give credence to the defendant's later claim that the documents' privileged status continued once they were made available to the Government's attorneys. The *Kelsey-Hayes* Court recognized the competing interests at work in the discovery process but concluded that the disclosure by defendant's attorneys negates any argument counsel might later assert as to how or why the documents were shown to opposing counsel.

> As a result of the claimant's own acts, the context in which the rule is intended to serve, the protection of confidential communications is no longer present. Since the privilege exists in derogation of the overriding interest in full disclosure of all competent evidence, where the policy underlying the rule can no longer be served, it would amount to no more than mechanical obedience to a formula to continue to recognize it.
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> * * *
>
> Nor is this result affected by Budd's assertion that the privileged documents were inadvertently handed over to the Government's representatives; that the mass of documents in its files were so voluminous that it did not know nor did it have time to discover that privileged ones were among them. It is difficult to be persuaded that these documents were intended to remain confidential in the light of the fact

that they were indiscriminately mingled with the other routine documents of the corporation and that no special effort to preserve them in segregated files with special protections was made. One measure of their continuing confidentiality is the degree of care exhibited in their keeping, and the risk of insufficient precautions must rest with the party claiming the privilege. Wigmore, 3d Ed., Sec. 2325, p. 629.

Kelsey-Hayes, supra, 15 F.R.D. at 465. Many courts applying the Wigmore standard of strict responsibility cite the above language to support a holding that waiver was effected under circumstances where an opposing party is allowed to review documents in response to a discovery request. *Grand Jury Investigation of Ocean Transportation, supra; W.R. Grace v. Pullman Inc.*, 446 F. Supp. 771 (W.D. Okla. 1976).

Cases departing from this view generally acknowledge it, but discredit it primarily for its rigidity and the lack of consideration it accords to the intent of the disclosing party. *Mendenhall v. Barber Greene Co.*, 531 F. Supp. 951, 954 (N.D. Ill. 1982), *Weil v. Investment/Indicators Research & Management,* 647 F.2d 18, 24 (9th Cir. 1981). Other factors considered in these cases are the confidentiality of the document, whether reasonable arrangements were made to protect against disclosure (*Kelsey-Hayes, supra,* 15 F.R.D. at 464), whether disclosure was made under the compulsion of a court-ordered expedited discovery schedule, and whether a court expressly or implicitly reserved the disclosing party's right to protect privileged documents which may have slipped through its initial screening. *Transamerica Computer Co. v. IBM,* 573 F.2d 646 (9th Cir. 1978).

Where courts have not adopted Wigmore's view of the waiver standard, primary emphasis is usually placed on the disclosing party's intent. The intent is pivotal in these cases to determine if disclosure was inadvertent. Inadvertence has come to indicate that the disclosing party did not knowingly relinquish its right to make objections based on privilege because it did not intentionally divulge the information,[9] but only disclosed it through some accident or error in its own review. Courts are generally sympathetic to the arguments of inadvertence when the party can show not only that there was no intent to disclose, but a tremendous volume of material through which it had to sift, and strict time pressures in which to review the documents, including orders by the court compelling discovery under an expedited schedule. *Connecticut Mutual Life Insurance Co. v. Shields,* 18 F.R.D. 448 (S.D.N.Y. 1955); *Dunn Chemical Co. v. Sybron Corp.*, 1975-2 Trade Cas. (CCH) ¶ 60,561 at 67,463 (S.D.N.Y. 1975); *Control Data Corp. v. IBM Corp.*, 16 Fed. R. Serv. 2d 1233 (D. Minn. 1972); *Transamerica Computer, supra,* 573 F.2d

[9] *Mendenhall, supra,* 531 F. Supp. at 953 n.9.

at 653. We conclude that the rationale stated in these cases is the better view.

This view has recently been enunciated in *Magnavox Co. v. Bally Midway Manufacturing Co. and Sanders Associates Inc.*, third-party defendant, No. 83C 2357 (N.D. Ill., Nov. 5, 1984) and *Donovan v. Robbins*, Nos. 78C 4075, 82C 7951 (N.D. Ill., Nov. 14, 1983). Both cases follow the intent analysis and both held that a clearly inadvertent disclosure does not waive privilege. As noted above, we need not consider this issue because we have assumed that the waiver was inadvertent. Both cases also carefully recite and amplify the crucial factors which, if combined with inadvertent disclosure, result in preservation of the privilege. Both cases either expressly or implicitly acknowledged that not every claim of inadvertence is entitled to relief. We therefore move to the other factors which are to be weighed to determine if Kerr-McGee has waived its privileges.

The People submitted their discovery request to Kerr-McGee on July 13, 1984. At the August 22, 1984, Prehearing Conference, counsel for Kerr-McGee informed the Board that Kerr-McGee was prepared to submit its objections to discovery requests or provide the parties with the opportunity to review requested files by September 15, 1984,[10] the date which counsel had earlier requested and which had been granted on August 6. Based on this extension, Kerr-McGee's filing of December 27, 1984, seems to imply that it was in some way compelled under a Board-imposed expedited schedule. We do not agree. There have been no schedule disputes presented to us for resolution subsequent to the first prehearing conference. Discovery was proceeding on schedules agreed to by the parties, which we adopted. Counsel undeniably is aware of the right to come to the Board with any difficulties in complying with those schedules. Yet until our receipt of the motions here in question, we were not informed that problems had arisen.

We have assumed that the disclosure was inadvertent, as required by *Magnavox, supra*. However, under that holding the circumstances must be such that adequate precautions were taken initially to prevent disclosure if the privilege is to be preserved. Here an initial review was made and documents were removed from the files inspected by counsel for the People. We are not unsympathetic to the fact that more than a million pages of documents had to be compiled and reviewed by Kerr-McGee. We recognize that this meant the company and/or its law firm was faced with the need to amass substantial manpower to sift through

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[10] Prehearing Conference, August 22, 1984, Tr. 236.

the files. But, while the number of documents to be reviewed must be taken into account, we think it is also necessary to juxtapose that with the number of documents which were disclosed. Although only under the pressure of a schedule to which it had agreed, Kerr-McGee allowed ninety-two documents to slip through its review process. This is not an insignificant number and indicates that the precautions taken were inadequate, a fact recognized by Kerr-McGee's counsel when he indicated that a second review was necessary. A cursory review of the documents is not enough to prevent a waiver. The review process must accomplish its intended goal. In the cases relied on by Kerr-McGee, the review process was much more effective; despite the compulsion of schedules, only a few documents slipped through.

The courts also require prompt objection to prevent a waiver of discovery objections. Counsel for Kerr-McGee informed the People on the last day of counsel's review of the Oklahoma documents (September 21, 1984) that a second-stage review was planned by Kerr-McGee. In these circumstances, it was actually the People who timely objected to Kerr-McGee's proposal, allowing only the 2 days of the intervening weekend to pass before submitting a letter of objection to Kerr-McGee. If Kerr-McGee had indicated its intent to re-review the documents before producing them to the People for inspection, it seems likely that counsel for the People would not have engaged in the review until the "ground rules" for discovery had been resolved, either by stipulation or with the Board's assistance. After Kerr-McGee produced the Oklahoma City documents it could not unilaterally bind the People to an unconventional discovery routine by informing counsel at the close of her inspection of its intent to do a second review.

Counsel for Kerr-McGee maintains that counsel for the People acquiesced in the second-stage review. It is true that the West Chicago documents were inspected after the second stage was announced and objected to. In the face of the objection, Kerr-McGee should not have produced documents for inspection prior to a complete review and should have sought relief from the Board if necessary.[11]

Kerr-McGee also maintains that it asserted its privileges after inspection but before release and that this fact dictates that its privileges were preserved. We find that Kerr-McGee did not take adequate steps to preserve the confidentiality of these documents. In this circumstance, we

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11 We note that representatives of the People apparently continued their inspection of the Oklahoma City documents on the same day that counsel wrote objecting to the second-stage review. While we do not condone this practice, we find that it does not alter our conclusion. The other factors clearly outweigh this event.

do not believe that the fact that Kerr-McGee asserted its privileges prior to physically turning over the documents marked by the People is material. In short, we find that Kerr-McGee's claims of privilege for the ninety-two documents here in question have been waived.

SCOPE OF THE PEOPLE'S RESPONSE

Kerr-McGee has filed interrogatories and requests for documents on the Illinois Attorney General, counsel for the People of the State of Illinois and the Illinois Department of Nuclear Safety (collectively referred to as the "People"). The interrogatories and requests were directed to the State, and defined "State" to be:

> the State of Illinois and any departments or agencies of the State, as well as any employees, agents, consultants, contractors, or subcontractors of the State or any departments or agencies of the State.[12]

In response, counsel for the People produced documents of the Illinois Department of Nuclear Safety (IDNS) and Illinois Environmental Protection Agency (IEPA). Counsel took the position that

> the Attorney General, when representing particular agencies in litigation, produces the documents of only those agencies. The Attorney General's client agencies, i.e. those which have requested representation in either this or the related state court proceedings, are the [IDNS] and [IEPA]. Therefore only their documents were produced.[13]

Subsequently, counsel also produced documents from the Illinois State Geological and Water Surveys.

Kerr-McGee has moved for an order requiring counsel to respond to its requests with respect to all State agencies.[14] Staff supports this position.[15] The People continue to adhere to their position that only client agencies need respond.[16]

Section 2.741 of the Rules of Practice permits requests for production of documents to be filed only on parties. The Rules do not authorize requests to be filed on nonparties.

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[12] Kerr-McGee's Motion of November 30, 1984, at 2.
[13] People's Response of December 17, 1984, at 1-2.
[14] *See* note 3, *supra*.
[15] Staff's "Response to Kerr-McGee's and Illinois' Discovery Motions" of December 11, 1984.
[16] People's Response of December 17, 1984, at 1-2.

The People's petition to intervene in this proceeding was "filed on Petitioner's behalf by the Attorney General at the request of the [IDNS] and on his own motion."[17] Therefore, the IDNS is the only State agency which is a party to this proceeding, and consequently IDNS is the only State agency which must respond to requests for documents pursuant to § 2.741.[18] In this respect, the Rules of Practice are in accord with Federal practice. *See Trane Co. v. Klutznick*, 87 F.R.D. 473 (W.D. Wisc. 1980). Kerr-McGee's motion is denied.

This is not to say that document production may not be obtained from nonparties. Subpoenas issued pursuant to 10 C.F.R. § 2.720 may be used for this purpose.[19] *See Pacific Gas and Electric Co.* (Stanislaus Nuclear Project, Unit 1), ALAB-550, 9 NRC 683 (1979). Upon satisfactory application pursuant to § 2.720, the Board will issue subpoenas directing the production of documents by State agencies which have not responded to Kerr-McGee's requests.

STAY OF DISCOVERY

On December 7, Kerr-McGee moved for a stay of further discovery in this proceeding pending our rulings on the above discovery disputes and our rulings on motions for reconsideration of LBP-84-42, *supra*, filed by Staff and the People. The People oppose this motion; Staff has no objection to it.

Insofar as the motion sought to defer further discovery pending resolution of the above discovery disputes, it is now moot. And we see no reason to defer further discovery pending resolution of the motions for reconsideration.[20] Consequently Kerr-McGee's motion is denied.

In consideration of the foregoing, it is, this 9th day of January 1985,

ORDERED:

1. Kerr-McGee's motion for an order instructing the People to produce all relevant documents in the possession or control of the executive branch of the State is denied;

2. Kerr-McGee's motion for an order making clear that its privilege claims have been preserved with respect to ninety-two documents identified in Attachment A to its December 21, 1984 response to the People's motion for an emergency ruling is denied;

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[17] Petition to Intervene filed July 7, 1983, as amended February 29, 1984.

[18] Because counsel has never sought to add IEPA as a party to this proceeding, we assume that that agency is a party to the related State court proceeding.

[19] The Board has a supply of blank subpoenas which are available to the parties on request.

[20] We anticipate that our rulings on these motions will be issued shortly.

3. Kerr-McGee's motion for a protective order to protect confidential documents is denied as moot;

4. Kerr-McGee's motion for a stay of discovery is denied;

5. Kerr-McGee's motion for leave to file a reply to the People's response to its November 30, 1984 discovery motion is granted;

6. The People's motion for an order compelling Kerr-McGee to provide copies of the documents which counsel for the People inspected and marked for copying in Oklahoma City and West Chicago is granted;

7. The People's motion for an emergency ruling on the above discovery disputes is denied as moot; and

8. The People's motion for leave to file instanter a response to Kerr-McGee's motion for a stay in discovery is granted.

FOR THE ATOMIC SAFETY AND
LICENSING BOARD

John H Frye, III, Chairman
ADMINISTRATIVE JUDGE

Bethesda, Maryland
January 9, 1985

Cite as 21 NRC 24 (1985) LBP-85-2

UNITED STATES OF AMERICA
NUCLEAR REGULATORY COMMISSION

ATOMIC SAFETY AND LICENSING BOARD

Before Administrative Judges:

Charles Bechhoefer, Chairman
Dr. Frederick P. Cowan
Dr. Jerry Harbour

In the Matter of

Docket Nos. 50-329-OL&OM
50-330-OL&OM
(ASLBP Nos. 78-389-03-OL
80-429-02-SP)

CONSUMERS POWER COMPANY
(Midland Plant, Units 1 and 2)

January 23, 1985

The Licensing Board issues a Partial Initial Decision in a consolidated operating license/enforcement proceeding involving a facility as to which construction has been halted (but as to which the operating license application has not been withdrawn). The Decision resolves, subject to specified conditions or technical specifications, various technical issues arising out of the excessive settlement of soils upon which safety structures are founded. The Board also denies the Applicant's motion for reconsideration of an earlier order concerning the procedural steps which the NRC must follow when seeking to impose new seismic criteria on a facility at the operating license stage of review.

RULES OF PRACTICE: PARTIAL INITIAL DECISION

Although the conformance of a structure with applicable safety standards may depend both on the adequacy of design of the structure and on the manner in which the design is implemented, the adequacy of design is conceptually different from the sufficiency of design implementation and need not necessarily be considered in the same decision.

ATOMIC ENERGY ACT: LICENSING STANDARDS

The circumstance that construction is in progress (or has even been completed) cannot legally have any effect on a Licensing Board's evaluation of the adequacy of a structure's design. However, should problems with a design being followed be uncovered during construction, those problems may be taken into account in assessing the technical adequacy of the design. *Cf. Power Reactor Development Co. v. International Union of Electrical, Radio & Machine Workers*, 367 U.S. 396, 415 (1961).

SEISMIC AND GEOLOGIC CRITERIA: SCOPE OF INQUIRY

At the operating license stage of review, an applicant must provide, and the NRC Staff reviews, "current information . . . which has been developed since issuance of the construction permit, relating to site evaluation factors," including the geologic and seismic matters comprehended by 10 C.F.R. Part 100. 10 C.F.R. § 50.34(b)(1).

RULES OF PRACTICE: BACKFITTING

Where the NRC Staff seeks to apply new seismic criteria during its operating license review from those applied at the construction permit stage of review, and where there has been a progression in seismological review techniques in the intervening period, the Staff need not follow the backfitting procedures set forth in 10 C.F.R. § 50.109.

RULES OF PRACTICE: BACKFITTING

A progression in seismological review techniques may constitute "current information . . . which has been developed since issuance of the construction permit," within the meaning of 10 C.F.R. § 50.34(b)(1), thus calling for a reevaluation at the operating license stage of review without need to resort to the backfit standards of 10 C.F.R. § 50.109.

RULES OF PRACTICE: OPERATING LICENSE/SHOW CAUSE HEARINGS

Where an operating license and a show cause proceeding are being carried on simultaneously and are consolidated, and where the proceedings would utilize different procedural rules, the rules governing the operating license proceeding would apply in consolidated hearings on joint issues.

SEISMIC AND GEOLOGIC CRITERIA: GROUND MOTION

Use of site-specific response spectra to define the vibratory ground motion at a site of the safe shutdown earthquake is consistent with 10 C.F.R. Part 100, Appendix A, §§ IV(a), V(a)(1) and VI(a).

SEISMIC AND GEOLOGIC CRITERIA: SCOPE OF INQUIRY

The terms "important to safety" and "safety-related," when applied to seismic design requirements, are used interchangeably in 10 C.F.R. Part 100, Appendix A.

SEISMIC AND GEOLOGIC CRITERIA: SAFE SHUTDOWN EARTHQUAKE

An inadequacy in seismological data may warrant requiring, pursuant to 10 C.F.R. Part 100, Appendix A, § V(a)(1)(iv), that the controlling earthquake be larger than the maximum earthquake that has occurred historically within the tectonic province.

TECHNICAL ISSUES DISCUSSED

Dewatering
Differential settlement of structures
Ground acceleration value resulting from safe shutdown earthquake
Quality assurance
Safe shutdown earthquake (intensity; resulting vibratory ground motion)
Seismic design criteria
Seismic shakedown
Site-specific response spectra (SSRS)
Soil compaction
Soil density
Soil liquefaction
Structural design — cantilever designs
Structural design — evaluation of cracks

Tectonic provinces
Underground piping — corrosion
Underpinning of safety structures.

APPEARANCES

Michael Miller, Esq., JoAnne Bloom, Esq., Alan Farnell, Esq., Rebecca Lauer, Esq., Philip P. Steptoe, Esq., Anne West, Esq., Ronald G. Zamarin, Esq., Chicago, Illinois, **James Brunner, Esq.,** Jackson, Michigan, and **Frederick Williams, Esq.,** Washington, D.C., for Consumers Power Co., Applicant/Licensee.

Ms. Barbara Stamiris, Freeland, Michigan, *pro se,* and **Lynne Bernabei, Esq.,** Washington, D.C., for Ms. Barbara Stamiris, Intervenor.

Mr. Wendell Marshall, Mapleton, Michigan, *pro se,* Intervenor.

Ms. Mary Sinclair, Midland, Michigan, *pro se,* Intervenor.

William Paton, Esq., Michael Blume, Esq., Ellen Brown, Esq., Donald F. Hassell, Esq., Ann Hodgdon, Esq., James Thessin, Esq., Michael Wilcove, Esq., and **Nathene Wright, Esq.,** for the U.S. Nuclear Regulatory Commission Staff.

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PARTIAL INITIAL DECISION
(Remedial Soils Issues)

Opinion

I. INTRODUCTION

A. Nature of the Proceedings (Findings 1-3, 12)

This Decision represents the culmination of proceedings initiated more than 6 years ago. It involves a project which was novel — indeed unique — but which most likely will never come to fruition: namely, the proposed construction and operation by Consumers Power Company

(CPC or Applicant) of the Midland Nuclear Plant, Units 1 and 2. It reflects the difficulties (both monetary and technical) which were engendered by various quality assurance/quality control (QA/QC) deficiencies which have plagued the project from its inception. And it reflects the suspension of work on the partially completed project because of CPC's inability to finance its completion.

The issues before us arise from two consolidated proceedings: (1) the application of CPC for licenses to operate the Midland Plant, Units 1 and 2 (OL proceeding) and (2) the Order under 10 C.F.R. § 2.204 for modification of licenses, dated December 6, 1979 (OM proceeding).[1] The facility in question consists of two pressurized water nuclear reactors designed by Babcock & Wilcox Co. (B&W), located on a site on the south shore of the Tittabawassee River in Midland County, Michigan, adjacent to the Dow Chemical Company's main industrial complex in the city of Midland.

The facility's uniqueness stems from the once-planned usage of a large percentage of the capacity of Unit 1 (which had been scheduled to be the second unit completed) to produce process steam for the nearby Dow plant. Thus, as designed, Unit 2 would have produced 852 MWe whereas Unit 1 would have produced 504 MWe in addition to the process steam. However, reflecting delays and cost increases in the project, there developed a contractual dispute and litigation between Dow and CPC, and Dow gave up its plans to use the process steam. Thereafter, because of its inability to finance the project, CPC halted construction, first of Unit 1 and later the entire project.

The OL proceeding involves CPC's application for licenses to operate these two units. At present, the application has not been formally withdrawn, notwithstanding the halt of construction. The OM proceeding is a show-cause-type proceeding which eventuated from the discovery in July 1978 of excessive settlement of soils and structures (particularly the diesel generator building (DGB)). The two proceedings were consolidated (at the request of the Applicant) because of an overlap of certain issues raised in each of them.

The adjudication before us has produced an extensive record on many issues. The shutdown of construction on the project might arguably dictate our awaiting a motion to dismiss the OL application, without a ruling on the merits of any of the issues. This result in our view would not be in the public interest: among other things, it would render for

[1] With respect to the OM proceeding, CPC is a "licensee." Nonetheless, to avoid confusion, we shall refer to Consumers as CPC or Applicant, irrespective of the particular proceeding or proceedings to which the reference is applicable.

naught the extensive efforts devoted to these issues by the parties, their witnesses, and this Board. Moreover, absent withdrawal of the application for operating licenses, the proceeding is technically alive. Indeed, post-shutdown communications to the NRC have referred to the project in terms of "current deactivation." Letter, CPC to J.G. Keppler, NRC, dated July 27, 1984, file 0.4.9, serial 31797. Furthermore, CPC has advised us that, although "it is unlikely that the Midland project will be revived in the near future," the Company wishes to "preserve its options" and has no plans to withdraw its operating license application or to surrender its construction permits. Letter, CPC counsel to Board and parties, dated September 10, 1984. Accordingly, despite the potential mootness of the various issues before us, we nevertheless are issuing a decision on some of the technical issues which have been extensively litigated and which, if the project should ever be revived, might have some continuing applicability. We hope that our resolution of these issues will preclude the necessity for relitigation of the same issues if work on the project should ever be resumed.

On the other hand, the issues involving quality assurance/quality control (QA/QC) and management attitude, which have occupied the greatest amount of hearing time to date, focus in large part on the implementation of certain procedures and the performance and attitude of certain personnel. As such, they would appear to be of uncertain materiality, even if work on the project were ever to be resumed. Materiality would depend on the form and nature of the organization and the identity of the persons directing the resumed project. Given the announced indefinite suspension of the project, we do not intend to resolve those issues at this time. (In the Conclusions section of this Opinion, we offer a few observations on some of them.) Nothing herein should be taken as indicating that the project would be licensable absent resolution of any of those issues which remained pertinent to a revived project.

B. Identification of the Parties (Findings 6-7, 12)

Ms. Mary Sinclair, Mr. Wendell H. Marshall, and Ms. Barbara Stamiris were admitted as Intervenors in the OL proceeding. The Attorney General of the State of Michigan was admitted as an interested State (but has not actively participated in the proceeding to date). Ms. Stamiris and Ms. Sharon Warren were admitted as Intervenors in the OM proceeding (with Ms. Warren subsequently withdrawing). Reflecting both the overlap of certain issues between proceedings and the Commission policy to permit intervenors to conduct cross-examination and file proposed findings on issues raised by others, we permitted all of the Intervenors to participate in the development of the record and in the filing of proposed findings on any of the issues, whether nominally denominated as OL or OM issues. (Ms. Stamiris was the only Intervenor who filed proposed findings on the particular issues covered by this Decision.)

C. Procedural Posture of the Case (Findings 4, 7-17)

The OL adjudicatory proceeding commenced in May 1978, with the publication of a notice of opportunity for hearing. By a Special Prehearing Conference Order dated February 23, 1979 (unpublished), we accepted a number of Ms. Sinclair's proposed contentions, including one which raised safety questions concerning the excessive settlement of the diesel generator building (DGB).[2] We also accepted a similar contention of Mr. Marshall.

The OM proceeding was initiated on December 6, 1979, by the issuance by the Staff of an Order Modifying Construction Permits ("Modification Order"), a type of show-cause order. The Modification Order was based on the excessive settlement of the DGB (initially discovered in July 1978) caused by poor compaction of soils on which it was constructed, the QA/QC practices which permitted such poor soils compaction to have occurred, and the potential that similar inadequate compaction practices may have been utilized with respect to other safety structures founded in whole or in part on fill materials. The Modification Order would have suspended all soils-related and remedial work on the Midland facility until the related safety issues were resolved and construction permit amendments for the soils remedial work were submitted by CPC and approved by the Staff. Through its December 26, 1979 request for a hearing, CPC stayed the effectiveness of the Modification Order pending conclusion of the OM proceeding.

Under the Modification Order, the broad issues (which were put into contest by virtue of CPC's request for a hearing) are (1) whether the facts set forth in Part II of the Order (setting forth the factual basis for the Order) are correct, and (2) whether the Order should be sustained (i.e., the specific relief put into effect). In addition, in response to an Amended Notice of Hearing published in May 1980, two Intervenors (Ms. Stamiris and Ms. Warren) were admitted to the OM proceeding. We accepted a number of contentions sponsored by each of them in our Prehearing Conference Order of October 24, 1980 (unpublished). Because of the overlap of Ms. Sinclair's and Mr. Marshall's OL contentions

[2] All soils-related contentions, whether or not dealt with or resolved in this Decision, are set forth *infra* in Appendix A.

relating to the DGB settlement and the issues that had been subsequently raised by the Modification Order (including certain contentions of Ms. Stamiris and Ms. Warren), we also granted CPC's request that we consolidate the two proceedings. Ms. Warren subsequently withdrew from the proceeding (although she made a limited appearance statement). Since her issues were encompassed within the broader OM issues, we asked the parties to address the substance of her contentions, and they have done so. *See infra* note 41. Later, we accepted several additional late-filed OM contentions sponsored by Ms. Stamiris, engendered by the litigation between Dow and CPC. LBP-84-20, 19 NRC 1285 (1984).

Hearings on soils-related OM-OL issues commenced in July 1981, and extended intermittently through December 3, 1983, utilizing 96 hearing days. (In addition, 9 days of hearings on strictly OL issues were held in March and April 1983.) Limited appearance statements from members of the public were accepted at several of the hearing sessions.

Two general types of soils issues are involved in the OM-OL consolidated proceeding: those which question the QA/QC performance and managerial attitude of CPC or its contractor, Bechtel (most of Ms. Stamiris' contentions) and issues involving the technical aspects of remedial soils activities (the remainder of Ms. Stamiris' and all of the other Intervenors' soils-related contentions). The Applicant and NRC Staff have often been in disagreement on both types of issues, although currently they generally agree with respect to most of the technical aspects of the remedial soils activities.

Early in this proceeding, prior to the close of the record on the technical aspects of remedial soils activities, we had planned to issue a Partial Initial Decision on QA/management attitude issues, followed by another decision covering the technical adequacy of the remedial soils activities (or "fixes"). Notwithstanding that plan, we found it necessary to reopen the record twice on QA/management attitude issues — the first time at the instance of Ms. Stamiris, and the second time at the request of the NRC Staff. Prior to the most recent closing of the record on QA/management attitude issues, we completed hearings on the technical aspects of the remedial "fixes." Proposed findings and conclusions on those technical issues were submitted by CPC, Ms. Stamiris, and the NRC Staff.[3] Although we could possibly have issued an Initial Decision cover-

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[3] Applicant's Proposed Findings of Fact and Conclusions of Law (FOF) on Remedial Soils Issues, dated August 5, 1983 (hereinafter App. FOF); NRC Staff Responsive Findings, dated November 15, 1983 (Staff FOF); Intervenor [Stamiris] Proposed FOF, dated December 16, 1983 (Stamiris FOF); Applicant's Replies to Staff and Stamiris FOF, each dated January 3, 1984 (App. Reply to Staff (Stamiris)

(Continued)

ing both QA/management attitude and the technical aspects of the remedial "fixes," considerations of timing and length, as well as the recently announced suspension of work on the facility, have caused us to adhere to our earlier plan of separating the decisions on QA/management attitude and on the technical aspects of remedial soils activities.

After the record on QA/management attitude issues had been closed for the second time (i.e., before the most recent reopening of the record), and during the course of our preparation of a decision on that subject, we determined it to be necessary to issue an order imposing interim conditions on further soils-related construction activities, pending completion of our decision. In our Order of April 30, 1982, we required, *inter alia*, that the Applicant obtain explicit prior approval from the NRC Staff (with limited exceptions) before proceeding with further soils-related construction activities (as defined therein). Memorandum and Order (Imposing Certain Interim Conditions Pending Issuance of Partial Initial Decision), LBP-82-35, 15 NRC 1060. In other words, soils-related construction activities were halted in the absence of authorization by the NRC Staff. Thus, the effect of that Order in substance was to sustain, on an interim basis, the requirements of the Modification Order, except with respect to the submission and approval of amendments to the construction permits, a procedural step which in our opinion was not necessary to attain the safety goals which we believed should be achieved. In order to comply with the requirements of LBP-82-35, CPC put into effect, *inter alia*, its "Work Authorization Procedure."

The conditions imposed on the Applicant by LBP-82-35 were motivated by QA (including QC) considerations. As a result of the subsequent reopening of the record on QA/management attitude matters, and more recently the project shutdown, we have not issued the decision which would supersede those interim conditions. Accordingly, to the extent that any soils-related construction were to be resumed, they continue in effect. This Partial Initial Decision does not generally treat QA or management attitude issues and has no effect on those interim conditions.

FOF). Unless otherwise specifically pointed out, references to various parties' proposed findings will be to those on remedial soils issues, as catalogued in this footnote.

References to all parties' proposed findings (FOF) will be to the paragraph numbers and/or pages. Since Ms. Stamiris' FOF did not include numbered paragraphs, we have numbered each paragraph of her findings consecutively (¶¶ "1-27"), for ease of reference. Thus, the first paragraph under "Introduction" is ¶ "2", the first paragraph under "The Soils Remedial Fixes" is ¶ "6."

D. Summary of Decision (Findings 17-18)

This Partial Initial Decision deals with the technical adequacy of the remedial soils activities which have been proposed by CPC. The subjects covered are seismic matters (including the appropriate safe shutdown earthquake, standards for the proposed seismic margin review, soil liquefaction and dewatering), the designs and plans for assuring the structural adequacy of the auxiliary building (except with respect to differential settlement of the control tower relative to the main building), the service water pump structure (SWPS), the borated water storage tanks (BWSTs), the diesel fuel oil tanks (except with respect to liquefaction and soils stability), underground piping, underground electrical duct banks and conduits, and baffle and perimeter dikes. For reasons stated below (*see infra* p. 37), we are not making any findings with respect to the DGB; we are, however, including a general description of the problems and corrective actions associated with that structure.

In her proposed findings on remedial soils issues, Ms. Stamiris takes the position that the Applicant's remedial program is only a "paper" program and that CPC's problems have always been "not with their conceptual programs, but with the implementation of those programs" (Stamiris FOF, ¶ "6," at 2, citation omitted). She asserts that technical findings should be considered only along with findings concerning implementation and that our decisions on these subjects should be combined (*id.* ¶ "9," at 3-4). She also implies that the status of ongoing plant construction must of necessity influence our rulings on the adequacy of the various remedial fixes.

It is obvious, of course, that CPC has suffered through numerous serious QA/QC implementation problems in the past. The issuance of LBP-82-35 is but one reflection of those problems. Indeed, it is apparent that the soils settlement problems stem in large part from a QA deficiency: the failure of the Applicant or its contractor to have had available a qualified geotechnical engineer with authority to control soils placement during the time when the fill soils were being compacted — despite a previous commitment to the NRC to utilize a geotechnical engineer for such purposes (*see infra* p. 111). Both theoretically and practically, therefore, the question of the conformance of the facility with applicable safety standards depends not only on the adequacy of design but also on the implementation of those designs. No party to this proceeding contends otherwise.

That does not mean, however, that design and implementation must necessarily be considered in the same decision. The adequacy of design is conceptually different from the sufficiency of design implementation.

If the design turns out to be consistent with applicable requirements, the adequacy of implementation still remains an open question. (If the design is inadequate, however, the sufficiency of implementation becomes irrelevant.) Moreover, contrary to Ms. Stamiris' apparent claim, the circumstance that construction was in progress (or had even been completed) could not legally, and did not, have any effect on our evaluation of the adequacy of design in this Decision. There is but one exception to this general approach: if, during construction, problems with the design being followed were uncovered, those problems were factored into our decision on the technical adequacy of the remedial soils measures. *Cf. Power Reactor Development Co. v. International Union of Electrical, Radio & Machine Workers*, 367 U.S. 396, 415 (1961).

We have factored problems revealed during the course of construction into our consideration of two of the technical subjects on which CPC has submitted proposed findings: the structural adequacy of the DGB and the effects of differential settlement of the control tower relative to the main auxiliary building. As a result of greater-than-expected cracking in the DGB, the Staff undertook further studies and evaluations of the DGB's structural adequacy and also moved to reopen the record on that question (Tr. 22,678-83). Although we had not yet determined prior to the halt in construction whether to reopen the record on the DGB, and were awaiting a further Staff report before we made that determination, we permitted Ms. Stamiris and the Staff to defer filing proposed findings and conclusions on the DGB remedial measures (Tr. 22,687). We are accordingly excluding from this Decision any consideration of the adequacy of the remedial soils activities associated with the DGB. (Since this Decision may turn out to be our last major decision in these proceedings dealing with substantive issues, we are including a general description of the problems and corrective actions associated with the DGB. *See infra* pp. 81-86.)

Similarly, Ms. Stamiris has pointed to Board Notification BN 83-174, dealing with the corrective actions utilized for the auxiliary building, particularly the effects of differential settlement between the control tower and the main auxiliary building; she sought to reopen the record, *inter alia*, on open items in the Board Notification (Tr. 22,672; Stamiris FOF, ¶ "13," at 5). Although we denied Ms. Stamiris' motion as premature (Tr. 22,675-76), we agree that, in the absence of further information on the questions raised in BN 83-174, the record is not complete enough to cause us to rule on whether the proposed remedial measures for the auxiliary building adequately take these aspects of differential settlement into account. For that reason, we are also excluding from this Decision any evaluation of that subject.

In addition, we have taken into account incomplete or erroneous information discovered during the pendency of the soils hearings in our evaluation of two other technical subjects: the soil spring constants proposed to be used in a seismic reevaluation of various structures, and the assessment of soil liquefaction potential and soils stability under the diesel fuel oil tanks. Through Board Notification BN 84-115, dated June 18, 1984, we were advised by the Staff of the Applicant's discovery, during a design review, of a deficiency in the original seismic design of certain Seismic Category I structures. This deficiency in particular would affect the analysis of the auxiliary building and the SWPS. With respect to those structures, our findings and conclusions reflect this outstanding open question. *See* discussion, *infra* pp. 70-71, 90-91, 94-95, 98 and Findings 88-89, 141, 151, 164, 166.

Finally, on November 21, 1984, CPC submitted a report to the Staff (with copies to the Board and parties) advising that certain logs of borings assertedly taken in the area of the Midland diesel fuel oil tanks were in fact logs of borings taken elsewhere in the Midland area. By letter dated December 6, 1984, the Applicant advised that the only technical issue potentially affected was that of liquefaction of soils below the diesel fuel oil tanks. The Applicant regarded the record on this question to be "inconclusive." In its response dated December 21, 1984, which included the affidavit of Mr. Joseph Kane, a geotechnical witness, the Staff agreed that its analysis of liquefaction beneath the diesel fuel oil tanks would be affected but added that other technical issues might also be affected (*see infra* pp. 103-04). In her December 24, 1984 response, Ms. Stamiris took the position that the erroneous boring logs, which had been discovered during the Dow-CPC litigation, represented only one example of erroneous information uncovered in that litigation. She cited other examples bearing upon several of her QA/management attitude issues. She requested that we order an investigation by the NRC Office of Investigations (OI) and that, before issuing any decision depending in whole or in part on information provided or sponsored by CPC, we hold a further evidentiary hearing on facts surrounding the disclosure of the erroneous soil boring data. The Staff did not mention further hearings but indicated that further inquiry on this subject might be warranted.

Based on the state of the record, we are at this time making no findings concerning liquefaction or soils stability relative to the diesel fuel oil tanks, nor are we reaching any "reasonable assurance" conclusions concerning the tanks. We regard the matters as to which Ms. Stamiris seeks further hearings (i.e., "Dow" issues) as essentially QA/management attitude matters, on which we are not now ruling. As set forth *infra* p. 40, we are leaving open the possibility (following submission of a status report by CPC and responses of other parties) of further hearings. In the Board's view, the circumstances underlying the NRC Staff's "extreme difficulty" in understanding how the "mix-up" in boring logs occurred suggests that new hearings may very well be warranted, at least in the event a restart of construction is proposed. Kane Affidavit, dated December 21, 1984, ¶ 3, at 4.

We have no authority to order an OI investigation (Stamiris Exh. 135, Policy 4); the Staff, of course, could — and perhaps should — do so. In any event, to permit us to consider newly discovered information derived from the Dow-CPC litigation bearing upon issues covered by this Decision, we are retaining jurisdiction to reopen the record to modify any of our determinations which may be significantly affected thereby.

With respect to the matters we are considering, and for reasons hereinafter set forth, we conclude that the remedial soils measures proposed by CPC and accepted by the Staff are generally satisfactory, subject in certain instances to the imposition of appropriate technical conditions or specifications. Assuming that the remedial soils activities would have been correctly carried out, and that open technical questions would have been satisfactorily resolved, we would have had reasonable assurance that the structures on which we are ruling in this Decision would pose no undue risk to the public health and safety. If the project is ever revived, the manner in which the structures and soils remedial activities have been or would be implemented, as well as the design aspects of the DGB, auxiliary building, SWPS and diesel fuel oil tanks on which we are not now ruling, would remain as open questions, subject to further decision or litigation or relitigation, as appropriate.

In the body of this Decision we discuss our concerns regarding the deficiency inherent in the stepped-foundation design of portions of the auxiliary building, the SWPS and the borated water storage tanks. *See infra* pp. 93-94, 102. It is apparent that the differential settlement of these structures was the result of the overall settlement of the soil. However, there is evidence that stepped-foundation designs have the potential for developing problems even when built on properly compacted backfilled soil, because of cantilever and bending moment stresses that could result from greater-than-anticipated soil settlement. We are recommending that the NRC Staff study, generically, the acceptability of the future use of such stepped-foundation designs in safety-related structures.

As for the status of these proceedings, the Applicant, through its letter of September 10, 1984, has proposed that no further hearings be held at this time, that its current obligation to forward audit and nonconformance reports to the Board and parties be discontinued, and that it file an additional report on the status of the project in 6 months. In a document dated October 24, 1984, which we are treating as a response to the request concerning documents, Ms. Sinclair raised certain questions concerning the propriety of discontinuing reporting requirements as long as the construction permits and OL application remain active. *See* Memorandum and Order dated November 2, 1984 (unpublished). In its October 26, 1984 response, the Staff agreed that hearings at this time would not be productive but suggested that the Applicant include a recommendation as to future hearings in its status report. The Staff also suggested a conference call with respect to the discontinuance of reporting requirements. The call was held on November 7, 1984, and it was agreed that the Applicant and Staff would consult on the reporting question (as well as the related question of the types of data which should continue to be collected while construction is suspended) and report back to us early in 1985. For the interim, we reduced the number of copies of audit and nonconformance reports which need to be supplied to the Board. *See* Memorandum dated November 8, 1984 (unpublished).

We agree that no further hearings should be held in the near future and that the Applicant should file a 6-month status report. Such report should include recommendations as to future hearings. In particular, it should outline information discovered in the Dow-CPC litigation which would affect these proceedings, as to which Ms. Stamiris seeks further hearings. Such report should be filed on or before April 1, 1985. Parties may respond within 10 days of service (15 days for the Staff). Notwithstanding this schedule, the Applicant should notify us promptly of any significant developments, including but not limited to plans or proposals for the restart of construction. Pending our receipt of a report during early 1985 on the questions outlined in our November 8, 1984 Memorandum, we take no action on CPC's request to eliminate certain reporting, except to reduce the number of copies of audit and nonconformance reports which must be furnished to the Board.

In the future, following receipt of CPC's status report, and responses thereto, we expect to confer with (or otherwise seek the views of) the parties as to whether, and if so when and how, these proceedings should be continued or terminated. In particular, we will consider whether we should issue a further decision (or conduct further hearings) on any issues remaining unresolved after this Decision (including the various QA/management attitude issues). We invite the suggestions of the parties on the potential resolution of such open issues.

II. SEISMIC MATTERS

A. Legal Standards (Findings 19-36)

Several regulations specify the seismic and geologic criteria to which the design of nuclear power plants must adhere. In general, "[s]tructures, systems, and components important to safety" are required to be "designed to withstand the effects of natural phenomena such as earthquakes . . . without loss of capability to perform their safety functions." 10 C.F.R. Part 50, Appendix A, GDC 2. The specific design criteria are set forth in 10 C.F.R. Part 100, Appendix A (Seismic and Geologic Siting Criteria for Nuclear Power Plants). The Final Safety Analysis Report (FSAR) submitted in support of the operating license application must include, *inter alia*, "current information . . . which has been developed since issuance of the construction permit, relating to site evaluation factors identified in Part 100 . . . ." 10 C.F.R. § 50.34(b)(1).

The construction permits for the Midland Plant were issued by the Atomic Energy Commission on December 15, 1972.[4] That date followed the publication of the proposed Appendix A to 10 C.F.R. Part 100 (36 Fed. Reg. 22,601 (Nov. 25, 1971) but preceded the issuance of the final rule, which was published on November 13, 1973 (38 Fed. Reg. 31,279) and became effective on December 13, 1973. When it published its proposed rule, the Commission (AEC) set forth its expectation that "the proposed amendments will be useful as interim guidance until such time as the Commission takes further action on them." 36 Fed. Reg. at 22,601.

At the construction permit stage, the Staff's review of the applications, as set forth in the Staff's "Safety Evaluation" dated November 12, 1970 (CP "SER"), preceded the issuance of the proposed as well as the final versions of 10 C.F.R. Part 100, Appendix A. As a result, the Staff in its review did not utilize certain of the criteria which were adopted through issuance of Appendix A (e.g., delineation of a tectonic province);[5] nor did the Licensing Board which authorized the issuance of construction permits, even though its decision followed the promulgation of the proposed Appendix A.[6]

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[4] Pursuant to the Energy Reorganization Act of 1974, as amended, 42 U.S.C. § 5801, *et seq.*, the Atomic Energy Commission (AEC) was abolished, and the Nuclear Regulatory Commission (NRC) assumed the AEC's licensing and related regulatory functions.

[5] *See* further description of the Staff's CP review criteria, *infra* p. 51 and Finding 21.

[6] During the CP hearings, no issue was raised about the seismic or geologic analyses which had been undertaken. In its normal CP review, the Licensing Board probably did not use the proposed Appendix A as guidance, inasmuch as it merely approved the Staff's seismic and geologic conclusions as reflected in the CP "SER." LBP-72-34, 5 AEC 214, 219-20 (1973), *aff'd*, ALAB-123, 6 AEC 331 (1973).

The OL application, as represented by the FSAR, was filed in 1977, after the effective date of Part 100, Appendix A. It incorporated a seismic analysis which followed the procedures of Appendix A, including a proposed tectonic province for the Midland site. The analysis resulted in the same maximum earthquake as had been approved at the CP stage, with terminology changed to reflect that utilized in Appendix A — e.g., the design basis earthquake (DBE) at the CP stage became the safe shutdown earthquake (SSE)[7] described in the FSAR (FSAR SSE). The FSAR proposed design response spectrum (modified Housner) was the same as the DBE response spectrum at the CP stage.

During the course of its OL review, however, the Staff began to doubt whether the CP earthquake (DBE or proposed FSAR SSE) was adequate and consistent with the requirements of Appendix A. The Staff's concerns in this regard were set forth in a letter dated October 14, 1980 from Robert L. Tedesco, Assistant Director for Licensing, to Mr. J.W. Cook, CPC Vice President, re: Seismological Input for the Midland Site (Holt Exh. 3; hereinafter "Tedesco letter").[8] That letter offered CPC two alternatives for characterizing the SSE, both of which, according to the Staff, are consistent with the Staff's Standard Review Plan (SRP, NUREG-0800, not introduced into evidence):

1. The largest historic earthquake in the Central Stable Region tectonic province, assumed to occur "near the site," with ground acceleration based upon the standardized response spectra of Regulatory Guide 1.60 anchored at 0.19*g*.
2. The "site-specific response spectra" (SSRS) approach using the magnitude of the same highest earthquake with epicentral distances assumed to occur less than 25 km from the site, and using the 84th percentile of the response spectra as derived directly from real time histories.

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[7] "Safe Shutdown Earthquake" is defined as

> that earthquake which is based upon an evaluation of the maximum earthquake potential considering the regional and local geology and seismology and specific characteristics of local subsurface material. It is that earthquake which produces the maximum vibratory ground motion for which certain structures, systems, and components are designed to remain functional. These structures, systems, and components are those necessary to assure:
>
> (1) The integrity of the reactor coolant pressure boundary,
>
> (2) The capability to shut down the reactor and maintain it in a safe shutdown condition, or
>
> (3) The capability to prevent or mitigate the consequences of accidents which could result in potential offsite exposures comparable to the guideline exposures of this part.

10 C.F.R. Part 100, Appendix A, § III(c).

[8] Mr. Richard J. Holt, one of the Applicant's witnesses, submitted 11 exhibits which the Board accepted into evidence in connection with his prepared testimony. These exhibits ranged from single-page figures to multi-page reports with their own figures and tables. The Holt exhibits were not bound into the transcript, but are part of the evidentiary record. Tr. 4538-40, 4550-51, 5117-18. These exhibits are hereinafter referred to as "Holt Exh. __."

The Applicant elected the SSRS approach. It further agreed to design remedial structures to this standard (or what it viewed as the equivalently conservative 1.5 x FSAR SSE standard) and to conduct a seismic reevaluation or "seismic margin review" to determine whether various Seismic Category I structures which had already been constructed could conform to the newly ascertained SSE. This study had commenced but, insofar as we are aware, had not been completed (or reviewed by the Staff) prior to the shutdown of construction.

Early in this proceeding, shortly following its receipt of the "Tedesco letter," the Applicant moved that we defer consideration of all seismic issues until the later, OL, portions of the hearing. The Board believed that to have done so would have required us to evaluate the planned construction of structures, such as underpinnings and new foundations, on the basis of potentially invalid criteria, i.e., essentially the same seismic criteria as those approved during the CP stage (which were not materially changed by the Applicant's proposed FSAR SSE). The Applicant and Staff reached an agreement, which we had encouraged and thereafter accepted, for a schedule under which (1) the establishment of seismic criteria, including determination of the SSE, ground motions and associated response spectra, and (2) the analysis model for each structure as modified by the remedial actions would be heard during the early hearings on soils-related (OM) issues. This would have left for the later stages of this consolidated OL-OM proceeding the question of whether the safety-related structures as built (including those with and those without modifications necessitated by the soils remedial actions) conformed to the newly determined seismic criteria. *See* Applicant's Motion to Defer Consideration of Seismic Issues Until the Operating License Proceeding, dated March 18, 1981; Stamiris' Response, dated April 6, 1981; Staff's Response, dated April 7, 1981; Prehearing Conference Order (Ruling upon Applicant's Motion to Defer Consideration of Seismic Issues Until the Operating Licensing Proceeding and upon Other Matters), dated May 5, 1981 (unpublished). For these reasons, we are not ruling in this Decision on whether various safety structures built under DBE or FSAR SSE standards in fact conform to the standards required by the new SSE.

Two significant legal questions have surfaced by virtue of the Applicant's election to utilize the SSRS approach -- namely, the procedures which the Staff must follow to require structural changes based on that approach, and the consistency of the SSRS approach with the requirements of Part 100, Appendix A. We turn now to these questions.

(1) Procedures for Applying the SSE in OL Review (Applicant's Motion for Reconsideration)

In its March 18, 1981 scheduling motion mentioned above, the Applicant took the position that the application of new seismic criteria to the Midland facility is and should be governed by the backfit requirements of 10 C.F.R. § 50.109.[9] Although the major thrust of the motion concerned the scheduling of seismic issues, the Applicant's view of the difficulty of resolving the seismic issues in a timely fashion was based in large part on its position that, because a DBE had been formally established at the CP stage, a change in the applicable seismic criteria would be a "backfit" decision which, pursuant to 10 C.F.R. § 50.109, would require a cost-benefit type of finding to the effect that such action will provide "substantial, additional protection which is required for the public health and safety . . . ."

Both the NRC Staff and Ms. Stamiris opposed that motion. At a prehearing conference on April 27, 1981, we resolved the scheduling aspects of the motion by accepting the Applicant-Staff agreement described *supra* p. 43. In doing so, however, we specifically rejected the Applicant's proposal to consider changes in seismic design only under the backfitting criteria of 10 C.F.R. § 50.109. Our ruling appears in our Prehearing Conference Order dated May 5, 1981 (unpublished), at 2-12.

The Applicant now seeks reconsideration of our ruling insofar as it holds that the backfitting criteria need not be utilized (App. FOF, ¶ 498). Other parties did not respond to this motion, although the Staff commented that it would not respond unless the Board specifically requested it to do so (Staff FOF at 53 n.12). (We made no request.)

In our view, the Applicant's motion for reconsideration presents no information which we had not already considered, and provides no persuasive reason for us to change the basis or result of our earlier ruling. We are therefore declining to do so.

However, we wish to reiterate our view that Commission regulations and practices contemplate a separate review at the OL stage of site

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[9] That section reads, in relevant part

§ 50.109 Backfitting

(a) The Commission may, in accordance with the procedures specified in this chapter, require the backfitting of a facility if it finds that such action will provide substantial, additional protection which is required for the public health and safety or the common defense and security. As used in this section, "backfitting" of a production or utilization facility means the addition, elimination or modification of structures, systems or components of the facility after the construction permit has been issued.

(b) Nothing in this section shall be deemed to relieve a holder of a construction permit or a license from compliance with the rules, regulations, or orders of the Commission.

factors, including geology and seismicity, particularly where new information has developed since the CP stage of review. The FSAR must include all "current information . . . which has been developed since issuance of the construction permit, relating to site evaluation factors identified in Part 100 . . . ." 10 C.F.R. § 50.34(b)(1). Those factors include the geologic and seismic matters comprehended by Part 100 (particularly Appendix A).

As we pointed out in our May 5, 1981 Prehearing Conference Order, the Staff attributed its reasons for the DBE reevaluation to "a progression during the last ten years in the state-of-the-art with respect to seismology (Tr. 867-869)" (Order at 5). Elsewhere in this Decision, we describe some of the substantial differences in the criteria utilized at the CP stage and those which the Staff is currently following. Among other matters, no tectonic province was ever developed at the CP stage. By including it in its FSAR, the Applicant has implicitly recognized the developing nature of the Staff's seismic criteria and the necessity for incorporating such criteria into the OL review. Further, the Staff regards the design response spectrum utilized during the CP review for ascertaining ground motion (modified Housner) as insufficiently conservative; and, for reasons expressed later in this Decision (*infra* pp. 67-68, Finding 71), we agree. We conclude that the progression in seismological review techniques constitutes "current information . . . which has been developed since issuance of the construction permit," within the meaning of 10 C.F.R. § 50.34(b)(1), thus calling for a reevaluation at the OL stage without need to resort to the backfit standards of 10 C.F.R. § 50.109.

We note that, in our Prehearing Conference Order, we pointed to the use of the backfit criteria as a type of enforcement activity. The Applicant now states (App. FOF, ¶ 498, at 313) that this case is in part an enforcement matter and that the seismic issue was raised in that context as well as in the OL context. If the new seismic criteria were sought to be applied only in an enforcement context, then the procedures required by 10 C.F.R. § 50.109 might well have to be applied. But where, as here, the OL review provisions of 10 C.F.R. § 50.34(b)(1) come into play, they supersede the procedures applicable only in enforcement situations.[10]

Finally, we would agree with the Applicant that, despite its agreement with the Staff to perform the seismic margin review using an SSRS SSE,

[10] The OL provisions would apply in any enforcement proceeding carried on during the pendency of an OL application. *Cf. Consumers Power Co.* (Midland Plant, Units 1 and 2), ALAB-283, 2 NRC 11, 17-18 (1975), *clarified*, ALAB-315, 3 NRC 101 (1976) (burden of proof). Thus, our view has not been influenced by the consolidation here of the OM and OL proceedings.

the procedures to be employed in applying the results of the new seismic review to this facility make a difference: in the words of the Applicant, "the Seismic Margin Review results may lead the Staff to require modifications which Applicant is unwilling to make" (App. FOF, ¶ 498, at 312 n.827). If that situation were to occur, the Applicant could still challenge the Staff's determination. But the decisional criteria would be the normal OL review criteria, not the backfit standards of 10 C.F.R. § 50.109.

(2) Compatibility of SSRS Approach with 10 C.F.R. Part 100, Appendix A (Finding 34)

Prior to the hearings concerning seismic issues relating to the choice of an SSE and related ground motion, and as a result of the option afforded by the Tedesco letter (and later accepted by the Applicant) to utilize the SSRS approach, we asked the Applicant and Staff (and permitted other parties) to file briefs addressing the compatibility of the SSRS approach with the requirements of 10 C.F.R. Part 100, Appendix A (in particular, ¶¶ V(a)(1)(ii) and (iv) of the Appendix). *See* Memorandum dated August 18, 1981 (unpublished). The Applicant and Staff, each filed responses on September 29, 1981 (hereinafter App. Brief or Staff Brief); and each asserted that, as used at Midland, the SSRS approach was consistent with the requirements of Appendix A. The Applicant and Staff, respectively, reiterated that position in their proposed findings (App. FOF, ¶¶ 8-16; Staff FOF, ¶¶ 8-16). For the reasons which follow, we agree with that conclusion.

Appendix A to 10 C.F.R. Part 100

> describes the nature of investigations [currently] required to obtain the geologic and seismic data necessary to determine site suitability and to provide reasonable assurance that a nuclear power plant can be constructed and operated at a proposed site without undue risk to the health and safety of the public. It describes procedures for determining the quantitative vibratory ground motion design basis at a site due to earthquakes . . . .

10 C.F.R. § 100.10(c)(1); *see also* 10 C.F.R. Part 100, Appendix A, § II. In general, the Appendix A criteria and procedures provide for determination of the appropriate SSE and of the ground motion which that earthquake would generate at the site. General elements of investigation contained in Appendix A for determining the SSE and its representative ground motions where (as here) no capable faults (or similar tectonic structures with which historical earthquake activity can be reasonably correlated) exist within the vicinity of the site, are (1) determi-

nation of the tectonic province in which the site is located, (2) determination of the size and ground motions of the controlling earthquake within that tectonic province, (3) determination of the size and ground motions, at the plant site, of earthquakes associated with distant tectonic structures and those associated with adjacent tectonic provinces, and (4) definition of the response spectra corresponding to the maximum vibratory ground accelerations at the various foundation levels of safety-related structures on the plant site, as derived from the determinations in steps (2) and (3).

Because the data upon which the Appendix A investigations are founded are historical and geologic in nature, the procedures of Appendix A have been characterized as "deterministic" rather than "probabilistic." At the time of our August 18, 1981 Memorandum, there was controversy over the extent to which the use of probabilistic methodology was permissible under Appendix A. *See Public Service Co. of New Hampshire* (Seabrook Station, Units 1 and 2), CLI-80-33, 12 NRC 295, 298 (1980); *cf. id.*, ALAB-667, 15 NRC 421, 426-42 (1982). For that reason, we specifically inquired whether the Applicant's methodology for determining the SSE and its ground motions satisfied certain of Appendix A's requirements. Although not explicitly stated in our Memorandum, the aspects of the cited Appendix A criteria that we perceived to have the greatest potential incompatibility with probabilistic determinations, depending upon how those determinations were made, were:

(1) how the requirement that the determinations be carried out in a conservative manner would be treated;

(2) how probabilistic or statistical *averages* of ground motions would be reconciled with the often-used requirement that *maximum* vibratory ground motions be determined and applied; and

(3) how both the requirements that the controlling earthquake in the site's tectonic province be assumed to occur *at the site* and that effects of *more distant* earthquakes would be accounted for; and the related question,

(4) what data or techniques would be applied to assure that the maximum vibratory acceleration at the site *throughout the frequency range of interest* is included.

It is in the definition of the vibratory ground motion associated with the SSE (i.e., defining a response spectrum) where the SSRS methodology is being used at Midland. Appendix A requires that the "vibratory ground motion produced by the Safe Shutdown Earthquake shall be defined by response spectra corresponding to the maximum vibratory accelerations at the elevations of the foundations of the nuclear power plant structures . . ." (10 C.F.R. Part 100, Appendix A, § VI(a)). A response spectrum (defined in Appendix A, § III(l)) is "a plot of the maximum responses (acceleration, velocity or displacement) of a family of idealized single-degree-of-freedom damped oscillators against natural frequencies (or periods) of the oscillators to a specified vibratory motion input at their supports." (*See* note 59, *infra* p. 137, for additional explanation of response spectra.) The regulations further require that the spectra represent an appropriately conservative description of motions associated with the SSE throughout the frequency range relevant to the design of a nuclear facility (Appendix A, § V(a)(1)(iv)), but they do not specify the methodology for deriving the required spectra. They do require that seismology, geology, and seismic and geologic history of the site and surrounding region, and the characteristics of the underlying soil material in transmitting earthquake-induced motions, be taken into account (Appendix A, § V(a)).

The Staff currently regards at least two different methodologies for representing vibratory ground motion as acceptable — the standardized response spectrum, as defined in Regulatory Guide 1.60 (*see infra* note 49), and the SSRS. As described by the Staff, the Reg. Guide 1.60 approach is a standardized spectrum derived from strong motion records of a large number of earthquakes of various magnitudes, recorded at various distances and on varying site conditions. The ground motion values of these records were normalized to the same acceleration, a spectral shape was derived representing the mean plus one standard deviation, and, after some smoothing, the response spectrum became the standardized Reg. Guide 1.60 spectrum. Although it can be used at a wide range of sites to define the vibratory ground motion of a large variety of earthquake intensities, it does not depend on the characteristics of any one site to which it is applied. When used, the Reg. Guide 1.60 spectrum is scaled to the ground acceleration level associated with the intensity of the site's SSE. Staff Brief at 10-11.

On the other hand, according to the Staff, the SSRS methodology takes into account more closely the seismology and geology of the site and surrounding region and the engineering properties of the soil. As described by the Staff:

> The principle underlying the use of a site-specific response spectrum is straightforward. Because earthquakes of similar magnitudes have been found to have similar ground motion characteristics when recorded at similar distances from the epicenter and in similar soil conditions, an accurate representation of possible ground motion for an earthquake of a postulated magnitude can be derived from analyzing an adequate set of recordings for similar-magnitude earthquakes at similar sites elsewhere. To make this comparison, the data base for strong motion records is searched for all recordings of historical earthquakes of similar magnitude to the chosen safe shutdown earthquake recorded close to the epicenter of the event and

> recorded in similar geologic conditions. If the ensemble of recordings fitting these parameters is of sufficient size then the ground motion data for each of the records are plotted, and an idealized spectrum is drawn representing a mean-plus-one-standard-deviation. This idealized spectrum is the response spectrum specific to the site.

(Staff Brief at 12-13, citation omitted.)

The Applicant, in both its brief and witness' testimony, offers that the approach used in determining the SSRS for the Midland site primarily is deterministic but goes on to explain the limited use made of probabilistic techniques in determining the SSRS. App. Brief at 1-2, 4, 12-13; Holt, ff. Tr. 4539, at 14; Holt, Exh. 10, at 5-10 and Figs. 1-8. In its brief, the Staff points out (at 12-13) that the SSRS method recommended in the Tedesco letter is a straightforward empirical approach to design a response spectrum that is specific to a site (and to its SSE, based on earthquake magnitude) and which complies more closely than the alternative standardized-spectra approach with the mandate of Appendix A to account for specific site conditions. It is not a probabilistic methodology as used here; it does employ certain statistical treatment of a sufficiently large population of earthquakes, matched as to their size and similarity of applicable site conditions, which are reviewed for appropriateness on a case-by-case basis. The Staff points out that the design of a site-specific response spectrum is no more than the adjusting or tailoring of a standardized response spectrum for the particular seismic and geologic characteristics of the selected site. *Id.* at 11-14. The Applicant agrees that use of SSRS is no more probabilistic than use of the Regulatory Guide 1.60 spectral shape. App. FOF, ¶ 14.

The Staff, also in response to our Memorandum, provided information in its brief on past applications of the SSRS approach, referencing the licensing of Sequoyah, Units 1 and 2, and San Onofre, Units 2 and 3. The Commission has approved licenses for both of those facilities. At the time of the submission of its brief, the Staff was also in the final stages of approving site-specific spectra, designed using methodology similar to that employed at Midland, for the Enrico Fermi Unit 2, Watts Bar, and Bellefonte plants. Safety Evaluation Report, Sequoyah Nuclear Power Plant, Units 1 and 2, Tennessee Valley Authority, Docket Nos. 50-327 and 50-328, March 1979, NUREG-0011, § 2.5.3; Safety Evaluation Report (Geology and Seismology), San Onofre Nuclear Generating Station, Units 2 and 3, Southern California Edison Co., *et al.*, December 1980, NUREG-0712, § 2.5.2; Safety Evaluation Report, Enrico Fermi, Unit 2, Detroit Edison Co., NUREG-0798, July 1981, § 2.5.2; Staff Brief at 15-17.

The Staff's application of the SSRS methodology at Sequoyah resulted from a situation quite similar to that at Midland; i.e., during its OL review the Staff had questioned both the spectrum and the ground acceleration value originally chosen at the CP stage. In all material respects the procedure used at Sequoyah was identical to that employed for designing the Midland SSRS, and the procedure was reviewed in depth and endorsed by the Advisory Committee on Reactor Safeguards (ACRS). Staff Brief at 15 and Attachment I (Letter from ACRS Chairman M. Carbon to NRC Chairman J. Ahearne, "Interim Low Power Operation of Sequoyah Nuclear Power Plant, Unit 1," dated December 11, 1979).

The Staff disagreed with the Applicant's Proposed Finding 10 (that seismicity is a "probabilistic consideration") and with the Applicant's Proposed Finding 14 (that the statistical process of combining earthquake records in the construction of response spectra is probabilistic). Both of these views of the Applicant on the "probabilistic aspects" of establishing the SSE and constructing the SSRS also occur in the Applicant's Brief (at 6-7, 12), in the testimony of the Applicant's witness (Holt, ff. Tr. 4539, at 17), and are viewed by the Board as unnecessary, and incorrect, arguments to justify use of the SSRS methodology.

In sum, we view the SSRS methodology as employed at Midland as no more than a specific site application of the technology used to develop the standardized spectra contained in Reg. Guide 1.60. Only historical records made in substantially similar soil conditions are chosen for designing the SSRS. It takes into account the expected maximum vibratory acceleration at the site throughout the frequency range of interest, as required by §§ V(a)(1)(iv) and VI(a)(1) of Appendix A. The design of the spectrum is based on an objective analysis of empirical historical records of earthquake ground motion, analytically related to the SSE, as required by Appendix A, §§ IV(a) and V(a)(1). Finally, the SSRS takes account of seismology, geology and underlying soil characteristics of the site, as required by § V(a) of Appendix A. Accordingly, we agree with the Applicant and Staff that the SSRS methodology, as employed at Midland, satisfies the governing requirements of 10 C.F.R. Part 100, Appendix A.[11]

[11] We are informed that the NRC Staff has developed SSRS using a different methodology than that described above for use in its Systematic Evaluation Program or "SEP" (which includes the La Crosse Boiling Water Reactor). The SEP SSRS are based on a complex synthesis of deterministic judgments and probabilistic modeling, which do not, at least explicitly, follow the deterministic procedures outlined in Appendix A. This SEP methodology is not involved in this case, and we express no opinion as to its validity. *See* App. Brief at 6 n.3; Staff Brief at 14; *see also Dairyland Power Cooperative* (La Crosse Boiling Water Reactor), LBP-83-23, 17 NRC 655, *aff'd (sua sponte)*, ALAB-733, 18 NRC 9 (1983).

B. Maximum Earthquake and Associated Ground Motion at the Midland Site (Findings 19-79)

The Design Basis Earthquake (DBE) approved for the Midland site at the CP stage was based on a Modified Mercalli Intensity (MMI) of VI, the size of the largest earthquake within about 150 miles of the plant site. CP "SER," at 13, 114, 116. The DBE was not associated with any tectonic province, since the CP review was performed before promulgation of either the proposed or final version of 10 C.F.R. Part 100, Appendix A, which required such determinations. (*But see supra* note 6.) The ground motions associated with the DBE were represented by a modified Housner design response spectrum anchored at 0.12*g* (where *g* = acceleration due to gravity at the earth's surface). The Housner spectrum was modified by increasing its levels of response motions by an additional 50% in the frequency range between about 1.6 Hz and 5 Hz (or 0.6- and 0.2-seconds-period range). CP "SER," at 13; Finding 21, *infra*.

Because the seismic design basis for the Midland Plant followed procedures and regulations in existence before promulgation of Appendix A, the Staff, during its review of the OL application, questioned whether the plant safety systems were designed to withstand the effects of an earthquake as would be determined by current standards. It raised questions as to the adequacy of both the ground acceleration value (0.12*g*) and the design response spectra (modified Housner) used to represent the earthquake motions.

The Board has found remarkably little disagreement, in the end, between the technical positions of the Applicant and the Staff; but the route to this conclusion has not always appeared so clear. The final result, with which we agree, was a commitment by the Applicant to use site-specific response spectra (SSRS) to represent Safe Shutdown Earthquake motions that differ from the original modified Housner design spectra mainly in shape. *See* Figures 2 and 3, *infra* pp. 66-67. While site-specific response spectra, by their method of construction, are not "anchored" at a peak acceleration value, those derived by the Applicant are very close at most frequencies to what would be obtained by current standardized (Regulatory Guide 1.60) response spectra anchored at 0.12g, the original (DBE) peak acceleration value determined for the Midland site. These site-specific response spectra were to be used by the Applicant in the seismic reevaluation of structures, systems, and components important to safety[12] and as minimum input values in the seismic design[13] of certain remedial structures (underpinnings and new foundations) required to be built as a result of improper compaction of soil fill on which some of the safety-related[14] buildings were partly or completely founded. Thus, the earthquake represented by these site-specific response spectra and determined by this Board to meet the requirements of Appendix A (*see* discussion, *infra* pp. 63-69), is properly termed the Safe Shutdown Earthquake (SSE). The original DBE was the seismic design basis for the bulk of the structures, systems, and components important to safety at the Midland Plant, at the time they were initially designed.

In its 1977 FSAR, the Applicant proposed an SSE that was based upon a newly proposed Michigan Basin tectonic province. That SSE, which was never accepted by the Staff, came to be called the "FSAR SSE" in these proceedings. Its size and ground motion characteristics are identical to those of the original DBE, and are at issue in these proceedings. The terms "FSAR SSE" and "FSAR spectra" as used in these proceedings should be read as "DBE" and "DBE spectra," respectively. Because there can be only one SSE for the Midland site, and if the project were to be continued or resurrected, a future revision of the FSAR would need to reflect the SSE and its ground motion characteristics, as determined by the outcome of these proceedings.[15]

While the December 6, 1979 Modification Order did not specifically address seismic issues, one of its major concerns was "the unresolved safety issue concerning the adequacy of the remedial action to correct the deficiencies in the soil construction under and around safety-related structures and systems . . ." (Modification Order at 4). Seismic design bases (the SSE and representation of its motions) for the underpinning

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[12] This Board does not distinguish a difference between the terms "important to safety" and "safety-related" when applied to seismic design requirements. It seems clear to us that 10 C.F.R. Part 100, Appendix A, uses the terms quite interchangeably. Staff practice in this regard is reflected in Regulatory Guide 1.29 which designates as "Seismic Category I" those structures, systems and components which shall be designed to remain functional if the safe shutdown earthquake (SSE) occurs. The Regulatory Guide includes, *inter alia*, as Seismic Category I "[t]hose portions of structures, systems, or components whose continued function is not required but whose failure could reduce the functioning of any plant features [whose function is required] to an unacceptable safety level . . ." (at C.2). *See also* note 94, *infra* p. 195.

[13] Those remedial structures already designed were designed to 1.5 times the original DBE response spectrum which was found to be higher than the SSRS for this particular purpose. Tr. 6003 (Kennedy).

[14] *See* note 12, *supra*.

[15] This Board is ignoring another term introduced by the Applicant (App. FOF, ¶ 5), the "Seismic Margin Earthquake" or SME, said to represent the earthquake corresponding to the site-specific response spectrum ground motions. It is synonymous with the SSE as used here.

work clearly are included under the required acceptance criteria necessary for the Staff to evaluate the technical adequacy and proper implementation of the proposed remedial actions (*id.* at 3).

The operating basis earthquake (OBE) proposed in the FSAR, represented by modified Housner response spectra anchored at 0.06*g* (also as accepted at the CP stage), has not been at issue in these proceedings. We accordingly are making no findings with respect to the adequacy of the OBE. We note, however, that it has been accepted as sufficiently conservative by the Staff in light of the definition, in part, of the OBE as the earthquake expected at the plant site during the operating life of the plant. SER, § 2.5.2.5, at p. 2-39; 10 C.F.R. Part 100, Appendix A, § III(d).

(1) Tectonic Province

In its 1980 "Tedesco letter," the Staff had offered the Applicant two alternative approaches to resolve the Staff's concerns about the adequacy of the DBE and its corresponding response spectra. The first would have been to use the standardized response spectra of Regulatory Guide 1.60, a design practice regarded by the Staff as acceptable since December 1973 (the date of issuance of the current version of the Guide). The other would be to develop SSRS based on actual site-and-magnitude-matched accelerograms recorded at distances within 25 km of an earthquake, an approach made possible by the increased number of close-in earthquake recordings that have become available since derivation of the earlier standardized response spectra. The Staff further specified that either of these approaches should be based upon an SSE similar to the Anna, Ohio earthquake, with a magnitude of 5.3 or intensity of MMI = VII-VIII which the Staff had come to recognize as the controlling earthquake in the Central Stable Region tectonic province that included the Midland site.

The Applicant elected to use, and submitted reports on, the SSRS approach but maintained (1) that the low seismic hazard at the Midland plant site did not warrant use of an SSE as large as the Anna, Ohio earthquake; and (2) that the Michigan Basin, with a magnitude 4.5 controlling earthquake, satisfied the requirements of Appendix A to Part 100. The Applicant also maintained, in our view incorrectly (*see infra* Finding 58), that the assigned magnitude of the Anna, Ohio earthquake should be 5.0, not 5.3. Additionally, results of comparative probabilistic seismic hazard studies performed for five sites, as specified by the Staff, in other parts of the Central Stable Region were submitted in 1981 to show the relatively lower seismic hazard at the Midland site.

Based almost entirely on its evaluation of these seismic-hazard study results, the Staff changed its position, agreeing that the Midland site lies in a region of lower seismicity that could be subdivided from the Central Stable Region, but whose boundaries extend westward from the Michigan Basin to include the upper peninsula of Michigan, northern Wisconsin and all of Minnesota, and perhaps other areas, as well. This larger area included a magnitude 5.0 historic earthquake that occurred in Minnesota in 1860 and which would be the controlling earthquake for the proposed tectonic (or seismotectonic)[16] province.

The Staff's changed position on the smaller SSE and appropriate tectonic province came late in the proceeding, after the Applicant's expert witness, Mr. Richard J. Holt, had written his prepared testimony, and only shortly before the Staff's expert witness, Mr. Jeffrey K. Kimball, prepared his own testimony. A result of this late development was that the Staff had insufficient time to develop fully its justification for the definition of its proposed tectonic province or indeed its extent. Another effect was that much of the Applicant's testimony that was directed against the now-abandoned magnitude 5.3 SSE became moot or appeared immoderately overstated in light of the Applicant's general endorsement of the new Staff position. As a result, we heard some testimony on "nonissues" and some to correct inconsistencies which were a source of confusion at the time and in the record as it stands. While not specifically abandoning the Michigan Basin as a proposed tectonic province to include Midland, Mr. Holt agreed that the choice of a magnitude 5.0 SSE would be appropriate and would correspond to the largest historical earthquake which should be associated with the tectonic province in which the Midland site resides.

On the basis of the record, five choices became available to the Board for determining the appropriate tectonic province for the Midland site and the size of the controlling earthquake to be designated therein. Because the evidence indicated (a) that there are no capable faults or other tectonic structures with which earthquakes may reasonably be correlated within 200 miles of the site, and (b) that earthquakes in adjacent tectonic provinces would not govern maximum ground motions at the site, the controlling earthquake within the tectonic province in which the site is located would become the SSE, subject in this case to additional limited effects from a postulated recurrence of the more distant (about 500 miles), but very large, New Madrid earthquake. The five possible choices are:

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[16] The Staff consistently used the term "seismotectonic province" but explained that it equated that term with [illegible]ic province as used in Appendix A. Tr. 4698-99, 4757-58 (Kimball).

(1) Undivided Central Stable Region, with a magnitude 5.3 or intensity VII-VIII controlling earthquake.
(2) The Staff's ill-defined proposed tectonic province, with a magnitude 5.0 or intensity VII controlling earthquake.
(3) The Applicant's proposed Michigan Basin tectonic province, with the originally proposed magnitude 4.5 or intensity VI controlling earthquake.
(4) The Applicant's proposed Michigan Basin tectonic province, with the agreed-upon magnitude 5.0 or intensity VII earthquake.
(5) Indefinite tectonic province (i.e., no resolution of the different tectonic provinces proposed by the Applicant and by the Staff), with the agreed-upon magnitude 5.0 or intensity VII controlling earthquake, but limited to this proceeding only.

By reducing two of the Applicant's map portrayals to a common scale and overlaying them, the Board has provided a single map here (Figure 1, *infra* p. 56) for convenience to show the proposed tectonic province boundaries, major tectonic structures, seismic source zones, and Central Stable Region sites used in the relative seismic hazard studies. To this map the Board has added the delineation of what we understand from the verbal descriptions to be the boundaries of the Staff's proposed westward extension and an area in southeastern Michigan that we would exclude based on the Staff's reservations about its inclusion, as well as a few place names from the testimony.

In regard to determination of the appropriate tectonic province, the Board notes first of all that the total range of sizes of controlling earthquakes that we are to consider here is not very great — magnitude 4.5 to 5.3 and intensity VI to VII-VIII. Because of the testimony we heard that accuracy of assignment of magnitude to an individual earthquake is, at best, about 0.2 magnitude units (we heard estimates for the Parkfield earthquake ranging from 5.5 to 6.2), and because intensity is even more subjectively assigned than magnitude, we believe that determination of a controlling earthquake, or SSE, to within about one-fourth magnitude unit or one-half intensity unit is about as fine a discrimination as can be made. The choices between magnitude 5.0 and 5.3 or between intensity VII and VII-VIII involve what we believe to be the minimum practical limit for distinguishing controlling earthquakes in different tectonic provinces. In this same regard, the seismic hazard calculations which we heard that carried both magnitude and intensity differences out to two decimal places strained our credulity. They imply a degree of accuracy which is not now attainable.

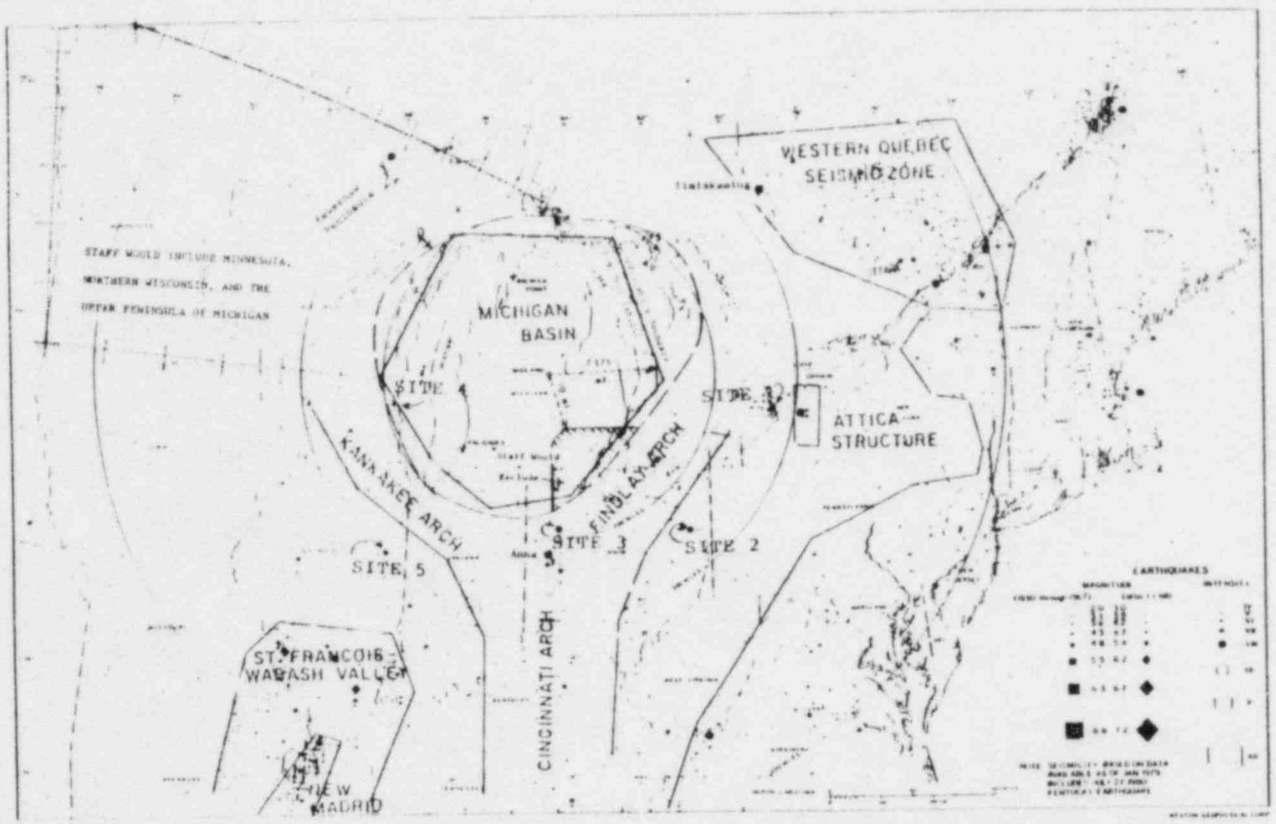

FIGURE 1. Map showing boundaries of Tectonic Provinces proposed by Applicant and Staff. Sites 1-5 are those utilized in comparative seismic hazard analyses. (Compiled from Holt Exhs ^ and 10, Fig. 5; and from verbal descriptions by Staff witness Kir . d.)

Both the Applicant and Staff presented sound testimony to the effect that the Central Stable Region can be subdivided, and that the Midland site lies in a region having a lower seismic hazard than other parts of the Central Stable Region. The evidence indicated that the controlling earthquake for the region surrounding the Midland site can be smaller than the magnitude 5.3 Anna, Ohio earthquake.

The maximum historical earthquake that has been recorded in the Applicant's proposed Michigan Basin province is 4.5. However, the time interval of record (since about 1850) is short when compared to the estimated statistical recurrence interval that Staff practice deems acceptable for an SSE, 1,000 to 10,000 years. Also, the total number of historic earthquakes is small, between about nine and seventeen, which may be an insufficient sample, statistically, to overcome the uncertainty that the maximum historical earthquake is a sufficient basis for the SSE. Furthermore, acceptance by the Applicant of a magnitude 5.0 controlling earthquake for the tectonic province in which the Midland site resides (derived by the Staff from the Applicant's own seismic hazard studies) indicates abandonment of the originally proposed magnitude 4.5 controlling earthquake. Findings 42, 52-54, 56.

As set forth in our findings, we find that the Staff failed to provide adequate tectonic and geologic bases to support its proposed tectonic province, or even to define its boundaries. On this latter point, the Staff witness (Mr. Jeffrey Kimball) testified that given the opportunity and ample studies he would be able to define the boundaries concisely, but that he had not done so. It was clear that he perceived a uniformity of low seismic hazard across the entire region, which included all of the Michigan Basin, except for the southeastern corner, as well as the proposed westward extension. This perception was borne out by the seismicity, there having been about fourteen historic earthquakes in the proposed westward extension, which extension alone had about twice the area of the Michigan Basin. However, the Board finds the Staff's theory linking seismicity and, *ipso facto*, undefined tectonic structure too weak upon which appropriately to base definition of a tectonic province. We also find that the Staff should have addressed differences in orientation of tectonic structures in the westward extension, that we noted on Staff Exhibit 5, and those cited by the Applicant as indicating relative uniformity of tectonic structure in the Michigan Basin. We believe the Staff also should have addressed the possible tectonic significance of small earthquakes with anomalously high intensities (presumably resulting from shallow depth of occurrence) that have occurred in the Keweenaw Peninsula of Michigan, an area where the tectonic structures are apparently orthogonal to those in the Michigan Basin. Findings 43, 45-50, 55, 57.

For purposes of this Decision, and taking into account the degree of agreement between the Applicant and Staff on the appropriate SSE and the representation of its ground motions by the SSRS, this Board was urged to avoid choosing between the Staff's or Applicant's proposed tectonic provinces, because either province would have a controlling earthquake of magnitude 5.0. App. FOF, ¶ 30; not contested by the Staff (Staff FOF, ¶ 30). *See* option (5), set forth *supra* p. 55. However, we reject this option to leave the tectonic province indeterminate for four main reasons. First, we read Appendix A as requiring such a determination for each license application — particularly where, as here, the ascertainment of the tectonic province is an issue in a proceeding (*see infra* Findings 35-36, 38, 42-43, 49-51, 52, 54-55). Second, since either of the proposed tectonic provinces would be subdivided from the larger Central Stable Region, the boundaries between the new and the "parent" province must be sustainable under the provisions of Appendix A to Part 100; otherwise the already-established controlling earthquake of the Central Stable Region should apply. We have already commented on why we found the boundaries of the Staff's proposed tectonic province not to be sustainable, and in fact they were not drawn.

Third, we heard, and agree, that the Central Stable Region can be subdivided because of its inherent nonuniformity of seismic hazard. To reach a decision here that would be applicable only to the Midland site will not further the longer-term objective of accomplishing that subdivision. Regulatory stability would not be enhanced.

Finally, we have found the Applicant's proposed tectonic province, and its boundaries as modified here, sufficient to meet the requirements for definition under the provisions of Appendix A to Part 100. Thus there is no reason to consider an indeterminate tectonic province as a basis for our decision.

The Applicant maintains that the Michigan Basin meets the requirements in Appendix A for definition as a tectonic province. We agree. It is a very large tectonic structure itself (nearly 400 miles across), a structural depression of the earth's crust containing ancient sedimentary rocks of Paleozoic age about 3.5 km thick near the center of the basin, but thinner near its margins. It is distinguishable from the tectonic arches around its southern perimeter on the bases of structural relief, parallel and cross structures on the arches, and seismicity differences. It has a relative consistency of tectonic features within it, namely the northwest-southeast trending anticlines, monoclines, and possible related faults, known mainly in the deep subsurface from petroleum exploration in the State. The largest historic earthquakes that have occurred in the basin were two events in the southern part of the basin, both of which had an intensity MMI = VI, or an equivalent magnitude m_{blg} = 4.5.

Two maps introduced by the Applicant show somewhat different boundaries for the Michigan Basin tectonic province, but the differences between them appear to fall within the degree of acceptable uncertainty ascribed to them in the testimony. The Board would accept either of the sets of boundaries provided by the Applicant (but prefers the smaller), except that we would exclude the southeastern corner of the State of Michigan about which the Staff expressed reservation. *See supra* Figure 1. We base our exclusion on the assumption that the structures shown as occurring near Detroit and Ann Arbor on Staff Exhibit 5 were thought by the Staff witness to be representative of those on the Findlay Arch, rather than of those in the Michigan Basin, and possibly related to similarly aligned structures that exist in the vicinity of Anna, Ohio, located just to the south. Findings 37, 38, 40, 53.

The Staff's objections to subdividing just the Michigan Basin from the Central Stable Region, as the Applicant had proposed, were partly based on the same problem as perceived with retaining the Central Stable Region as a tectonic province, i.e., both would be based on features present in the "surficial Paleozoic geology" which both the Staff and Applicant asserted bore little or no relationship to the underlying tectonic features causative of earthquakes. However, the Staff as well as the Applicant relied on those very features, the arches along the southern margin of the Michigan Basin, in proposing the position of portions of the boundary of their respective tectonic provinces. The Staff's witness stated that, in the past, the Staff has relied upon the Central Stable Region as a tectonic province (Tr. 4786 (Kimball)); hence it must be regarded as meeting the requirements of Appendix A to Part 100, at least in the Staff's view. He also stated that there are some experts who would consider that portion of the Kankakee Arch that has experienced essentially no earthquakes in historic times to have a potential for seismic activity (Tr. 4760 (Kimball)). The Board sees no reason to accept the argument against using features in the "surficial Paleozoic geology" to reject either the Michigan Basin or the remaining parts of the Central Stable Region as valid tectonic provinces. While Appendix A may implicitly require some correlation of tectonic features with levels of earthquake activity in defining a tectonic province, it does not require a full understanding of the causal relationships.

The Staff's witness also proffered that it would be inconsistent to establish one structural basin in the Central Stable Region as an area of relatively low seismic activity when another, the Illinois Basin, exhibits a much higher level of seismic activity (Tr. 4837 (Kimball)). Again, we can assign little probative value to this argument against basing a tectonic province on the Michigan Basin since we do not know the causes of earthquakes in either basin and do not assume that the causative tectonic mechanisms of earthquakes should be the same in all basins. Also we note that the Illinois Basin (*see* Staff Exhibit 5) is adjacent to the very active New Madrid seismic zone where tectonic stresses are obviously high.

(2) Controlling Earthquake (SSE)

While the Board finds that the total number of historic earthquakes that have occurred in the Michigan Basin tectonic province (between nine and seventeen by our count) does indicate a low seismic hazard, we also find that this very paucity of data casts doubt on the appropriateness, or conservatism, of relying on the size of the largest historic earthquakes (two events of intensity VI with a corresponding magnitude of 4.5) to represent the controlling earthquake in the tectonic province. We believe this perceived inadequacy of seismological data warrants requiring that the controlling earthquake, hence the SSE, be larger than the maximum earthquake that has occurred historically within the tectonic province.

We base this conclusion on the fact that inadequacy of the seismological data is essentially the same condition as that described by the original version of ¶ V(a)(1)(iv) of Appendix A to Part 100 as the reason for requiring that the procedures used in determination of the SSE be applied in a conservative manner. Prior to clarification by the Commission's amendment in 1977, sentence four of ¶ V(a)(1)(iv) of Appendix A of the Siting Criteria read:

> *In order to compensate for the limited data,* the procedures in paragraphs (a)(1)(i) through (a)(1)(iii) of this section shall be applied in a conservative manner.

10 C.F.R. Part 100, Appendix A, ¶ V(a)(1)(iv), final rule published at 38 Fed. Reg. 31,279 (Nov. 13, 1973) (emphasis supplied to words replaced in the 1977 clarifying amendment).

This requirement appeared in both the proposed rule issued in 1971 (36 Fed. Reg. 22,601 (Nov. 25, 1971)) and the final rule promulgated in 1973. Paragraph V(a)(1)(i) of Appendix A specifically states that "[t]he magnitude or intensity of earthquakes based on geologic evidence [that are used in the determination of the SSE] may be larger than that of the maximum earthquakes historically recorded," albeit in connection with earthquakes associated with tectonic structures (which would include capable faults). The clarifying amendment issued in 1977 (42 Fed. Reg. 2051 (Jan. 10, 1977)) made it quite clear that this conservatism is to be applied to earthquakes associated with tectonic provinces as well, in the event that geological and seismological data warrant. This was accomplished by replacing the introductory phrase with specific subsequent wording, *viz:*

> The procedures in paragraphs (a)(1)(i) through (a)(1)(iii) of this section shall be applied in a conservative manner. The determinations carried out in accordance with paragraphs (a)(1)(ii) and (a)(1)(iii) shall assure that the safe shutdown earthquake intensity is, as a minimum, equal to the maximum historic earthquake intensity experienced within the tectonic province in which the site is located. In the event that geological and seismological data warrant, the Safe Shutdown Earthquake shall be larger than that derived by use of the procedures set forth in Sections IV and V of the Appendix.

In its Statement of Considerations accompanying the 1977 clarifying amendment, the Commission emphasized that the provisions of Appendix A are minimum requirements and that they have consistently been interpreted as such in licensing decisions. It further stated that the amendment related solely to minor matters of a clarifying nature. By this we interpret the Commission's intent as not to change the underlying basis of the requirement, as reflected in the replaced words. We also note that in at least the second and third examples given by the Commission to illustrate conditions where a larger-than-historic earthquake in a tectonic province might be warranted, limited geological or seismological data might be considered to be an underlying cause for the warrant.

We find that the magnitude m_{blg} = 5.0 SSE proposed by the Staff and agreed to by the Applicant is appropriate for Midland. We do not, however, base this finding upon the historical earthquake that occurred in Minnesota within the Staff's proposed westward extension of the tectonic province containing the Midland site, but upon the results of the Applicant's probabilistic seismic hazard studies which compared five sites in the Central Stable Region with the Midland site, and upon the Staff's analyses of those studies. While we could not find that it was permissible to define a tectonic province on the basis of comparative seismicity studies alone, as the Staff seemingly had proposed, we do accept the Staff's evaluation of the Applicant's seismic studies, and the results of the studies themselves, as appropriate methods for use in determining the size of the tectonic province's controlling earthquake and, hence, the SSE.

We agree with the prudence of the Staff's precautions about using probabilistic results only in a comparative manner and at several sites, rather than relying on any calculated "absolute" probability at any specific site (*cf.* Kimball, ff. Tr. 4690, at 16). We would further repeat that we regard as significant only those differences that exceed about one-half of an intensity unit or about one-quarter of a magnitude unit. Also, we could not have accepted the results had they indicated a smaller SSE than the maximum historic earthquake in the tectonic province, since such acceptance would be contrary to the mandate of Appendix A to Part 100.

The probabilistic seismic hazard study methodology compared the estimated earthquake intensities that would be assigned to the Midland site and five other sites in the Central Stable Region at different probability levels dependent upon the size and number of earthquakes that have occurred in the regions surrounding each site, assuming different zonation models, or boundaries for earthquake zones, each earthquake zone having an *assumed* upper-bound cutoff for its respective controlling earthquake. The Applicant's witness (Holt Exh. 10, at 4) explained the principle of seismic hazard simply as "the closer a site is to an earthquake zone, the higher the hazard." The probabilistic methodology inexactly quantifies that principle.

The results of the Staff's analyses showed that at a 10^{-4} annual probability-of-exceedance the calculated intensity level for all study sites is essentially the same (about "7.5" or VII-VIII)[17] when the undivided Central Stable Region zonation model is used. This result is to be expected since each site was assumed to experience the controlling earthquake for that source zone. At the same probability level, the other zonation models, including the Michigan Basin-and-arches Model, show the Midland site to have a calculated intensity level of about VII (expressed as "6.9"), well below the average intensity calculated for the other sites, which ranges from "6.9" to a high of "8.75." The highest intensity, using these zonation models at the 10^{-4} probability level, was predicted at Site 3, located near Anna, Ohio. Kimball, ff. Tr. 4690, Table 1.

In the Board's view the Applicant and Staff over-elaborated the numerical calculations and comparisons, and implied greater accuracy of the results than attained. We believe that the most considered conclusion to be drawn from the relative seismic hazard studies is that the intensity at the Midland site, calculated at a probability-of-exceedance of

[17] The Board has some difficulty in understanding the significance of decimal values applied to the Modified Mercalli Intensity Scale which properly uses Roman numerals for its descriptively based, nonuniform divisions. *See* Holt Exh. 4.

10^{-4} per year, is about one-half intensity unit (or about one-quarter magnitude unit) lower than that at most of the other sites studied in the Central Stable Region. The values obtained in the Staff's analysis were "0.50" to "0.70" intensity units corresponding to 0.25 to 0.35 magnitude units. *Id.* at 20. The sites studied were selected to be representative of areas both where significant earthquakes have occurred and have not occurred within the Central Stable Region (Tr. 4761 (Kimball)).

In determining the SSE ground motions, it was also necessary to consider the effects at the Midland site which might result from occurrence of the controlling earthquakes in adjacent tectonic provinces, assuming that each occurred at a point on the tectonic province boundary closest to the site. The first earthquake to be considered would be similar to the Anna, Ohio event, which occurred in 1937, and is the controlling earthquake within the Central Stable Region. It occurred at a location about 205 miles south of Midland. Even with the Board's exclusion of the southeastern corner of the Michigan Basin, the nearest approach of the tectonic province boundary to the site would be no closer than about 70 miles. *See* Figure 1, *supra* p. 56. The Staff's calculations indicated that a magnitude 5.3 Anna-type event would have to occur much closer than 70 miles, something like 25 miles, from the site before its motions would exceed those of a magnitude 5.0 event occurring at the site.

The Board questioned the Staff's witness about another, larger, earthquake which had occurred in Canada at a location about 340 miles northeast of Midland. This was the magnitude 6.2 Timiskaming event which occurred within the Applicant's "Western Quebec Seismic Zone." *See* Figure 1, *supra*. Because of the indefiniteness of the boundaries of the Staff's proposed tectonic province the Board wanted to be reassured that the Timiskaming earthquake had not been overlooked because of its occurrence outside the United States. While the Staff's witness allowed that the Staff's proposed tectonic province *might* extend northeastward to abut the province containing the Timiskaming earthquake, he estimated that the Canadian earthquake would have to occur within 100 miles of the site before its motions would exceed the ground motion spectrum accepted for the SSE at the site, and in no case would the tectonic province boundary in that direction be closer than 100 miles from the site.

(3) Construction of the SSRS

The Staff evaluated the SSRS that were submitted by the Applicant to meet the Staff's criteria for a magnitude 5.3 SSE. The Staff concluded that as submitted, without the inclusion of any spectra from the magnitude 5.65 Parkfield earthquake, the SSRS were appropriately conservative to be used to represent a magnitude 5.0 SSE at the Midland site. The Staff's already-stated criteria were that the SSRS would be derived from enveloping, at the 84th percentile statistical level, response spectra calculated from an ensemble of actual site-and-magnitude-matched earthquake records taken from within 25 km of the recorded earthquakes. Site matching was to be based on similarity of the soils beneath the recording site, in terms of thickness, layering and shear moduli, to soils beneath the Midland site. Different spectra were to be constructed to correspond to the top of the natural soils (glacial till and lacustrine clays) and to the top of the approximately 30-foot-thick softer soil fill, on each of which some of the safety-related structures were founded. The effect of the softer fill layer would be to further amplify seismic ground motions at certain frequencies, mainly those in the range of 1-4 Hz. Magnitude matching was specified as the SSE magnitude ± 0.5 magnitude units. The magnitude range of the "without-Parkfield" ensemble of earthquakes used in construction of the SSRS submitted by the Applicant was 4.9 to 5.5, thus falling within the Staff's magnitude-matching criterion for a magnitude 5.0 SSE. Recording-distance and foundation-materials-properties criteria were also deemed by the Staff to be satisfactorily matched. We agree.

The Applicant used forty-four component records taken at twenty-two instruments during ten earthquakes to construct the top-of-natural-soils ("original ground surface") SSRS. Records from thirty-six components taken from eighteen sets of records at ten sites during twelve earthquakes were used to construct the top-of-fill SSRS. While all the earthquakes from which records were used occurred either in California or Italy, they were selected to include all those available worldwide taken from within the 25-km range, and meeting the specified site-and-magnitude-matching criteria. The 25-km range specified meets the requirement of Appendix A to Part 100 that the SSE within the tectonic province in which the site occurs be assumed to occur at the site; it is also the range within which the Staff considers that no significant source-to-site attenuation differences need be considered, irrespective of whether the earthquakes occurred in Michigan, California or Italy, so long as the materials properties are similar at all the sites.

Given a sufficient number of records from different earthquakes, as used here, the diversity of spectral data in the individual spectra should account for uncertainties of what ground motions might result from the postulated future occurrence of an earthquake the size of the SSE near the site. In this regard, statistical combination of the spectra at the 84th percentile level was judged to be appropriate for design purposes to account for unknown variables, other than magnitude, in earthquake source properties such as stress drop, fault rupture velocity, rock properties along the fault, and style of faulting. Combination at the median level would tend to average out the effects of those unknowns, which conservatism requires to be included. On the other hand, enveloping all the records at the 100th percentile level would overemphasize every anomalous peak that might be present in any record spectrum. Combination at the 84th percentile, while somewhat arbitrary, has been tested through past application of the Regulatory Guide 1.60 standardized spectrum, in which combination of its component spectra was at this statistical level, and which is deemed conservative.

In the low-frequency, or long-period,[18] portion of the spectrum, the SSRS constructed from the records meeting the criteria described above fell off more rapidly than did the original DBE spectrum. *See* Figure 2, *infra* p. 66, which is reproduced here for convenience from Figure 2.7 of the SER, and Figure 3, *infra* p. 67, which combines two of Applicant's representations (Holt Exhs. 1 and 2), and can be used for visual comparison of the two SSRS, the original DBE spectrum and a Regulatory Guide 1.60 spectrum anchored at 0.12*g*. Both the "top-of-natural-soils" and "top-of-fill" SSRS were constrained so as not to fall below the original DBE spectrum at frequencies below about 1 Hz (Holt Exh. 11). This SSRS modification was said to assure protection in design against the effects of very large earthquakes, such as a recurrence of the New Madrid events, at great distances. This is reasonable, considering the greater attenuation with distance of high-frequency seismic motions than of low-frequency motions, but there are few data on which to establish the prope level.

These SSRS, which represent the input seismic design motions of the SSE accepted here, generally exceed the original DBE spectrum. The SSRS and original DBE spectra are closest at frequencies where the original DBE spectrum had been modified by raising the Housner spectrum by 50%. The greatest exceedance of the DBE spectrum occurs at frequencies above 5 Hz; the two SSRS are higher than the DBE spectrum by a factor of about 2 between 5 Hz and 15 Hz, above which frequency they all tend to converge. Thus the DBE spectrum is significantly less conservative (except at the low frequencies discussed above) than either of the two SSRS.

The relationship between the SSRS and the Regulatory Guide 1.60 generalized response spectrum anchored at 0.12*g* (*see* Figure 2, *infra*)

[18] Frequency of vibratory motion, in hertz, abbreviated Hz, or in cycles per second, is the inverse of the period of that motion, in seconds. Thus, high frequencies correspond to short periods, and low frequencies to long periods of motions.

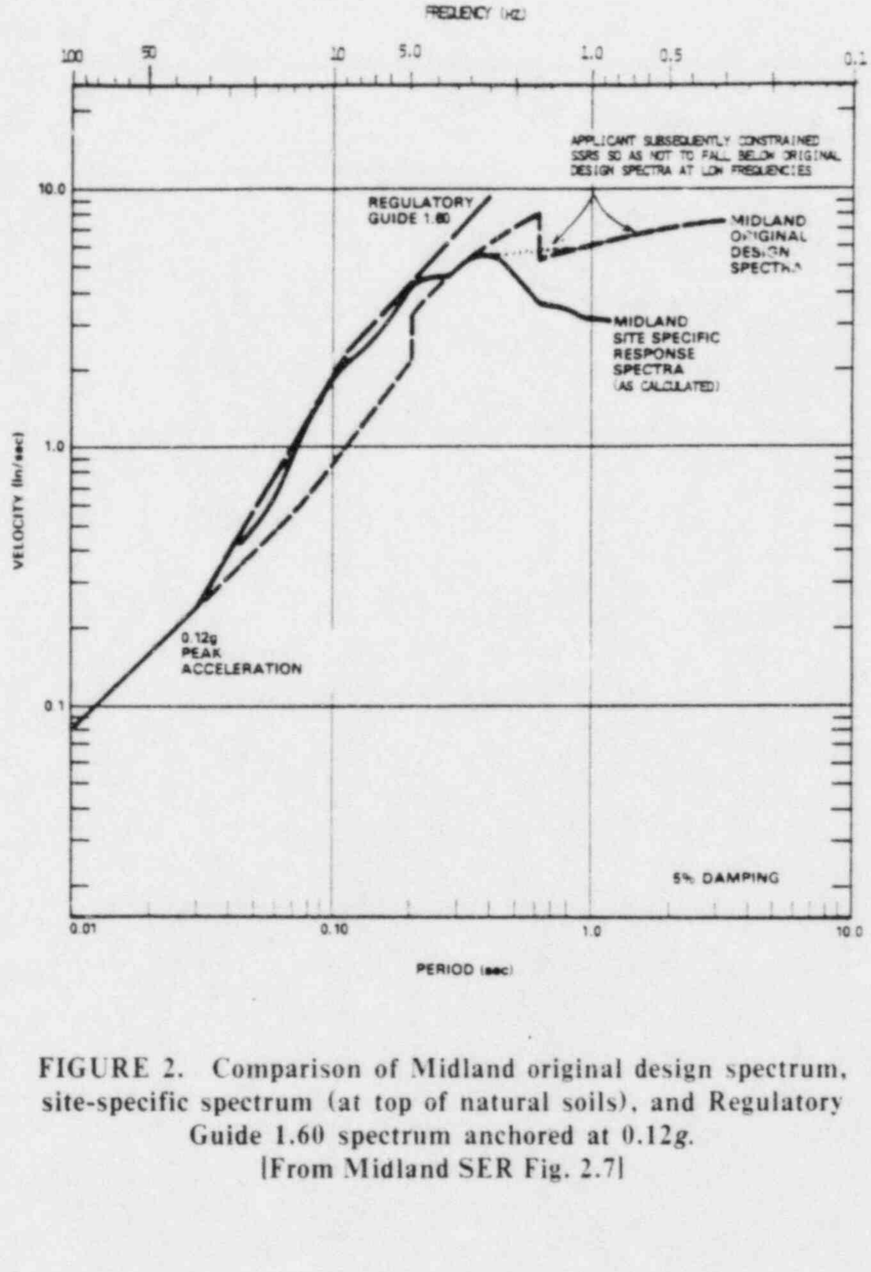

FIGURE 2. Comparison of Midland original design spectrum, site-specific spectrum (at top of natural soils), and Regulatory Guide 1.60 spectrum anchored at 0.12*g*.
[From Midland SER Fig. 2.7]

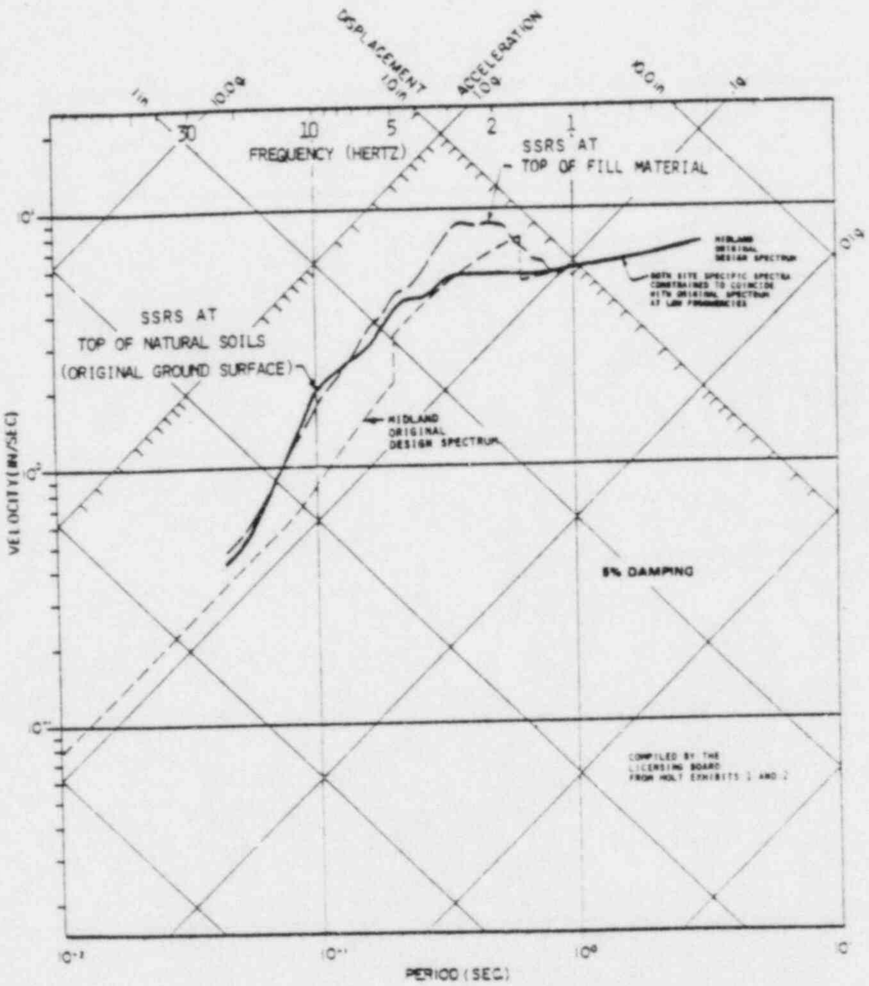

FIGURE 3. Comparison between site-specific response spectra at top-of-natural-soils and top-of-plant-fill and original (Modified Housner) design response spectrum.

is useful only for general comparison purposes. The comparison shows, as might be expected from the testimony, that the SSRS is only slightly lower than the Regulatory Guide spectrum. The Board is not certain that the comparison shown is a completely fair one, because of the differences in maximum or cutoff frequencies used, i.e., 33 Hz for the Regulatory Guide spectrum and 25 Hz for the SSRS. However, we heard no testimony on details of this comparison, and we need not rely on comparisons to the Regulatory Guide 1.60 response spectrum in this Decision.

The Board also notes that Figure 2, *supra*, portrays the significant differences between the now-accepted Regulatory Guide 1.60 spectrum and the older, modified Housner spectrum, used for the original (DBE) seismic design at the Midland site, when both are "anchored" at similar cutoff frequencies. We recognize that these differences in spectra, older (and less conservative) versus more recent, were part of the Staff's early concern in the OL review about adequacy of the seismic design. We agree, however, with the Staff's and Applicant's positions that the SSRS employed here conform to current seismic design practices and are appropriately conservative for the purposes intended.

An alternative approach to determining the SSRS at the top of the plant fill layer would be to multiply the spectral motions of the top-of-natural-soils SSRS by analytically determined amplification factors. The one-dimensional wave propagation computer code (SHAKE) applied by the Applicant utilized the materials properties and layer thicknesses to calculate the amplification of motions at different frequencies to produce an amplification spectrum. To account for the heterogeneous nature and spatial variation of the plant fill, four different soil profiles were used in the calculations. Because the calculated spectra were lower than the spectra calculated directly from the site-and-magnitude-matched earthquake records for the top of the plant fill, the calculations were offered to show the conservatism inherent in the SSRS method. The Staff verified this conservatism using the same computer code but with more realistic (and even more conservative) material properties and earthquakes as input. Thus we find that the top-of-fill SSRS are suitable for seismic reevaluation of those structures founded entirely on plant fill, such as the diesel generator building, the railroad bay of the auxiliary building, and the borated water storage tanks.

At the time when the Applicant undertook design of the underpinning structures for parts of the auxiliary building and service water pump structure foundations, and the new ring-beam-foundation addition to the borated water storage tanks, no agreement existed on the seismic design bases for those structures. In order to proceed, the Applicant incorporated what it believed to be a reasonable margin over the original DBE into the design of those structures. The Applicant directed its contractors to use 1.5 times the DBE (or "FSAR SSE") response spectra as the seismic design basis for those remedial structures. Subsequently, the Applicant committed to use of the SSRS, as accepted here, as a seismic design basis for the remedial structures, but it continued to use the 1.5 times the DBE ("FSAR SSE") spectra in the actual remedial design work (App. FOF, ¶ 70). The Applicant also had dynamic analyses performed which demonstrated that for purposes of design of the remedial structures, the seismic design basis used exceeded the responses derived from the SSRS.

In answers to questions about the adequacy of 1.5 times the DBE as a design basis, the Applicant's witness testified that in parts of at least one structure or substructure not founded on plant fill (the missile shield in the main portion of the auxiliary building) the SSRS responses were 1.7 times the DBE spectral responses, but that the SSRS responses will be used in the seismic reevaluation of the missile shield. That reevaluation, as part of the seismic margin review, would have been considered in the later-scheduled OL portion of this proceeding, but is not material to this Decision.

Accordingly, the Board finds that the Applicant's use of the SSRS for seismic reevaluation of safety-related structures, systems and components of the plant, and its substitute use of 1.5 times the DBE ("FSAR SSE") response spectra in seismic design of the remedial structures is reasonable and conservative.

(4) Seismic Models and Soil Spring Constants (Findings 80-89)

In our May 5, 1981 Prehearing Conference Order, *supra*, we approved an agreement between the Applicant and Staff under which the mathematical models to be used for dynamic analyses of structures as modified by the remedial soil settlement measures, including the bases for the derivation of the spring constants, would be considered in the soils hearings. Consideration of the results of the seismic margin review (i.e., whether various structures conformed to appropriate seismic standards) was postponed until subsequent stages of the OL proceeding, although several witnesses at the soils hearings advanced preliminary views with respect to certain structures.

The Applicant presented testimony on the dynamic seismic models through its consultant, Dr. Robert P. Kennedy of Structural Mechanics Associates, Inc. (SMA). Dr. Kennedy addressed the models being used to perform the seismic evaluation of structures in conjunction with the foundation remedial work — i.e., models for (1) the auxiliary building-control tower-electrical penetration area ("auxiliary building"), an interconnected foundation system; (2) the SWPS; and (3) the BWSTs. The auxiliary building and SWPS models were developed by Bechtel Corporation and reviewed by Dr. Kennedy and SMA. The BWST model was developed by Dr. Kennedy and SMA; it superseded an earlier model developed by Bechtel. The NRC Staff reviewed these dynamic models. The details of these models are set forth in the testimony of Dr. Kennedy (ff. Tr. 5995) as well as in the testimony of the Staff reviewers (Mr. Frank Rinaldi, NRC; Dr. Paul Hadala, of the Corps of Engineers; and Mr. John Matra, of the Naval Surface Weapons Laboratory) (Finding 80).

Dr. Kennedy concluded that the dynamic models for the auxiliary building, SWPS and BWSTs are adequate for establishing the conservative seismic forces to be used in the design of the remedial work and in the seismic margin review. The Staff found the methodology used by the Applicant and its consultant in determining soil spring constants and damping parameters to be sound, and the methodologies used to develop and review other aspects of the dynamic mathematical models to be within the state of the art. The Staff concluded that the auxiliary building and SWPS models adequately represent those structures within the state of the art, and that the dynamic analysis of the BWSTs was satisfactory. The Applicant submitted extensive proposed findings to this effect (App. FOF, ¶¶ 59-76) and the Staff offered no disagreement (Staff FOF, ¶¶ 59-76, at 12). Ms. Stamiris' proposed findings do not cover the seismic models; we treat her claims bearing on other aspects of the analyses of the auxiliary building in our opinion on that structure, *infra* pp. 92-93.

Several months following the presentation of testimony concerning the seismic models, the Applicant conducted a design review which discovered that, in the original seismic design, Category I structures were analyzed using only the nominal soil dynamic modulus value without considering the ± 50% variation of that value as required by the FSAR. This design deficiency, along with others uncovered by the Applicant's design review, was made known to this Board and the parties through Board Notification BN 84-115, "Seismic and Structural Design Departures from Licensing and Design Criteria — Midland Plant," issued June 18, 1984, by the Staff. BN 84-115 was provided to the Board following submission of proposed findings concerning the issues on which we are now ruling. Thereafter, on August 2, 1984, the Staff advised the Board and parties of testimony and evidence which would be affected by the reported deficiencies (including Staff testimony by Messrs. Rinaldi, Matra and Hadala).

While the impact of this design deficiency potentially is applicable to all Seismic Category I structures at the facility, its applicability to the structures considered in this Decision is mainly to the seismic design of the underpinning structures — i.e., the auxiliary building and SWPS — and to the criteria to be established for subsequent seismic margin reviews of plant safety structures — i.e., the soil spring constants. The deficiency does not affect the BWST model developed by Dr. Kennedy, who took into account the ± 50% variation in that model. With respect to the auxiliary building and SWPS models, the testimony presented by the Staff and Applicant gives this Board reasonable assurance that the *nominal* values of the soil spring constants were adequately established. The record further establishes some measure of conservatism in the seismic design by virtue of the exceedance of the SSRS by 1.5 x the DBE (FSAR SSE) response spectra actually used in the design of the underpinning. However, the record is not sufficient to permit a determination of whether the conservatism in calculation of seismic loads provided by use of the 1.5 x DBE (FSAR SSE) response spectra is sufficient to include the range of seismic loads that would result from the required variation of soil spring constants in those calculations. Our conclusions with respect to the seismic models for the auxiliary building and SWPS — but not the BWSTs — are therefore qualified to the extent they may be affected by the design deficiencies.

In BN 84-115 (which preceded the shutdown in construction), the Staff indicated that it would be conducting further analyses of the design deficiencies. Should construction be restarted, these open questions would have to be resolved.

C. Soil Liquefaction and Dewatering (Findings 90-117)

Following the discovery of excessive settlement of the partly built DGB in July of 1978, the Applicant undertook an extensive underground soils investigation program at the Midland site. The general results of the soils investigation revealed that there were, in certain locations, improperly compacted clayey (cohesive) soils, and improperly compacted sands (noncohesive soils) in the plant fill, but that the natural soils (hard clay and sandy clay) beneath the plant fill were competent to provide foundation support for plant structures, providing the foundations were properly designed and constructed without disturbance of the natural soils.

The improperly consolidated clay fill caused settlement through a change in volume as pore water was squeezed out by the weight of overlying soils and buildings ("primary consolidation"). Sand layers in the fill, even where they were low in density and cohesion, presented enough resistance to retard excessive settlement under the static overburden and structural loads. However, certain of the sand bodies were sufficiently loose and low in cohesion that, if saturated by ground water, they would present a potential for soil liquefaction in the event of occurrence of a strong earthquake.

Liquefaction is a phenomenon by which loose, cohesionless, saturated sandy soil loses shearing strength during strong ground shaking, and develops a degree of mobility sufficient to permit large permanent displacements or liquid-like flow behavior. (For a further explanation of soil liquefaction, *see* note 69, *infra* p. 147.) Soil liquefaction below building footings can cause rapid settlement, tilting, or other damage to the structure. Evaluations of the potential for soil liquefaction and differential soil consolidation associated with the SSE ground motions, as well as evaluation of ground-water-induced loads (e.g., uplift of the structure or hydrostatic pressure on underground walls) on safety-related structures are prescribed by NRC regulations. *See* 10 C.F.R. Part 100, Appendix A, §§ IV(a)(1), IV(a)(4), V(d)(1), VI(a)(1), and 10 C.F.R. Part 50, Appendix A, GDC 2.

Potentially liquefiable sands in the plant fill were identified as occurring mostly above elevation 610 feet, but beneath certain safety-related structures and utilities at the Midland facility; these included the DGB, the electrical penetration areas (EPAs) and railroad bay area (RBA) portions of the auxiliary building, the overhanging portion of the SWPS, and a portion of the service water system piping (and duct banks) near the SWPS. Potential soil liquefaction was determined by both the Applicant and the Staff not to be a problem beneath other safety-related structures. However, for reasons set forth *supra* p. 38, and *infra* p. 103, both the Applicant and Staff now regard the evidence on liquefaction under the diesel fuel oil tanks to be inconclusive and the issue to be unresolved.

The Applicant proposed the following corrective measures to reduce or eliminate concerns for soil liquefaction potential: permanent dewatering to maintain the ground water level below elevation 610 feet beneath the DGB and the RBA portion of the auxiliary building; underpinning the present foundations of the EPAs and the overhanging portion of the SWPS so that those structures would be supported entirely by the underlying natural soils;[19] and replacement of poorly compacted fill by competent backfill below the service water piping (and below safety-related electrical duct banks) in the area north of the SWPS.

In order to provide relatively dry working conditions during underground excavation and construction for underpinning the southern portions of the auxiliary building and FIVPs, the Applicant temporarily dewatered that part of the site to an elevation of about 565 feet. Also, a freezewall, or freeze-curtain dam, was emplaced from elevation 610 feet down to the underlying natural clay. The freezewall was put in place by circulating a coolant through pipes in lines of closely spaced boreholes, which froze existing ground water near each hole (or would freeze any ground water seeping into the area of low temperature) to form an impermeable barrier in the soil. *See infra* Findings 135-136. If construction of the underpinnings were to resume, construction dewatering, and presumably the freezewall, would again need to be implemented in the vicinity of the underground work.

Contentions directly challenging the effectiveness of the proposed site dewatering plans are Stamiris Contention 4.D and Warren Contention 2 (one of those which we requested the parties to address following withdrawal of Ms. Warren from the OM proceeding).[20] Stamiris Contention 4.D specifically addresses permanent dewatering concerns. Contention 4.D(1) asserts that the soils remedial actions proposed and performed are inadequate because permanent dewatering would change water table, soil, and seismic characteristics of the site, on which evaluations of the safety and integrity of the plant were based. Contention 4.D(2) asserts that the same inadequacy exists because dewatering may cause an unacceptable degree of further settlement of safety-related structures. Failure or degradation of the permanent dewatering (system) is asserted in Contention 4.D(3) as leading to a situation where there would be inadequate time in which to initiate plant shutdown (before ground water conditions recurred which, in the event of an earthquake, could potentially result in soil liquefaction). These assertions in regard to the evaluation of permanent dewatering of parts of the plant site are considered in this part of our Opinion.

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[19] The applicant also proposed to underpin the foundation of the control tower portion of the auxiliary building and to replace the soil beneath the feedwater isolation valve pits (FIVPs), but as a result of consideration of soil characteristics other than liquefaction potential (*see infra* Findings 126, 144). Also, underpinning of the northern portion of the turbine building, a nonsafety-related building, was to be accomplished as incidental to excavation and access requirements for underpinning the adjacent portions of the auxiliary building and FIVPs, and to ensure that settlement of the turbine building did not adversely impact Seismic Category I structures.

[20] *See infra* note 41. For the full text of these contentions, *see infra* Findings 90 and 98, and Appendix A to this Decision.

Part of Ms. Stamiris' Contention 4.C essentially overlaps her Contention 4.D(1), in that it questions the adequacy of evaluations of dewatering effects, differential soil settlement and seismic effects on specific groups of safety-related structures and systems. The effects of temporary dewatering on the auxiliary building, which was part of the underground construction process, are discussed here. Also, to the extent that soil liquefaction and seismic shakedown are seismic effects, this part of Stamiris Contention 4.C. is treated below.

Warren Contention 2 (in two parts) is very similar to Stamiris Contention 4.D(3). Ms. Warren's contention cites events such as increased seepage from the cooling pond, flooding, failure of pumping systems, and power outages as specific threats to the proposed dewatering procedures. The contention specified liquefaction of site soils and its adverse effects on Class I structures, as potential consequences of inadequate dewatering procedures. Warren Contention 2 is, accordingly, also addressed in this part of our Opinion.

Independent evaluations of loose sands found in the plant fill were conducted by the Applicant and the Staff. The U.S. Army Corps of Engineers, acting as a consultant to the Staff, performed a study of both the liquefaction potential of the soils and the permanent dewatering system that was proposed by the Applicant to reduce or eliminate liquefaction potential in the loose sands beneath the DGB and RBA. Both the Applicant and the Corps of Engineers assumed a magnitude 6.0 earthquake and a peak acceleration of 0.19*g* in their liquefaction analyses. Both the earthquake magnitude (which is used to assign the number of stress-reversal cycles) and the acceleration used are higher than the corresponding magnitude (5.0) and acceleration (0.12*g*-0.13*g*) of the SSE associated with the Midland site. This use of higher values of earthquake magnitude and peak acceleration imparts a measure of conservatism to the empirically derived determinations of liquefaction potential.

In addition to the duration and strength of postulated earthquake motions, three main properties of a sand body determine its susceptibility to liquefaction. First, the sand must be loosely compacted, i.e., relatively low in density. Second, it must be low in cohesion, or cohesionless, i.e., it does not have a high proportion of clay or other binders. Third, it must be saturated; this occurs when the sand is below the water table and the pore spaces between grains are full of water. Other factors, such as confining pressure, ease of escape of pore water and lateral extent of the sand body, may influence susceptibility to liquefaction.

Where feasible, dewatering loose, cohesionless sands will eliminate one of the main conditions that would cause liquefaction. If partial compaction of the dewatered loose sands were to occur during a strong earthquake, any overlying materials and structures might settle ("seismic shakedown"), but without sufficient pore water to take up the overburden load, liquefaction (the concomitant transient loss of shear strength) would not occur.

Separate calculations of the amount of settlement that might result from future seismic shakedown of loose sands beneath safety-related structures were performed by the Applicant. Seismic shakedown is a partial consolidation of low-density sands during earthquake shaking and might occur whether the sand is saturated or not. It is governed generally by the same characteristics of the loose sand that caused concern for liquefaction, except that the removal of pore water, in order to reduce liquefaction potential, removes the buoyant effect of the water on the individual grains, and increases the load on the sand. This increases the potential for seismic shakedown. The amount of predicted settlement from this cause was determined for each layer of loose sand beneath each safety-related structure and summed to determine the total settlement potentially attributable to seismic shakedown at each location. The amounts of predicted seismic shakedown generally were quite small (e.g., 0.25 ± 0.15 inch for the DGB, and about ¼ inch or less for the other affected structures). The Staff evaluated the Applicant's method of calculating seismic shakedown and agreed that the amounts predicted were reasonable and acceptable for use in design.

The Applicant's soils exploration program identified and located potentially liquefiable sands in the plant fill. Identification was accomplished by the standard penetration tests (SPT) made during drilling, in conjunction with analyses of recovered samples. The SPT involves driving a standard sampling tube into soil in a borehole by dropping a hammer of standard weight a specified distance onto the drill stem to which the sampling tube is attached. The number of blows needed to drive the samples 1 foot is counted and recorded, and correlated with the material recovered from the samples. In general, a low "blowcount" from the SPT, in sand soil, would indicate low density and a high liquefaction potential.

Testimony during the hearings indicated that some of the low-blowcount sands, e.g., near the diesel fuel oil tanks, were not encountered in nearby borings and were surrounded above and below by nonliquefiable soils. Subsequently, however, we were advised that the logs of borings near the diesel fuel oil tanks were erroneous (*see supra* p. 38, and *infra* pp. 103-04). In general, small, isolated sand bodies, especially where deeply buried and under a relatively high confining pressure, were not considered by the Applicant's or Staff's experts as presenting significant liquefaction problems. In the case of the diesel fuel oil tanks, the passive resistance of nonliquefiable soil which confines the foundation of the tanks as well as the sand pocket, would have been sufficient to prevent tank failure, even if the sand pocket were assumed to liquefy. Although we agree with the general conclusions of the Applicant and Staff on this point, and further that the small amount of seismic shakedown which had been predicted for the diesel fuel oil tanks (0.1 inch) presented no significant hazard to their safety, as a result of the erroneous boring logs we are making no findings concerning liquefaction or soils stability under the diesel fuel oil tanks.

Potentially liquefiable sands beneath the service water piping and electrical duct banks in the area just north of the SWPS presented a special problem. Because most of the recharge of ground water in the plant fill would come from the cooling pond through natural sands occurring in this area and hydraulically connected to the sands in the fill, failure of the dewatering system would cause the water table near the SWPS to rise rapidly. The rapid rise of ground water and resultant saturation of the loose sands in the plant fill near the SWPS might not allow sufficient time for plant shutdown. While this would not cause liquefaction to occur, it would have caused the potential for soil liquefaction to exist beneath the safety-related utilities in this locality during plant operation. Accordingly, the Applicant committed to removal of the loose sands above 610-foot elevation and beneath the safety-related utilities in this area and replacement with nonliquefiable materials. This remedy would eliminate concern for both liquefaction and seismic shakedown potential.

Elsewhere at the plant site, the bodies of loose sand in the plant fill occurred mainly above elevation 610 feet. The few pockets that lie below that elevation are of such limited extent and under such high confining pressure that they would not present a significant liquefaction problem, even if saturated. The Applicant and Staff, based on their independent evaluations and reviews, both agreed that lowering the ground water table and maintaining it at a level below 610 feet beneath the RBA and DGB would ensure that there would be no potential for liquefaction of soils to affect the integrity of either structure. However, where these bodies occurred beneath safety structures, effects of seismic shakedown were evaluated.

Removal of the buoyancy effect by dewatering and the increase in the load on plant fill layers at depth would have the beneficial effect of increasing the bearing capacity of those dewatered layers. Dewatering of the plant fill would also reduce uplift and hydrostatic pressure loads on embedded structures. In these respects, as in its reduction or elimination of soil liquefaction potential, dewatering would produce effects advantageous to the safety of plant structures. For these reasons, we disagree with a portion of Ms. Stamiris' proposed Findings of Fact (¶ "13," at 5), where she asserts that there has been a "discovery that the bearing capacity of the base soils for the underpinning is ½ that used in the original analysis (BN 83-174)." Ms. Stamiris has apparently confused the term "bearing capacity" with "elastic modulus," another soil parameter. For an explanation of the Applicant's change in elastic modulus value, *see infra* Finding 140.

The effect of dewatering on the clay soils was to increase the amount of compression and the rate of consolidation of the clays, particularly those in the plant fill that were not properly consolidated during their placement. Part of the compression from the dewatering load was recoverable as shown by small amounts of rebound measured when the ground water level was allowed to rise during a recharge test. The part not recoverable on removal of the load is termed consolidation. The effect on the clay soils was expected and predictable on the basis of the settlement observations made. For each of the safety-related structures and underground utilities at the Midland site, the Applicant assessed the additional settlements that would be caused by dewatering, and the Staff was satisfied that they are adequately included in the predicted settlements that were to be used in the structural analyses. While we repeat that we are reaching no conclusions concerning the acceptability of the DGB or its foundation soils, nor on the prediction of differential settlement between the main portion of the auxiliary building and the control tower, no unresolved controversy over dewatering effects at those (or any other) structures exists between the Applicant and the Staff. Intervenor Stamiris did not submit proposed findings on the technical adequacy of the dewatering system, nor upon the effects of dewatering on soils, except for the conclusory denial that the Applicant has adequately and conservatively taken them into account (*see* Stamiris FOF, ¶ "12," at 4-5).

As pointed out above, the threat of possible failure or degradation of the permanent dewatering system was alleged by Stamiris Contention 4.D(3) as resulting in insufficient time for plant shutdown before the ground water level rose to a level causing saturation of the potentially liquefiable sands in the plant fill. Postulated causes of such failure or degradation (as specified in Warren Contention 2) were increased seepage, flooding, failure of pumping systems, and power outages. During the hearings we heard testimony on the design and performance of the permanent dewatering system, the flow patterns and rates of water-level rise in the absence of any pumping, isolation of the ground water in the power-block area from laterally and vertically proximate regional ground water aquifers, and the proposed water-level monitoring system. We also heard testimony on the ability of the permanent dewatering system to detect and remove water from potential breaks in underground pipes and from infiltration resulting from the 100-year maximum precipitation.

Because the potentially liquefiable plant fill sands lie above 610-foot elevation, a principal design objective of the permanent dewatering system was to lower and maintain the ground water level beneath the RBA and DGB below that 610-foot level. In order to do this, it was planned to lower the ground water level beneath those structures to elevation 595 feet. At that level, even if total failure of the system occurred, there would be adequate time to repair or replace equipment in the dewatering system, or to shut down the plant before the ground water level beneath the RBA and DGB rose to the 610-foot elevation. Based on results of a recharge test, in which the water level was drawn down to below 595 feet and all pumps were then turned off, a minimum of 40 days would be required for the water level to rise to the 610-foot elevation beneath either of the two potentially affected structures.

Redundancy was to be provided to ensure effectiveness and reliability of the pumping system. Twenty interceptor and twenty backup interceptor wells located in two lines along the primary recharge area (near the SWPS), and twenty-four area wells in the plant area form the main components of the permanent dewatering system. One line of interceptor wells and only two area wells would need to remain in operation to dewater the RBA and DGB areas to the design level. All of the wells, however, would have been kept operational, should the need for any of them have arisen. One complete set of discharge well replacement parts was to be kept on site for quick repair or replacement, if needed. Also, electrical wiring was to be designed so that a temporary outage of one or more wells would have no impact on power to the other wells. In the event of a loss of power to the system, a separate diesel generator was to be provided to power the interceptor wells.

The discharge collectors, or header systems, were to be separate for the two lines of interceptor wells. If failure of one header system occurred it would not affect operability of the other. Also, individual wells could have flexible hoses attached to their outlets, bypassing the header systems entirely, in the event of header rupture underground near one or more dewatering wells. This was to prevent overloading the pumping capacity if water from a ruptured header "flooded" a well in the pumping area. Water from the system was to have been pumped back to the cooling pond.

The discharge wells were each equipped with well screens and filter packs to prevent removal of soil fines from the soils through which the ground water percolated. Monthly sampling of fines was to have been required to check on continued serviceability of the filter packs during the operating life of the plant. Actual tests to check for possible discharge of soil fines were conducted for each well, and all were indicated to be well below the Staff's acceptance criteria.

Water quality samples were to be taken annually during plant operation to determine concentrations of compounds associated with encrustation. Acid treatment of the wells would have been employed to remove encrusting minerals, if needed.

Six permanent water-level monitoring wells were to have provided continuous recordings of water levels during plant operation, and alarms to warn plant personnel of a significant rise in level at any well. Two of the six monitoring wells were to have been located near the DGB, and two near the RBA. The remaining two were to have been placed between each of those structures and the main recharge area. A technical specification would have required the initiation of plant shutdown if the water level beneath the RBA or DGB rose to 606.5-foot elevation. It was determined during the recharge test that it would take about 8.5 days for the water level to rise from elevation 606.5 feet to 610 feet. To reach cold shutdown would require about 36 hours.

The Applicant and the Staff each analyzed the impact of various pipe breaks on ground water levels and considered the ability of the permanent dewatering system to detect a water-level rise and to maintain water levels below 610-foot elevation at the DGB and RBA. The analyses included postulated breaks of the low-pressure 66-inch-diameter cooling-pond-blowdown line near the SWPS and the 96-inch-diameter Unit 2 circulating-water pipe near the DGB. Also, the effect of a postulated break in the 20-inch-diameter condensate pipe, which runs directly beneath the DGB, was analyzed. The Applicant and Staff agreed that, in all of these analyses, conservative conditions were assumed and that, even if the monitoring wells failed to alarm, the ground water level would not rise significantly above the 610-foot limiting elevation.

Because of the hydraulic isolation of the power block area and the flood protection provided by the plant dikes, the only source of flooding that might challenge the dewatering system would be from precipitation falling within the cooling-pond and power-block areas. Using the predicted 100-year-maximum precipitation, an analysis of the impact of this flood on ground water levels was made. The Applicant's and Staff's experts both concluded that the dewatering system could accommodate the runoff and infiltration from this precipitation and that it would not result in the ground water level rising to 610-foot elevation.

The impervious, widespread natural clay layer, about 135 feet thick, that underlies the plant site area, together with impervious dike cores, cutoff dikes and slurry trenches designed to extend down to the natural clay, provide hydraulic isolation of the cooling-pond and power-block areas from regional ground water systems. The dikes and slurry trenches prevent hydraulic connection with laterally adjacent shallow sediments where ground water occurs under water-table conditions. A confined aquifer of a lower ground water system, located beneath the essentially impervious 135-foot-thick clay layer is under artesian pressure with a hydrostatic head about equal to the water-table level of the upper ground water system. Observation wells drilled to the lower aquifer outside the dike perimeter showed no fluctuations of water level with changes of water level inside the dike and above the clay layer, indicating a lack of hydraulic connection. The casings of these wells drilled through the clay were grouted to prevent a connection whereby ground water could rise from the lower aquifer to the upper system. (Water flow in the other direction would be prevented by the artesian pressure in the lower aquifer.)

This Board concludes that, contrary to Stamiris Contention 4.D (and to Warren Contention 2), while the water table, soil, and seismic characteristics of the site would be changed as a result of dewatering, the Applicant has adequately taken these changed characteristics into account in evaluating and designing safety-related structures, piping and duct banks to resist future soil settlement loads (including those from soil consolidation and seismic shakedown) and other loads attributable to the effects of dewatering. We also conclude that, except with respect to the diesel fuel oil tanks, we have reasonable assurance that soil liquefaction will not affect the integrity of safety-related structures, piping or electrical duct banks during an earthquake as large in magnitude and associated ground acceleration as the SSE determined to be appropriate for this site, providing the permanent dewatering system lowers and maintains the ground water level to below elevation 610 feet beneath the RBA and DGB. (For reasons indicated earlier, we are not now ruling on liquefaction in the diesel fuel oil tank area.)

We also have reasonable assurance that the Applicant has provided adequate redundancy and other features in the design of the permanent dewatering system to reduce the likelihood of, or to obviate, failure or degradation of the system in the event of seepage, flooding, failure of pumping systems and power outages, over the life of the plant, if the plant were to be operated. The Applicant has provided reasonable assurance that, if the plant were completed and operated, its design of the permanent dewatering system (including water-level monitoring) will maintain the ground water level below elevation 610 feet, even in the event of total failure of the system, and will provide adequate time to repair or replace parts of the system, or to bring the plant to cold shutdown before the ground water rises to the 610-foot level of the potentially liquefiable sands beneath the RBA and DGB.

We also conclude that the Applicant has accounted for the effects of temporary drawdown of ground water levels during construction on the settlement of soils and the safety-related structures founded, or to be founded, on them. We note that Ms. Stamiris, in her proposed findings (Stamiris FOF, ¶ "13," item 9, at 6), refers to "continued water seepage problems in the underpinning excavations" as an unresolved question. However, in a previous Memorandum and Order (Denying Motion to Reopen Record on Containment Cracks), LBP-83-50, 18 NRC 242, 249-51 (1983), we ruled, *inter alia*, that Ms. Stamiris had misinterpreted reports on water seepage and that there was no persuasive connection between cracks in the containment buildings and dewatering, including construction dewatering of the natural clay on which the containment (and auxiliary) buildings are founded, or that settlement due to dewatering has been excessive. We reaffirm those rulings.

III. DIESEL GENERATOR BUILDING

As we previously pointed out (*supra* p. 37), we are not at this time formally making any findings or rulings with respect to the structural adequacy of the diesel generator building (DGB) or the sufficiency of the corrective measures which have been applied thereto as a result of soils settlement problems. Because of its significance with respect to various OM and several OL issues, however, we believe that a brief description of the DGB structure, the problems which have surfaced following its construction, and the corrective actions which have been followed would prove instructive and useful as background for considering the soils-related issues discussed elsewhere in this Decision.

The DGB, which is located directly south of the turbine building, is a rectangular, reinforced concrete, box-like structure which was to house four diesel generators. It is partitioned into four bays, one for each generator. The generators themselves rest on thick concrete pedestals which are structurally independent from the rest of the DGB. Both the DGB and its generators are classified as Seismic Category I items and hence are subject to the QA requirements of 10 C.F.R. Part 50, Appendix B.[21]

The DGB foundation consists of continuous spread footings around the building and beneath the three interior walls, resting upon approximately 30 feet of plant fill. Fill placement activities took place mainly from October 1975 to October 1977; the footings for the DGB were poured in October 1977, and construction of the building was carried out from that time until the Spring of 1979. During the course of construction, in July 1978, it was discovered that the DGB had settled in excess of that which would have been expected throughout the entire plant life. As of August 23, 1978, when construction on the building was temporarily halted as a result of the settlement problem, 55% of the concrete had been placed, with the walls in place to an elevation of 30 feet above grade, the generator pedestals poured, the mud mat poured inside the building, the electrical duct banks placed under the building with horizontal and vertical runs completed, the underground piping in the area under and adjacent to the building installed, and all backfill placed to grade level. In other words, with approximately half the construction completed and half the static structural load in place, the DGB settled to a greater degree than would have been expected throughout plant life, during which greater loads could be expected.[22]

The safety implications of the excessive settlement of the DGB gave rise to an OL contention of Ms. Sinclair (originally designated as Sinclair Contention 24, *see* Special Prehearing Conference Order dated February 23, 1979, at 8), questioning the suitability of the fill soils on which the DGB was founded. Mr. Marshall advanced a similar contention (*id.* at 21). Thereafter, the "unusual settlement" of the DGB formed the basis for the December 6, 1979 Modification Order, which raised questions as to an asserted "breakdown in quality assurance related to soil construction activities," the adequacy of corrective actions which had been followed up to that time or acceptance criteria for such actions which had been submitted, and an alleged material false statement in the FSAR concerning the condition of the plant fill. Finally, following the initiation of the OM proceeding, Ms. Stamiris raised numerous contentions bearing upon the DGB, including the managerial attitude which led to the extensive QA/QC violations, asserted financial and time schedule pressures affecting resolution of the soils settlement issues (including the nature of the corrective measures selected by CPC for the DGB), and

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[21] Wiedner, ff. Tr. 10,790, at vi, 1, and Figs. DGB-1, DGB-2, DGB-3; SSER # 2, § 2.5.4.4.2, at p. 2-24, and § 3.8.3.4, at p. 3-22.

[22] Keeley, ff. Tr. 1163, at 6; Tr. 3222-23 (R.B. Peck); Wiedner, ff. Tr. 10,790, at vi.

the asserted technical inadequacy of the DGB corrective actions.[23] In particular, Ms. Stamiris claimed that the proper corrective action for the DGB structure would have been the removal and replacement of the partially completed structure.

The remedial actions which in fact were chosen by CPC for the DGB, upon the advice of consultants who included Dr. Ralph B. Peck, a Professor of Foundation Engineering Emeritus, of the University of Illinois, and Dr. A.J. Hendron, Jr., Professor of Civil Engineering at the University of Illinois, were the severing of duct banks and conduits beneath the structure (to alleviate stresses resulting from differential settlement), the resumption of construction and completion of the DGB structure, and the surcharging or preloading of the structure with about 20 feet of sand over and around the soils under the DGB foundation. Construction was resumed in December 1978. The surcharging was begun in early 1979 and was essentially completed, and the sand removed, by the end of August 1979, prior to the issuance of the Modification Order. The remedial actions for the DGB further called for permanent dewatering of the plant fill in the vicinity of the DGB, to preclude liquefaction developing as a result of seismic stress in the underlying and adjacent sandy fill soils.[24]

The purpose of surcharging was to cause the soil to settle at an accelerated rate so that, under operating loads, future settlement would be small and within tolerable limits. The procedure was also intended to permit a conservative and reliable estimate of the amount of future settlement.[25] During the course of the hearing, however, significant questions were raised concerning such matters as whether the severing of the duct banks was performed in a manner which would keep stresses to the DGB structure as low as possible, whether the surcharge was left in place for a sufficient time to attain secondary, or to complete primary consolidation of the fill,[26] and whether sufficient reliable data were recorded to provide an adequate basis for future settlement estimates.[27]

Furthermore, the Staff recognized that surcharging the essentially completed DGB structure did nothing to avoid the undesirable and large total and differential settlements that had occurred, with the accompany-

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[23] Contentions of Ms. Stamiris specifically concerning the DGB are OM Contentions 1, 2(b) and (d), 3(c), 4.A, and 4.C(e). Ms. Warren's three contentions also dealt with the technical adequacy of the DGB corrective actions. *See* Appendix A to this Decision for a listing of all soils-related contentions.

[24] Wiedner, ff. Tr. 10,790, at 2-4; Keeley, ff. Tr. 1163, at 8; SSER # 2, § 2.5.4.4.2, at p. 2-31.

[25] R. Peck, ff. Tr. 10,180, at 6.

[26] The Applicant regards primary consolidation from the surcharge as that resulting from the dissipation of excess pore pressures and secondary consolidation as settlement that occurs after excess pore pressures have been dissipated. R. Peck, ff. Tr. 10,180, at 8-11.

[27] SSER # 2, § 2.5.4.4.2, at 2-24 and 2-31.

ing concern for warping and cracking. The settlement originally predicted for the DGB throughout its projected life had been 2.8 inches. By December 1978, prior to the surcharge, the largest measured settlement, located in the southeast corner of the DGB, had reached 4.25 inches. Following removal of the surcharge, the total settlement for this portion of the DGB had reached 7.45 inches.[28] One Staff witness estimated the amount of *differential* settlement between various segments of the DGB to have been about 7.5 inches and to have resulted in structural cracks in the building.[29]

There developed a difference of opinion among several Staff witnesses, and between the Applicant and the Staff, as to the significance of cracks in the DGB. Those cracks were caused in part by the differential settlement of different portions of the DGB, including that caused by application of the surcharge. The Applicant performed a structural reanalysis of the DGB, using a finite-element model to estimate stresses in the DGB.[30] It also presented experts who testified as to the observed condition of the DGB.[31]

The Staff's structural engineers considered the Applicant's approach to be consistent with sound engineering practice.[32] However, these structural engineers actually evaluated the structural adequacy of the DGB on the basis of a crack analysis, and they added the residual stresses calculated from crack widths to the stresses calculated in the Applicant's finite-element analysis.[33] The Staff's geotechnical engineers, on the other hand, raised questions as to the sufficiency of the Applicant's approach, and criticized the method of the structural engineers as not being normal engineering practice.[34] Moreover, an NRC Staff inspector in April 1983 expressed considerable doubt about the structural adequacy of the DGB, based in part upon similar considerations but also upon the design of the DGB utilizing spread footings founded upon fill.[35]

Because of the internal Staff differences of opinion with respect to the analyses of the DGB cracks and with regard to the structural adequacy of the DGB, the Staff commissioned Brookhaven National Laboratory to perform a further study. When completed, this study was reviewed by a

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[28] *Ibid.*

[29] Tr. 16,429 (Landsman).

[30] Wiedner, ff. Tr. 10,790, at 14-17.

[31] Sozen/Corley, ff. Tr. 10,950, Attachment 4, at 4.11, 4.34.

[32] Rinaldi, *et al.*, ff. Tr. 11,086, at 6, and Tr. 11,121-24 (Rinaldi).

[33] Rinaldi, *et al.*, ff. Tr. 11,086, at 2-5.

[34] Tr. 10,521, 11,187-88, 11,196-99 (Kane); Tr. 11,177-81, 11,189-90, 11,202-03 (Singh).

[35] Tr. 15,059-60, 16,410-13, 16,816-17 (Landsman). He also expressed these concerns to a congressional oversight committee in June 1983. The Staff testified, however, that there is no regulatory requirement that would preclude the use of spread footings on diesel generator buildings. Tr. 16,424-25 (Hood).

Staff task group, which prepared a report. The study and report were then reanalyzed by Staff witnesses to ascertain whether their earlier testimony would have to be changed.[36] Although opinions on the need to reopen the record were not unanimous, reviewers agreed that, at the least, further documentation of calculations which had been performed was needed. This documentation was still in progress at the time we declined to grant (pending completion of the review process) the Staff's motion to reopen the record on the DGB but also permitted the Staff and Intervenors to defer filing their proposed findings and conclusions with respect to that structure. *See supra* p. 37. Any final resolution of questions concerning the structural adequacy of the DGB would, of course, have to include a satisfactory resolution of the crack issues which we have been discussing.

In addition to the soils settlement questions, there have been other QA problems associated with the DGB which have been extensively litigated. In particular, a Staff inspection performed by Region III from October 12 to November 29, 1982 and January 19-21, 1983, primarily of work accomplished in the DGB, indicated (according to the Staff) another "significant breakdown" in the implementation of CPC's QA program. The Staff also proposed substantial civil penalties as a result of the violations which had occurred.[37] CPC as a result suspended most nonsoils-related work on the DGB (as well as other portions of the project) from early December 1982 to October 1983 (when the Staff approved CPC's Construction Completion Plan), and it paid the civil penalty after its request for mitigation was turned down by the Staff.[38] The Construction Completion Plan, under which construction of the DGB was resumed, applied to nonsoils-related construction activities; it included the application to those activities of Staff controls analogous to those which we earlier imposed on soils-related construction activities by LBP-82-35 (*see supra* p. 35). The general implications of the QA deficiencies at the DGB, as well as the potential effectiveness of the Construction Completion Plan, were extensively litigated before us as QA/management attitude issues (on which we are not at this time ruling).

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[36] The Board and parties have been kept advised of the progress of this review through several Board Notifications from the Staff. *See* BN 83-109 (July 27, 1983); BN 83-142 (September 22, 1983); BN 83-153 (October 11, 1983); BN 83-165 (October 26, 1983); BN 83-185 (December 2, 1983). The BN 83-165 notification includes copies of the Brookhaven report and the report of the NRC task group. BN 83-185 includes recommendations of several witnesses on whether the record should be reopened. Neither these notifications, nor their attachments, have thus far been entered into the evidentiary record of these proceedings.

[37] *See* Keppler, ff. Tr. 15,114, at 4-5, Attachments 3, 4, and 7.

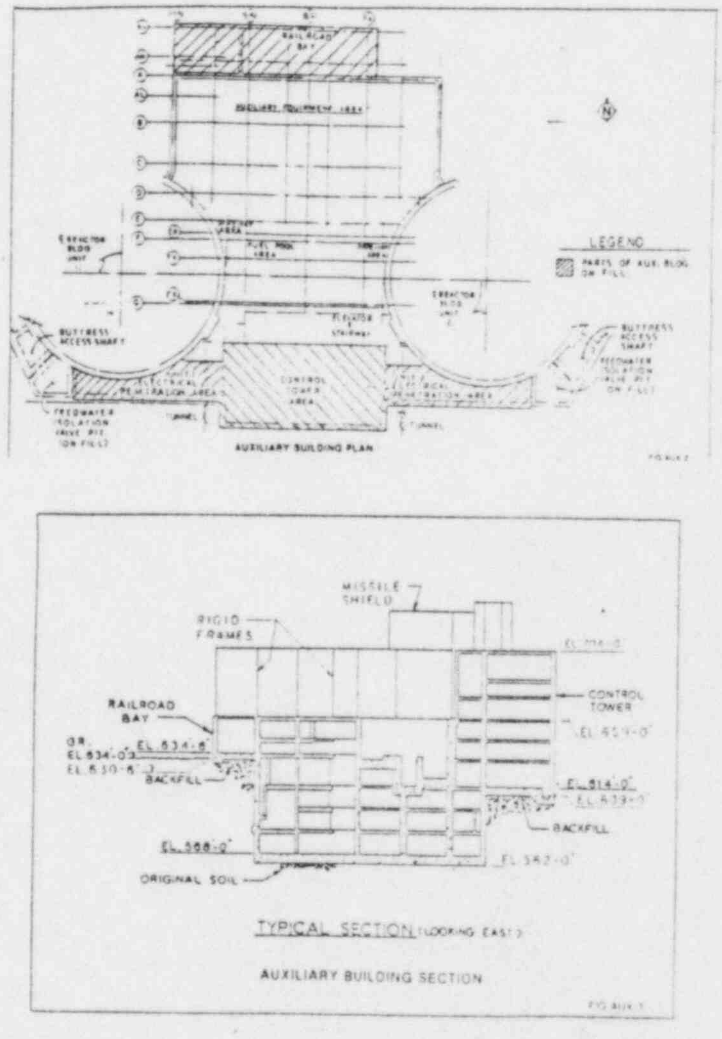
[38] Tr. 15,074, 15,086 (Shafer, Gardner); J. Cook, ff. Tr. 18,025, at 5; Letter to Board and parties from Staff, dated December 15, 1983, transmitting Confirmatory Order for Modification of Construction Permits (Effective Immediately), dated October 6, 1983.

IV. AUXILIARY BUILDING AND FEEDWATER ISOLATION VALVE PITS
(Findings 118-151)

The auxiliary building is made up of several parts. The main portion is founded on the same overconsolidated hard clays of lacustrine origin as are the containment buildings, which lie immediately to the east and west. Other parts of the auxiliary building project to the north (railroad bay area (RBA)) and to the south (control tower and electrical penetration area (EPA) wings) and are founded, at elevations higher than the footings of the main structure, on backfill. *See* Figure 4, *infra* p. 87, for the identification and arrangement of the several parts of the auxiliary building. Each of the feedwater isolation valve pits (FIVPs) is situated immediately outboard of an EPA wing and slightly beyond the line projecting southward from the center of the respective containment building that each serves. Although the FIVPs are structurally independent of the EPAs, they have been discussed in these proceedings along with the auxiliary building structures. The FIVPs are founded on plant backfill, like the EPAs, control tower, and RBA. All of these structures or substructures contain safety-related equipment and are required to be designed to Seismic Category I standards.

Following discovery in 1978 of excessive settlement of the DGB, the Applicant undertook a soils exploration program. At the time, construction of the auxiliary building and FIVPs was essentially complete. This program gave rise to various concerns about the integrity of the RBA, control tower, EPAs and FIVPs. In the Staff's opinion (*see* discussion, *infra* p. 93), the program revealed inadequately compacted backfill supporting these structures, demonstrated by differential settlement of the south end of the control tower, the location of cracks in the auxiliary building, and a 1-foot void between a concrete mudmat and the underlying plant fill. Potentially liquefiable sands in the fill were found above the 610-foot elevation beneath the RBA and EPAs. Clay soils in the fill posed a concern for differential settlement and attendant structural loads in the FIVPs and the EPAs.

Concern for the adequacy of the fill beneath the control tower arose partly from questions about the effect of added foundation loads from the attached EPAs, resulting from an early plan to support the other, or outer, ends of each EPA by caissons. Partial loss of support of the EPA foundations through soil compression would have produced a bridge-like effect, adding loads to the supports at either end. The loads thus added to the control tower from both EPAs might have resulted in an insuffi-



A

B

FIGURE 4. Auxiliary Building: (A) Plan view showing location of portions of structure founded on plant fill, and (B) cross-section showing stepped foundations with projections founded on plant fill. (Compiled from Figs. AUX-2 and AUX-3, Burke, *et al.*, ff. Tr. 5509.)

cient safety margin in the dynamic bearing capacity of its supporting backfill under earthquake loading conditions.

The proposed caisson-support remedy for the EPAs was subsequently abandoned. The approach that was eventually selected to eliminate concerns about the plant fill entailed underground construction of new foundation walls (underpinning) beneath the control tower as well as the EPAs in order to transfer their support directly to the underlying hard lacustrine clay. Also, the plant fill beneath the FIVPs would have been removed by excavation down to the clay and was to be replaced with properly compacted granular fill capped by a concrete jacking pad. The jacking space would finally have been filled with grout. Potential soil liquefaction concerns for the plant fill beneath the RBA (like the DGB) were to have been remedied by lowering and maintaining the ground water level below 610-foot elevation. (Our analysis of dewatering is discussed *supra* p. 71, *et seq.; also see infra* Findings 98-116).

The adequacy of the Applicant's proposed remedial measures to resolve questions of safety of the auxiliary building and FIVPs (and of other safety-related structures) stemming from the improperly compacted plant fill was questioned by the Staff in the Modification Order and challenged by Ms. Stamiris in her OM contentions. In her Contention 4.C(a), Ms. Stamiris asserted that the Applicant's remedial actions are not based on adequate evaluation of dynamic responses regarding dewatering effects, differential soil settlement effects and seismic effects.

The Applicant considered the effects of dewatering in its most recent design of the remedial measures (e.g., underpinning) for the auxiliary building and FIVPs. In addition to eliminating concern for soil liquefaction, dewatering also removes the effect of buoyancy caused by ground water on individual soil particles, and thus increases the load on the affected foundation soil. As a result, dewatering would increase the bearing capacity of the soil, a beneficial effect, but also would increase the settlement and rate of compression of the soils. The dewatering effect is small and predictable, based on the load added by the loss of buoyancy. Part of the settlement, or soil compression, is recoverable upon removal of the dewatering load when the ground water level is allowed to rise. Subsequent fluctuations of water level cause only minor settlement, if any, from the dewatering load after the initial effect has occurred.

To counter possible structural effects of temporary (construction) dewatering on the FIVPs and EPAs, temporary support systems were installed before underpinning began. A beam-and-tie system provided support for the FIVPs, and post-tensioning ties were installed through the control tower and attached to the upper part of the east-west walls of the

EPAs on either side. (Similar post-tensioning ties were applied to the SWPS north-south walls, as well. *See infra* Finding 156.)

The basic underpinning plan for the control tower and EPAs called for construction of piers beneath the existing walls, extending down to the hard clay. Construction was to be of reinforced concrete, cast in place. The bottoms of the piers were to be belled-out to increase the pier footing support area and to cause the bottom of each pier to touch its neighboring piers. After completion of the piers, walls were to be constructed in the intervening spaces between them, with provision made for tying the underpinning walls of the control tower and EPAs together and for fixing the walls to the supported structures, after jacking pressures between the piers and the supported structures were locked off.

The hydraulic jacking system between each pier and the supported structure was designed to preload the supporting hard clay soil, to ensure that full initial and elastic recompression of the soil was attained, and to provide a period of observation of secondary compression of the soil. The Applicant developed a schedule of jacking pressures at the different piers, to prevent nontolerable movements in the supported structures during construction and the period of soil preloading.

Horizontal and vertical motions of the structures were to be monitored during construction and jacking. Alert and action level limits of structure motions, based on tolerable limits, were to be established, and the movement data were to be checked for trends indicating that an alert level might be reached. Corrections of structure movements were to be made by adjusting jacking pressure on individual piers, and provisions for emergency mechanical support systems were to be made in the event of the possible occurrence of settlements not correctable by the methods planned. Loads in the piers as well as pier deflection were also to be monitored during construction of the underpinning. Cracks in the structures were mapped and were to be monitored as a check against predicted structure deflections. Monitoring of cracks and structure motions would have been continuing requirements if the facility were to be completed and operated.

The jacking procedures were intended to prevent or relieve any structural overstressing. The competency of the hard clay providing foundation support was determined to be adequate to preclude development of structural loads arising from differential settlement that, when combined with other loads, would be unacceptable. *See infra* Finding 138. While the testimony indicated that design changes could be implemented during underpinning construction — e.g., widening the pier bases to increase bearing area — we heard little or nothing about specific circumstances that might warrant such changes, only that the construction sequence and procedures could accommodate the option during the time prior to completion of the final design calculations. (*Cf.* our discussion, *infra* p. 91, of the unsuccessful pier W-11 load test.)

The underground construction sequence was planned so as not to weaken the foundation support excessively during removal of soil and installation of the piers and temporary structure supports. The plans also included measures to support walls of the excavations. The underground construction area was dewatered to an elevation about 30 feet lower than the planned permanent dewatering ground water level. To facilitate the construction dewatering, a freezewall was emplaced by circulating refrigerant fluids through boreholes that were closely spaced in lines around part of the work area (*see infra* Findings 135, 136). Construction proceeded from two access shafts dug on the east and west ends of the affected area and then from a tunnel between them located beneath the turbine generator building. The work was to progress in a stepwise fashion, tunneling far enough to construct temporary supports, constructing them, then tunneling far enough to accomplish the next part of the construction, constructing it, and so on.

Prior to the suspension of work activities on the project, a considerable amount of the underpinning construction had been accomplished. We understand that the Applicant intends to leave the underpinning, like other project construction, in a safe layup condition. *See* Board Notification 84-148, dated September 14, 1984, at 2 and Enclosure 3; I&E Report 84-25/26 (attachment to letter from R.F. Warnick to CPC, dated September 21, 1984). While the plans for activities to accomplish this (and including reporting requirement changes) are not now included in the evidentiary record, we regard such activities as subject to Staff approval pursuant to the Work Authorization Procedure adopted as a result of LBP-82-35, *supra*.

In evaluating the design of the remedial measures for the control tower, EPAs and FIVPs, the Applicant took into account the loads that would be imposed by postulated seismic events (as well as flooding events). Because the SSRS were not yet agreed upon when the initial design of the remedial measures was developed, seismic loads equal to 1.5 times the loads which would result from use of the DBE (or FSAR SSE) response spectra were used in the actual design. Subsequently, this design basis was demonstrated to be conservative: analyses performed by the Applicant's consultant, and an audit of the Applicant's design calculations by the Staff, determined that loads equal to 1.5 times the DBE (FSAR SSE) loads are conservative in relation to loads which would result from application of the now-agreed-upon SSRS (Finding 142).

Although we have reasonable assurance that the seismic designs of the auxiliary building and FIVPs are acceptable, this conclusion applies only to the extent that those designs are based on the nominal value of the dynamic soil modulus (or soil spring constant) used in the seismic analyses. This limitation stems from the design deficiency of which we were advised by Board Notification BN 84-115. *See supra* pp. 70-71.

A load test conducted on pier W-11 to evaluate soil parameters and settlement response of the lacustrine clay did not produce the results expected (Finding 140). Carlson stress meters installed on the pier indicated that the load applied at the top by jacking was not reaching the bottom of the pier. The Applicant ascribed the failure of the test to produce the expected results to a deficiency in the anti-friction installation; the Staff did not accept this explanation, but proffered no explanation of its own. Further, we were advised by both the Applicant and Staff that the pier, which was test-loaded initially to 130% of its design pressure, settled more than predicted (but we could not find in the record any testimony as to whether this was during, or subsequent to, the pier load test). Implicit in the indication that the load was not reaching the bottom of the pier, as well as in the Applicant's explanation, is the suggestion that some of the load was being transferred to the surrounding fill soil, and hence the load at the bottom was spread over an area of the supporting clay larger than the area of the pier footing alone. The observation that the pier settled more than was predicted, however, would apparently contradict the notion that the pier footing had not been fully loaded.

As a result of the unsuccessful pier load test, the Applicant reanalyzed the structure for settlement loads using an assumed settlement of ½ inch instead of the originally calculated ¼ inch. Such procedure was equivalent to assuming the soil modulus used for calculating settlement to be one-half that employed in the original calculations.

Following a design audit of the Applicant's reanalysis of the auxiliary building differential settlement loads using ½ inch, the Staff issued Board Notification BN 83-174. *See infra* Finding 127. The three open items that the Staff cited as relevant to soils-remedial activities potentially at issue in these proceedings concerned (1) the baseline length over which the ½-inch differential settlement of the control tower relative to the main auxiliary building, and hence the stresses in the structure, were to be calculated; (2) the permissible limits of vertical deflections of the structures during jacking operations; and (3) how existing settlement stresses in structures will be treated in the final analyses of stresses and combined loads in the structures, i.e., can all existing stresses be removed during final jacking? Because these design issues were not fully addressed by the testimony of record, we accordingly make no findings at this time as to the adequacy of their treatment by the Applicant. We note, however, that the Applicant's witnesses did address the last two items in their testimony (*see* Burke, *et al.*, ff. Tr. 5509, at 42-43, A9-A15), and we regard these two items as reasonable disagreements between experts that are susceptible to eventual resolution. We regard the final resolution of these items as subject to the Work Authorization Procedure established pursuant to LBP-82-35, *supra*.

In addition to BN 83-174, the absence of agreement among the expert witnesses as to the underlying reasons for failure of the W-11 pier load test to produce expected settlements provides a further reason for our declining to rule at this time, because of prematurity, on the issue of differential settlement between the control tower and the main part of the auxiliary building in the design of the underpinning. While the Staff's and Applicant's expert witnesses attested to the general competency of the hard lacustrine clay, a conclusion which the Board accepts as well-supported and reasonable, the final design of the underpinning was to rely on observations of settlement data. Data from pier W-11 settlements were to comprise part of that data base. In light of our concern, arising from the Modification Order (at 13-14), that acceptance criteria be sufficiently established to assure adequate design of the proposed underpinning prior to its construction, failure of the W-11 pier load test casts doubt on the foundation design or construction procedure. While we might envision several causes for that failure, evidence in the record is insufficient for us to reach a conclusion at this time about the relevance and significance of the unsuccessful load test to the foundation design acceptance criteria.

The Staff and Ms. Stamiris, in their proposed findings, both questioned the absence of any discussion of the unsuccessful pier load test in the Applicant's proposed findings. Staff FOF, ¶ 228; Stamiris FOF, ¶ "11," at 4. The Staff pointed to the test's relationship to the design audit conducted on September 14 and 15, 1983, and to the question of adequacy of the Applicant's treatment of differential settlement between the main portion of the auxiliary building and the control tower consequently raised in BN 83-174.

Ms. Stamiris went further, alleging that there had been a "discovery that the bearing capacity of the base soils for the underpinning is ½ that used in the original analysis" (Stamiris FOF, ¶ "13," item (1), at 5). It appears that Ms. Stamiris has confused bearing capacity with the soil modulus and erroneously concluded that circumstances leading to the Applicant's assumption of ½ inch (rather than ¼ inch) differential settlement necessarily implies a lack of competence of the base soil layer. The general quality of that clay layer as a foundation support was demonstrated through laboratory tests of the clay, *in situ* Standard Penetration Tests, and agreement between predicted values of settlement of structures founded on the clay with actual settlements measured. The main purpose of the test was to verify the soil parameters. While we must reject Ms. Stamiris' conclusions about the clay that stem from the unsuccessful pier load test, we repeat that the evidentiary record on the pier load test (and on the three items cited by the Staff in BN 83-174) is incomplete.

Ms. Stamiris also registers her dissatisfaction with the Applicant's and Staff's treatment of the cause of cracks in the auxiliary building that began to appear before remedial actions were initiated. Stamiris FOF, ¶ "10," at 4; App. FOF, ¶ 217; Staff FOF, ¶¶ 216-218. As we outline, *infra* Findings 123-125, the Applicant believed the subject cracks were attributable to volume changes in the concrete during curing. The Staff did not accept the explanation that all the cracks in the auxiliary building stemmed from volume changes; nor do we. Importantly, the Staff required the Applicant to evaluate the effect of cracking on all safety-related structures, and the Applicant did so. The Staff opined that the Applicant's crack assessment in the case of the auxiliary building was satisfactory. We agree. Ms. Stamiris' accusations that this treatment indicated evasiveness on the part of the Applicant and that the Staff attempted "to skirt this issue altogether" are unwarranted, particularly since she gave no indication as to why a finding on the cause of the cracking might be significant. Since our findings indicate that the cracks do not significantly affect the strength of the auxiliary building, and since the cracks were to be monitored for changes in size or new crack development, we attach little significance to the fact that some of them may have been caused by differential settlement, except in regard to the allegation that the stepped foundation design of the structure may be deficient. That allegation we address immediately below.

During the hearings an NRC Staff engineer, Dr. Ross Landsman, volunteered that several "design deficiencies" occurred at the Midland facility. One category of these alleged deficiencies included the stepped-foundation configuration present in the RBA, control tower and EPAs of the auxiliary building, and the north projection of the SWPS. In this configuration, where the main part of the structure is founded on hard soil, an extension projects from it so that its foundation is at a higher level and rests on backfill of considerable thickness. Dr. Landsman asserted that this stepped-foundation design had an inherent potential for developing problems as a result of differential settlement, even if satisfactory compaction methods were used on the backfill. The overhanging portion could act as a cantilever if the backfill supporting it settled more than anticipated in the design.

Since this potential differential settlement is principally what the Midland underpinning was intended to remedy, by transferring the foundation loads to the deeper hard soil, the potential safety problems to which the "cantilevered" design might give rise would be adequately resolved for the Midland structures. While this design was said (by others) to represent an acceptable engineering practice (indeed other examples have been accepted on licensed nuclear power plants), we are making no findings here on the adequacy of the *original* design of the auxiliary building. *See infra* Finding 128.

We recommend, however, that in the interest of conservatism the Staff study and review the practice of using cantilevered designs. That is, should stepped-foundation designs be utilized at all on nuclear power plant safety-related structures and, if so, should the NRC provide specific guidance on composition of backfill materials and their distribution, compaction standards or possible methods for assuring attainment of secondary consolidation of the backfill to control differential settlement when this design is utilized? While the record is not sufficiently detailed to permit this Board to specify its concern in clearer detail, and while we recognize that the potential problems of differential settlement in this case arose mainly from inadequate control of placement, moisture content, and compaction of the fill materials, the stepped-foundation design on certain structures, particularly those underlain by clay fill, appears to have contributed to the structural aspects of the potential differential settlement problem. Included in our concern is the practice of using concrete as fill material unless its use is specifically planned and the location of such materials in the fill is recorded and utilized in settlement predictions.

In summary, this Board concludes that the Applicant has adequately taken into account, in its design of remedial actions for the different parts of the auxiliary building and FIVPs, the effects of dewatering, seismic shaking (including potential soil liquefaction and seismic shakedown) and, except for open items specified in Board Notification BN 83-174 on which we express no opinion, differential settlement. As regards the seismic effects, we have reasonable assurance that the Applicant's use of the site-specific response spectra (SSRS) determined for the Midland site is appropriately conservative for assuring the seismic safety of the design of the underpinning of the auxiliary building structure and FIVPs, and that the response spectra used by the Applicant in the design of those underpinnings, based on a 1.5 multiple of the original DBE (or FSAR SSE) response spectra, adequately envelope (are higher than) the Midland SSRS. *See* our conclusions on seismic effects, *supra* pp. 68-69, and *infra* Findings 77-79. In regard to the seismic reevaluation of these structures, we have reasonable assurance that the general analysis methodology proposed by the Applicant, the seismic design basis (1.5 x DBE (or FSAR SSE) response spectra), and the *nominal* values for the soil spring constant (or dynamic soil modulus) to be used are appropriately conservative input for the planned seismic evaluations of the completed structures, should construction ever be resumed. Our conclusion on the soil spring constant is subject to resolution of the Applicant's failure to meet its commitment given in the FSAR, and relied upon in testimony (including the SER), to perform additional structural evaluations for the seismic margin review using ± 50% values of the nominal soil spring constant, as discussed *supra* pp. 70-71.

In the record on which we rely to come to our conclusions concerning adequacy of the Applicant's consideration of effects of dewatering, soil compression, and seismic shaking in the design of the remedial actions, we have attached considerable weight to evidence of the properties and predicted performance of the supporting soils under different loading conditions. Also, assurance that adequate consideration has been given to tolerable limits of structural response, or behavior, is inherent in our conclusion that the designs, if properly executed, will lead to structures posing no unreasonable threat to the health and safety of the public, or to the environment, if project construction were resumed. In other words, our conclusions here would be altered if greater differential settlement values or limits of structure deflection occur, or are proposed.

Our conclusions, also, are conditional upon satisfactory performance to be demonstrated by results of the structure-movement and crack-monitoring programs that have been, or were to be, initiated by the Applicant. (This conditional acceptance applies equally to other structures, pipes, and duct banks where monitoring programs were to be initiated.) We attach special significance to the results, as well as to the proper and continuous conduct, of the monitoring programs. Not only are they the "proof of the pudding" on predictions of soil performance and acceptable limits of structural deflection, but also their time-dependent data will be essential to a full understanding of the condition of structures if construction is ever resumed. The time-dependent nature of the soil responses — e.g., settlements ascribable to primary and secondary compression rates, or correlation of settlements with changes in ground water levels — was important evidence in our deliberations.

V. SERVICE WATER PUMP STRUCTURE
(Findings 152-167)

The service water pump structure (SWPS) is a rectangular, reinforced concrete building with upper and lower sections of the same width but different lengths. The larger upper section results in an overhang at the north end of the structure, supported by underlying soil. *See* Figure 5, *infra* p. 97. Excavation for the SWPS left areas under the overhang to be backfilled; borings taken later revealed that some localized areas of backfill underneath and adjacent to the overhang portion of the SWPS had not been sufficiently compacted.

Although no unusual settlement has thus far developed, the Applicant undertook an extensive program of monitoring, analysis, crack mapping, and underpinning. The underpinning was to consist of a continuous perimeter reinforced concrete wall beneath the north end of the SWPS, which would form a box structure beneath the overhang, connected to the sides of the lower portion of the structure, and extending from the upper foundation slab to undisturbed glacial till. Construction of the underpinning made it necessary to lower the ground water table temporarily, through dewatering.

Stamiris Contention 4.C(b) claimed that there had been inadequate evaluation of dewatering effects, differential soil settlement and seismic effects for the SWPS. All aspects of this contention were extensively addressed before this Board. Although borings had shown the presence of some inadequately compacted fill under the overhang portion of this building, measurement of differential settlement indicated that the building was initially stable. However, a survey of cracks led to a disagreement between the Staff and the Applicant as to whether the cracks were incidental to normal shrinkage of concrete or indicative of unacceptable stresses. CPC's decision to install underpinning resting on the underlying glacial till made this disagreement immaterial: the Staff agreed that, with technically acceptable design and construction of the underpinning, together with the proposed crack monitoring and repair program, the cause of the cracking need not be definitively established.

Our findings of fact discuss all aspects of the testimonial record, including a description of the SWPS, the results of borings and surveys of cracks, the CPC-Staff disagreement about crack interpretation, design of the underpinning, effects of ground water levels as affected by dewatering, monitoring arrangements (including acceptance criteria, alert and action levels, and actions to be taken at each level) and the status of a nearby retaining wall. Although the underpinning was designed to meet conditions equal to or exceeding the SSE as determined by the SSRS

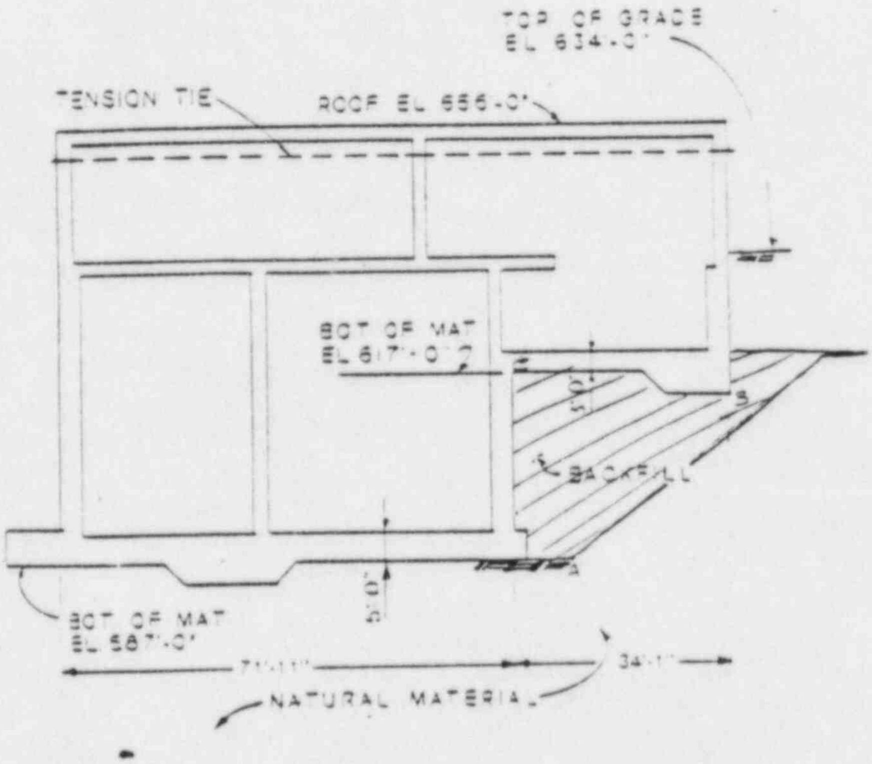


FIGURE 5. Typical section of service water pump structure (looking west) (from Applicant's Exh. 28 (corrections from the testimony)).

methodology, the basic SWPS structure was designed under the older DBE requirements and would be part of a project-wide seismic margin review were construction of Midland to be resumed under the existing construction permits. With underpinning in place, the entire SWPS structure would be founded on undisturbed till. As a result, soil liquefaction and seismic shakedown would not be factors in the SWPS' seismic response. In reaching our findings, we have taken into account proposed findings submitted by CPC and the Staff, which differ essentially only with respect to the sources of cracks. Ms. Stamiris submitted no proposed findings with respect to the design of the SWPS or the remedial measures applicable thereto.

We note that the seismic model which was to be utilized for the seismic margin review of the SWPS appears to be subject to the same design deficiency as was the model for the auxiliary building. *See* discussion, *supra* pp. 70-71. Our reasonable assurance findings with respect to the SWPS are therefore qualified to the extent that they apply only to the *nominal* values for the soil spring constant (or dynamic soil modulus).

Although the Staff initially had concerns similar to those expressed in Stamiris Contention 4.C(b), and in fact at one time supported that contention, as of the close of the record it was satisfied with CPC's remedial measures. With the exception of the design deficiency in the seismic model discussed above, the Board agrees and concludes that the Applicant has now adequately taken into account various dynamic responses in design of remedial soils measures for the SWPS. If completed as designed, the underpinning would provide an adequate and stable foundation for the overhang portion of the SWPS and would not adversely affect a nearby Seismic Category I retaining wall. These conclusions are subject to the outcome of a seismic margin review, including resolution of the design deficiency discussed above. The Board endorses monitoring arrangements agreed to by CPC and the Staff as well as arrangements for keeping the Staff well informed of the results of such monitoring.

VI. BORATED WATER STORAGE TANKS (Findings 168-195)

Two large borated water storage tanks (BWSTs), located to the north of the reactor and auxiliary buildings, were to have supplied borated water to the emergency core cooling system (and the reactor building spray system) during the injection phase of a loss-of-coolant accident. Because this function is necessary to safe emergency shutdown, the tanks are Seismic Category I structures. The foundations of the tanks were constructed between July 1978 and January 1979; erection of the metal tanks was completed by December 1979.

Each tank has a reinforced concrete ring foundation with an integral valve pit which projects like the handle of a pan outside the perimeter of the ring. (The valve pits serve to provide access to the piping connections to the BWSTs and house valves for the fill and drain lines.) Most of the weight of the contained water was to be transferred through the flexible tank bottom to compacted granular backfill inside the ring. Lateral pressure developed from this load in the interior backfill is resisted by the ring foundation wall. The ring foundation also carries the weight of the metal tank and of some of the contained water. The area of vertical loading includes the ring foundation wall footing, the backfill within it and the projecting valve pits. Both tanks are supported by plant fill about 25 feet thick that was placed over competent natural soils. The design originally called for other small tanks to be mounted on the projecting valve pits, but their location was changed. The foundation design was not changed as a result of relocation of the tanks.

Beginning in October of 1980, the Applicant conducted a proof load test by filling both tanks with water and monitoring movements of the foundations by means of repeated surveys. Differential settlement of the ring foundation and between the ring foundation and the valve pits occurred and was initially reported to the NRC, pursuant to 10 C.F.R. § 50.55(e), on January 22, 1981. Structural analyses conducted by the Applicant indicated that the allowable moment capacity for the dead load and the differential settlement condition was exceeded at several locations in the foundation structure. Examination revealed cracking in the foundations of both tanks at the areas of highest calculated stresses — the junction of the ring wall and the valve pits.

Essentially what occurred during the load test was that the more heavily loaded areas within the ring walls settled more than the lightly loaded valve pits. Because they extended beyond the ring walls, the valve pits induced bending moments that exceeded the capacity of the design. This condition caused cracking at the junction of the valve pits with the ring walls and out-of-plane distortions around the perimeter of the ring walls. The bending moments had not been considered in the original design. Furthermore, differential settlement of the foundations was not the same at both tanks. The greater differential settlement of tank 1 than of tank 2 is mainly attributable to lateral variation in the properties of the backfill supporting tank 1.

Analyses of BWST 1 showed that, although it had been stressed beyond normal operating stress limits in two respects (a single point of attachment of the tank to the foundation, and local tank wall compressive stresses), the tank had not undergone damaging stress resulting from the effects of the nonuniform support arising from differential settlement. (Since BWST 2 underwent lesser differential settlement, the analyses for BWST 1 were sufficient for evaluating both tanks.) With regard to the two exceptions cited, the stress conditions were within those allowed for emergency (short-term) conditions, and a considerable margin of safety was calculated to exist for buckling as a result of the local tank wall compressive stresses. Visual inspection of the tanks in the loaded condition verified that no buckling was present, and subsequent dye penetrant examination of the overstressed tank attachment point verified that no cracking was present.

The proposed remedial actions for the BWSTs involved (1) surcharging the valve pits and adjacent areas with sand (later removed) to compress the supporting soils and remove some of the deflection due to differential settlement; (2) constructing a new ring beam around the existing ring wall of each BWST, designed with sufficient capacity to withstand all future loads, and (3) releveling of tank 1. Also, existing cracks wider than 0.01 inch were pressure-grouted with epoxy, and monitoring programs for cracks in the new ring beams and for foundation settlement were proposed.

The new ring beams will rest on the upper surface of the existing ring wall footings, and shear connections will transfer shear force from the existing walls to the beams. New connections will be constructed to and through the valve pits. In the design of the new beam no credit was to be taken for any strength in the existing walls, although their stiffness was included in the design evaluations. Future settlement predictions used in the design of the new beams came from extrapolating settlement versus log-time curves for all the settlement markers, the settlement values being those recorded during the load test when the tanks were full.

The Applicant's consultants evaluated the settlement predictions and confirmed the adequacy of the static and dynamic bearing capacity calculations as well as the long- and short-term soil stiffness moduli for use in the seismic modeling of the BWSTs. The metal tanks were similarly reevaluated for their ability to withstand the predicted future differential settlement loads and seismic loads. The seismic evaluations and reevaluations were based on the 1.5 x DBE (FSAR SSE) response spectra which conservatively envelope the SSRS derived for the Midland site (and which we have found to be acceptable, *see supra* p. 69).

Plant fill soils beneath the BWST foundations were not found to be susceptible to soil liquefaction or to seismic shakedown. Settlement due to dewatering loads beneath the BWSTs was minimal and would be implicitly included in the settlement calculations. While no commitment to dewater the plant fill beneath the BWSTs was made, nor was it necessary, some dewatering would occur as a consequence of dewatering requirements for the plant fill beneath the RBA and DGB.

The Staff reviewed and evaluated the Applicant's assessment of the integrity of the BWSTs following the load test, and the proposed remedial measures and monitoring programs. With the exception noted by the Staff regarding the unresolved technical specification for future settlement monitoring (Staff FOF, ¶ 290, at 30), the Staff agrees that the Applicant has now adequately evaluated and analyzed the dewatering, differential soil settlement and seismic effects in its proposed remedial actions for the BWSTs. The adequacy of such evaluations and analyses had been questioned by Ms. Stamiris' Contention 4.C(c). By way of indicating that this contention was well founded when submitted, however, the Staff notes that "the concerns expressed by Ms. Stamiris in this and other contentions are similar to the concerns that caused the Staff to issue the [Modification] Order." Staff FOF, ¶ 292, at 30. We agree.

The Staff and Applicant disagree as to the cause, or the principal cause, of the differential settlement of the BWSTs. As in the case of the overhanging portions of the auxiliary building and SWPS, the effects of differential settlement are primarily what the remedial measures are intended to address, although different measures were to be taken in the different cases. The effectiveness of the remedial measures is not dependent on the cause of the differential settlement. Thus we need not dwell on that cause.

We note, however, that in the case of differential settlement of the BWSTs, the Applicant has taken the unusual position of asserting that the cause was its own initial design error(s); i.e., the valve pits' projection well beyond the perimeter of the ring wall foundation, the removal of the small tanks that would have added some additional bearing pressure to the valve pits, and the failure to include the effects of the resultant bending moments induced by the valve pits when calculating the stresses in the original design. On the other hand, the Staff holds that the primary cause of differential settlement of the BWSTs was inadequately compacted fill. The Staff witnesses pointed to 1.1 inches of total settlement of a BWST foundation marker even before the tanks were filled (Finding 176). The Staff also referenced the Applicant's witness' nonresponsive answers to Board questions on the amount of total settlement (Staff FOF, ¶ 277, at 27-28). The Board notes, in this connection, the "less stiff" (i.e., softer) soil under part of tank 1 which led to increased differential settlement and required releveling of that tank.

Dr. Kennedy, another witness for the Applicant, provided what we regard as the most balanced — and most persuasive — explanation of the BWST cracks. He believed that there were three causes of cracking in the BWST foundation walls: first, the soft soil under the west side of tank 1; second, the light loading and projecting geometry of the valve pits; and third, under-reinforcing of the ring wall — i.e., had sufficient reinforcing steel been used to produce a more rigid structure, the load would have been spread to include the area beneath the valve pits without cracking.

We can see that the differential settlement was caused by the overall settlement of the soil. Had there been no settlement, as if the BWSTs were founded on rock, there would have been no differential settlement. Alternatively, had the design included reinforcing steel sufficient to resist totally the bending moment, there would have been no failure (but possibly some tilting) during settlement. Thus we see the admitted presence of soil beneath tank 1 that was soft enough to contribute to the additional differential settlement of that tank as indicating nonuniformity of soil compaction.

This situation is not unlike the question of "deficient design" in connection with the stepped foundations of portions of the auxiliary building and SWPS: had either the supporting backfill not settled, or had the design of the auxiliary building included the "cantilever" stresses and the design of the BWSTs the bending moment stresses, they would have been adequate. Our discussion here, where design deficiency is admitted, amplifies the reasons for our recommending Staff review and study of the generic requirements for, or generic acceptability of, the future use of such configurations on safety-related structures. *See supra* pp. 39, 93-94, for our recommendation stemming from the design of portions of the auxiliary building and SWPS.

VII. DIESEL FUEL OIL TANKS
(Findings 196-203)

The design of the diesel fuel oil tanks became an issue in this proceeding because of uncertainties resulting from the presence of improperly compacted fill, as set forth in Stamiris Contention 4.C(d) and Warren Contention 2.B(2). Those contentions questioned whether the fuel oil tanks had been adequately evaluated with respect to such matters as the effects of dewatering, differential soil settlement, and seismic effects (including liquefaction). All aspects of this issue were considered thoroughly by both CPC and Staff witnesses. The hearing record and proposed findings of the Applicant and Staff indicate no areas of disagreement between them, as of the time the record was closed on the design issue. Ms. Stamiris submitted no proposed findings with respect to the design aspects of the fuel oil tanks. With respect both to the potential for liquefaction under the diesel fuel oil tanks and the stability of soils under those tanks, however, recent developments (see below) preclude our resolving those issues at this time.

The hearing record, as summarized in our findings, indicates that the Applicant undertook a program of measurement, analysis and monitoring to assure that the tanks could perform their intended functions. Among other measures, the tanks were surcharged by being filled with water and monitored for about 8 months. The Applicant also analyzed each of the factors cited in the relevant contentions. The Staff concluded that, subject to an audit and the results of a seismic margin review, the structural concerns expressed by these contentions were (as of the close of the record on these questions) without merit.

However, by copy of a report from CPC to the Staff, dated November 21, 1984, the Board and parties were informed that certain 1977 boring logs purportedly reflecting borings taken in the area of the diesel fuel oil tanks were in fact logs of borings taken elsewhere in the Midland area. In response to a telephone request from the Board, seeking information as to the extent the incorrect boring logs might affect testimony currently in the record, the Applicant by letter dated December 6, 1984, advised that the only technical issue potentially affected is the liquefaction of soils below the diesel fuel oil tanks. It further advised that its analyses did utilize at least one of the erroneous logs; that such analyses had been presented to the Staff for licensing review; and that, as a result, the CPC analysis of the liquefaction potential of soils beneath the diesel fuel oil tanks is inconclusive. By letters dated December 21, 1984, and December 24, 1984, the NRC Staff and Ms. Stamiris agreed that we should issue no decision on the liquefaction question, but they went further. The Staff indicated that it had also used the subsurface information from the erroneous boring logs "to assess the compacted density of the plant fill and to evaluate the adequacy of the foundation soils in the diesel fuel oil tank area" and to "assist in accepting the placement of the concrete foundation pads for the diesel fuel oil tanks at elevation 612 feet." Ms. Stamiris sought an OI investigation and further hearings on facts bearing on the erroneous logs. (*See supra* pp. 38-39, for our resolution of these requests.)

The Applicant further indicated that, as a result of the project shutdown, it does not at this time plan to perform the additional analyses or obtain additional field information to close out this issue. The Staff has advised that it has not received the correct boring logs for the diesel fuel oil tank area (Kane Affidavit, dated December 21, 1984, ¶ 3, at 4). Nor has this Board. Given the state of the record, this issue remains open. We are thus making no findings or conclusions at this time on either the liquefaction potential of soils beneath the diesel fuel oil tanks or the foundation stability of those soils. Furthermore, because of the significance of these "open items" to our evaluation of diesel fuel oil tank design issues, we also are not reaching any "reasonable assurance" conclusions with respect to those issues, or any final rulings on Stamiris Contention 4.C(d) or, insofar as it relates to liquefaction under the diesel fuel oil tanks, Warren Contention 2.B(2).

VIII. UNDERGROUND PIPING (Findings 204-292)

Underground piping is among the items which were covered by the Modification Order. Two of the contentions of Ms. Stamiris, and one of those of Ms. Warren (which the parties addressed[39]), raised questions concerning the technical adequacy of such piping, motivated particularly by the excessive settlement of some of that piping. These contentions questioned whether CPC's analyses of piping had adequately taken into account such matters as the effects of the DGB surcharge, dewatering effects, and differential settlement.

In our findings, we describe in detail the various types of underground piping which were installed (or planned to be installed) at Midland. There are two general categories: Seismic Category I (which must be designed to withstand earthquake motions and also are subject to QA requirements) and Nonseismic Category I. The first category of piping was reviewed to assure that the pipes would perform their intended safety functions throughout the plant's projected service life. The second category was reviewed to the extent necessary to assure that postulated failures would not have an adverse impact on nearby Seismic Category I structures or piping.

The concerns with respect to underground piping reflect the inadequate compaction of plant fill supporting that piping, resulting in excessive and in some cases differential settlement of the piping. All of the underground Seismic Category I pipelines (of which there are five types) rest on compacted backfill material. Such piping was discovered to be located from 6 to 21 inches below originally intended elevations (4

[39] *See infra* note 41.

to 19 inches if credit is taken for placement tolerances), with the majority in the range of 9-11 inches.

At the time the Intervenors submitted their contentions on underground piping, it is apparent that insufficient analyses of underground piping had been performed to provide a basis for a reasonable assurance finding concerning such piping. Indeed, during the first hearing session on piping, there were major unresolved questions between the Applicant and Staff on that subject (*see, e.g.*, Chen/Hood, ff. Tr. 7762; Tr. 7763-77 (Kane, Hood, Chen)), leading us to remark that we were being offered little more than a progress report on the resolution of as-yet open questions (Tr. 7777-78).

The Applicant and Staff subsequently resolved their differences. As is reflected in our findings, there have been detailed and extensive analyses performed of all of the underground piping, and corrective actions taken or proposed where required. Criteria for evaluation were developed by the Applicant and reviewed by the Staff. Corrective actions for the service water system (SWS) piping included replacement, rebedding and reinstallation, as well as extensive monitoring. For the borated water storage system piping, the corrective actions included partial recentering and rebedding, and monitoring. All of the Seismic Category I piping was analyzed for seismic effects and was subject to re-review as part of a seismic margin review. Finally, the Applicant and Staff agreed upon a number of technical specifications which would govern underground piping.

One subissue bearing upon underground piping was its susceptibility to corrosion. This is the major facet of the technical aspects of underground piping as to which Ms. Stamiris filed proposed findings. The potential corrosion of underground piping was not a part of any contention. However, during cross-examination on one of Ms. Stamiris' documents which dealt with other aspects of "soils deficiencies," as well as corrosion of the piping (Stamiris Exh. 35), it came to light that corrosive pitting had been discovered in two areas of underground stainless steel piping. The Board asked the Staff to furnish a witness who could address the corrosion of underground piping (Tr. 7835-36, 7863, 7914-16). The Staff responded by presenting Dr. John R. Weeks, a Senior Metallurgist who has been employed at Brookhaven National Laboratory since 1953.

The Board wishes to take this opportunity to give credit to the knowledgeability and forthrightness of Dr. Weeks. As detailed in our findings, we believe that Dr. Weeks has satisfactorily addressed and resolved the various outstanding open questions concerning the corrosion of underground piping. We also appreciate the Staff's efforts in obtaining Dr. Weeks as its witness.

One particular question which Dr. Weeks addressed warrants further comment in light of challenges to Dr. Weeks' opinion advanced by Ms. Stamiris in her proposed findings (Stamiris FOF, ¶¶ "23-27," at 8-10). Dr. Weeks expressed the opinion that the corrosion in stainless steel piping was probably caused by stray welding currents. In doing so, he was reaching the same conclusion that was reached in a 1981 study by Bechtel Group, Inc., the Applicant's consultant. Ms. Stamiris stressed that this conclusion varied from that of an earlier, 1979 study by Bechtel National, Inc., which had not been able to determine the cause of the pitting but had noted the lack of "known electrical sources" in the area of the corrosion. Dr. Weeks explained why he thought the second study was more likely correct — in particular because of the discovery of additional information concerning the welding procedures utilized on the site, and the contribution to the second study of a project engineer expert in corrosion matters with whom Dr. Weeks was familiar (Tr. 9180). He also explained how electrical sources could have caused the corrosion examined in the first report. Most important, however, Dr. Weeks reached his conclusion independently, after considering a number of pertinent considerations which he explicitly outlined. We have no hesitation in accepting Dr. Weeks' conclusions on this question, and in declining to adopt Ms. Stamiris' proposed findings which were premised on the information presented in the first report on the corrosion question. *See infra* Findings 279-280.

Based on the entire record on underground piping, we are in general agreement with the solutions to piping questions which, during the course of the hearings, were worked out between the Applicant and Staff. In addition, we are adding the following supplemental technical specifications or conditions (to take effect if the plant were to be operated or construction resumed):

1. If further placement or replacement of underground Seismic Category I piping were carried out, the Applicant must prepare as-built pipe profiles to verify the post-installation location of the pipes (Finding 210).
2. Based on the acceptance criterion of not more than 3 inches of additional settlement to occur at any pipe location, a technical specification should include alert and action limits. The alert limit shall require that, where settlement at any monitoring station reaches or exceeds 75% of the 3-inch acceptance criterion, the NRC Staff shall be notified (Findings 213, 260).
3. All Seismic Category I underground piping is to be subject to a seismic margin review (Findings 240, 244, 248, 250, 252).

4. An adequate monitoring program for strain gages must be instituted, extending throughout plant life and requiring repair or replacement of the gages, as necessary or appropriate (Findings 257, 263). The Staff should determine the monitoring frequency for the period beyond the first 5 years of monitoring.
5. There must be a pipe monitoring schedule for the period between the commencement of monitoring and the commencement of unit operation, at a frequency to be agreed upon by the Applicant and Staff (Finding 263).
6. The Staff shall have the authority to impose additional monitoring requirements to the extent necessitated by an extended period of time between the startup of Units 2 and 1, respectively (Finding 263).
7. There shall be annual rattlespace monitoring throughout plant life, subject to modification after 5 years if requested by the Applicant and approved by the Staff (under normal procedures for technical specification changes) (Finding 264).
8. To the extent that excavation of 36-inch pipes were yet to take place, the condition of the pipe wrappings should be checked (Finding 271).
9. If the galvanic protection system were to be shut down for an extended period of time, and construction were later resumed, the Staff should carefully consider whether further analysis of corrosion of existing underground piping is required (Finding 281).

In sum, we conclude that the questions concerning underground piping raised by Stamiris Contentions 4.A(4) and 4.C(f), and Warren Contention 3, have been satisfactorily addressed. Subject to the specifications or conditions to which the Staff and Applicant have agreed, supplemented by the further specifications or conditions set forth above, we have reasonable assurance that, so long as corrective actions would be carried out satisfactorily, the Seismic Category I piping would be able to perform its intended functions and would not place undue risk on the public health and safety. We further have reasonable assurance that postulated failures in Nonseismic Category I underground piping, were they to occur, would not adversely affect nearby Seismic Category I structures or piping.

IX. ELECTRICAL DUCT BANKS AND CONDUITS
(Findings 293-305)

The design adequacy of electrical duct banks and conduits became an issue in this proceeding because of uncertainties resulting from the presence of improperly compacted fill, as set forth in Stamiris Contention 4.C(f) and Warren Contention 3. All aspects of this issue were addressed thoroughly by both CPC and Staff witnesses.

The CPC proposed findings on electrical duct banks and conduits provide a comprehensive analysis of the hearing record. We have used these proposed findings as a basis for our own findings. Staff proposed findings were in substantial agreement but provided useful elaborations and clarifications that we have incorporated in our findings. Ms. Stamiris submitted no proposed findings concerning the design aspects of electrical duct banks and conduits.

The hearing record summarized in our findings sets forth the acceptance criteria developed by the Applicant and the detailed analyses that were made of surface loads, effects of construction, crossings of the freeze wall, interfaces with the SWPS and DGB and possible seismic effects. Corrective actions in one area where requirements were not met were developed. The Staff has expressed general agreement with these corrective actions and the rest of the CPC testimony.

The Board concludes that the concerns expressed in the contentions regarding the electrical duct banks and conduits have been adequately addressed. The Board also finds reasonable assurance that the duct banks and conduits would be capable of performing their intended safety function over the projected lifetime of the plant, subject to satisfactory completion of remedial work north of the SWPS and the satisfactory outcome of a seismic margin review (*see infra* Findings 301, 302 and 305).

X. SLOPE STABILITY OF BAFFLE AND PERIMETER DIKES
(Findings 306-318)

Stamiris Contention 4.B questions, *inter alia*, the slope stability of the cooling pond dikes, on the ground that the dikes were built with the same soils and procedures as was the soils foundation for the DGB. The issue was addressed fully by both CPC and Staff witnesses. It involves a safety concern of considerable importance because of possible adverse impacts on the emergency cooling water reservoir should dike stability suffer from the presence of insufficiently compacted soils similar to those present elsewhere on the Midland site. *See infra* Findings 306-309.

In response to a series of questions posed by the Staff and its consultant, the Army Corps of Engineers, CPC conducted a thorough study, including extensive borings by Woodward Cycle Consultants (at locations selected by the Corps of Engineers) and an analysis by Dr. Alfred J. Hendron of the University of Illinois of the shear strength of the dike materials. Based on the study and the analysis, the Staff concluded that the fill material placed in the baffle and perimeter dikes exceeds the design parameters and that the slopes of the dikes would remain stable under static loading conditions (*infra* Findings 310-312).

Dr. Hendron also analyzed dynamic conditions due to a rapid drawdown of pond water level associated with possible dike failure. Even using a very conservative method accepted by the Army Corps of Engineers, factors of safety of 1.34 for critical portions of the baffle dike and 1.50 for critical portions of the perimeter dike for such an event were obtained. The Staff agreed that this was adequate. Indeed, the Corps of Engineers considered 1.0 as the minimum factor of safety for this case. *See infra* Findings 313-315.

The Army Corps of Engineers initially had concern, based on preliminary hydrologic information, that a probable maximum flood (PMF) could breach the perimeter dike and cause erosion damage. PMF questions are not related directly to the shear strength and properties of dike materials and hence were peripheral to the contention under review. Nonetheless, these questions were extensively addressed on the record. After further study, the Staff and the Corps are now satisfied that the potential for dike overtopping during a PMF is small and any overtopping that might occur would not affect the safe operation of the plant. To preclude possible dike damage by erosion, the Staff would require a suitable dike inspection and maintenance program. *See infra* Finding 316. We concur in that requirement.

Dr. Hendron also analyzed dike stability under seismic loadings, using an approach that was accepted by the Staff. Based on conservative assumptions, he obtained yield accelerations for the critical sections of the dikes that were far larger than the 0.19*g* value which, in itself, was greater than that required at Midland. He also testified that soil liquefaction under the dikes will not be a problem. *See infra* Finding 317.

Based on the technical record summarized in our findings, we conclude that the dikes would be stable under all anticipated static and dynamic loads. Thus, contrary to Stamiris Contention 4.B (with respect to which Ms. Stamiris filed no proposed findings), we conclude that there is reasonable assurance that critical slopes of the baffle and perimeter dikes are stable and would not adversely affect safe operation of the Midland Plant, should it be finished and operated. This conclusion assumes the applicability of the inspection and maintenance program proposed by the Staff.

XI. CONCLUSION

A. Technical Issues

In this Decision, we have reviewed only the programmatic aspects of remedial soils measures or "fixes," to the extent we believe that the record with respect to any particular remedial activity is adequate to warrant a ruling on that activity. In general, and subject to certain technical specifications or conditions and the resolution of certain unresolved technical issues, we have found those programs which we have reviewed to be adequate. If construction were to be resumed under the outstanding construction permits, those programs could continue to be undertaken, subject to the controls authorized by LBP-82-35 and the eventual resolution of the various QA/QC management attitude issues and the particular technical issues which remain unresolved. Verification efforts relative to as-built structures, along the lines of those which have been required by the Staff, would also have to be carried out or completed. (We note that further construction may well be subject to additional conditions imposed by the Staff.)

In reaching these conclusions, we have reviewed with great care the entire record of this proceeding dealing with the issues on which we are ruling, including the proposed findings of fact and conclusions of law submitted by CPC, Ms. Stamiris, and the NRC Staff. Our Opinion is based upon, and incorporates, the Findings of Fact and Conclusions of Law which follow. Any proposed findings or conclusions on remedial soils issues submitted by the parties which are not incorporated directly or inferentially in this Partial Initial Decision are rejected as being unsupportable in law or in fact or as being unnecessary to the rendering of our Decision.

B. General Observations

It is somewhat ironic that, for a project which apparently is being halted for financial reasons, many of the extraordinary costs which have attended this project since its inception and undoubtedly contributed to its likely demise are costs which could easily have been — and should have been — avoided. As a Staff witness observed,

> In 1975, 1976, 1977, in my best estimation, one 30-thousand-dollar-a-year geotechnical engineer would have prevented each and every one of these [soils settlement] problems on site.

Tr. 2444 (Gallagher).

Nor would the employment of such a geotechnical engineer have been an unusual step to have been followed. In fact, CPC admitted that it had made a commitment to NRC to have such an engineer on site at all times when soils were being compacted. Stamiris Exh. 3, Attach. 7 (I&E Rept. 78-20), at 24-25; Modification Order, Appendix, Allegation 2.b(2); and CPC's Answer to Notice of Hearing, dated April 16, 1980, Appendix at 4, Allegation 2.b(2). Such a requirement was in effect throughout the entire history of the project (Tr. 1834-35 (Gallagher)). For that reason, we can only reasonably conclude that the soils problems were to a significant extent the product of QA/QC implementation deficiencies for which both CPC and its contractor, Bechtel Corp., must assume responsibility. The soils problems have been a prime ingredient in the project's delay.

Although the soils problems were perhaps the most visible of the QA/QC implementation problems which have surfaced, we must observe that such implementation problems have been endemic to this project, arising even prior to the award of construction permits. *See* ALAB-106, 6 AEC 182 (1973). QA/QC implementation problems continued to surface prior to the time frame in which the soils problems arose. *See, e.g.*, ALAB-283, 2 NRC 11 (1975), *clarified*, ALAB-315, 3 NRC 101 (1976). Following the Modification Order, and despite extensive corrective efforts, problems kept recurring. For example, when CPC (through Bechtel) attempted to cure the lack of a geotechnical engineer (mentioned above), it first hired a qualified engineer but thereafter replaced him with an individual whom the Staff judged to be unqualified for his position. Gallagher, ff. Tr. 1754, Attachment 4 (Appendix B, Notice of Deviation; I&E Rept. 81-01, at 10); Tr. 1834-37 (Gallagher); Tr. 1321, 1325-26 (Keeley). Also, as pointed out in LBP-82-35, various incidents such as improperly drilling into buried duct banks continued to recur. And, in the words of the Staff, a "significant breakdown" in implementation of the QA program with respect to the DGB later surfaced, resulting in numerous nonconforming conditions (Keppler, ff. Tr. 15,113, at 4 and Attachments 3 and 4); Tr. 15,131-32 (Keppler)). *See supra* p. 85. "[C]learly there has been a series of recurrences of quality assurance lapses at the site which should not have taken place" (Tr. 15,116 (Keppler)).

The controls imposed by LBP-82-35, together with the other extensive review efforts upon which the Staff insisted, were intended to assure that further soils-related construction activities would be carried out satisfactorily. Although we are not now ruling on whether these measures were successful, we do observe that, on the basis of recent I&E inspection reports which have been transmitted to us and the parties, covering periods prior to the shutdown of construction, there have appeared to be fewer violations of regulatory requirements than in the past. (Since the various reports are not part of the record, these observations should in no event be regarded as final.) We also must observe that considerable hearing time was devoted to alleged violations of the requirements imposed by LBP-82-35. Although we are not now resolving those issues, we note that, as a result of its investigation, the Staff required CPC to have a third-party "management appraisal" (which, insofar as we are aware, has not been completed). 49 Fed. Reg. 2562 (Jan. 20, 1984). By copy of a letter from NRC (Region III) to CPC, dated November 13, 1984, this Board and the parties were advised that NRC is requiring completion of this management appraisal as a predicate for resumption of construction.

The various controls imposed on construction were designed to assure the adequacy of construction but not necessarily to correct the root causes of the QA/QC implementation deficiencies. Indeed, the Staff was unable to discern exactly what those root causes were. Tr. 15,122, 15,163, 15,178, 15,182, 15,196 (Keppler). The QA/QC implementation difficulties were often attributed by both the Staff and Applicant to a lack of "attention to detail." Tr. 15,125 (Keppler); Tr. 14,731 (Landsman); Tr. 1199 (Keeley). Taking that into account, our own general observation would attribute the root cause of the difficulty to the general managerial attitude of those in control of the project — an attitude which failed to appreciate and stress the importance of taking all of the steps necessary to build quality into the project. Although the latter goal was often enunciated (*see, e.g.*, J. Cook, ff. Tr. 1693, at 22), there appeared to be a number of occasions when steps necessary to achieve that goal were bypassed or ignored (Tr. 15,124 (Keppler)).

That general attitude, in our view, contributed to CPC's attempt to blame others for its own deficiencies. In that regard, we must express our strong disagreement with (and disapproval of) the statements of CPC management officials (in particular, Mr. Stephen H. Howell, a CPC Executive Vice-President) made around 1980 to the press or Congress, to the effect that, were it not for the activities of Intervenors and/or the NRC Staff, the facility would long ago have been built and operating. Tr. 1723-24 (J. Cook); Tr. 2859-60, 20,988-95, 21,076, 21,083 (Howell);

Stamiris Exh. 118; *see also* Tr. 15,135-38 (Keppler). Mr. Howell, who had made the statement with respect to Intervenors, admitted that he had not examined, and did not know, how much time (if any) the conduct of hearings had delayed the plant (Tr. 21,092, 21,082). Indeed, Mr. Howell acknowledged that construction at Midland had not been halted for even 1 day because of the Intervenors (Tr. 21,103). The basis for the statement was a comparison between the licensing time for Midland and a statistical average of the times for five assertedly uncontested or little-contested facilities initiated in about the same time period (Tr. 21,091, 21,146-47 (Howell)). The comparison had not taken into account the facts at issue in any of the licensing proceedings; Mr. Howell conceded, however, that none of the "comparable plants" had had problems during construction of the magnitude of the soils problems encountered at Midland (Tr. 21,117-19).

Were we to resolve the QA/QC management attitude issues of this proceeding, we would regard CPC management's efforts to blame the Intervenors or the Staff for project delays as a reflection of poor managerial attitude. On the other hand, we view Mr. Howell's renunciation of further attempts to blame others for CPC's shortcomings (Tr. 21,087, 21,146-47) as a positive indication. (We express no opinion here as to what the effect of these preliminary findings would be on an eventual evaluation of CPC's managerial attitude.)

For our part, we view the contribution of the Intervenors in this proceeding as positive — particularly that of Ms. Stamiris, who devoted the greatest effort among the Intervenors to the resolution of the various soils issues which we have thus far heard. (Most of Ms. Sinclair's many contentions deal with other matters which for the most part have not yet been litigated.) We reiterate that the QA/QC and management attitude issues, including most of Ms. Stamiris' OM contentions, as well as issues raised by the Staff through the Modification Order, were extremely important issues in terms of the facility's licensability. Although we are declining (for reasons previously outlined) to rule on those issues at this time, we wish to commend both Ms. Stamiris and the NRC Staff for their efforts to build an adequate record on these questions.

In that connection, we wish to note that, early in this proceeding, CPC and the NRC Staff stipulated to the effect that (1) prior to issuance of the Modification Order, there were significant QA deficiencies related to soil construction activities under and around safety-related structures and systems; (2) CPC agreed not to contest the Staff's conclusions that the specified deficiencies constituted "a breakdown in quality assurance with respect to soils placement" and an "adequate basis" for the Modification Order; but (3) the QA/QC program then being followed was adequate and NRC had "reasonable assurance" that such program would be "appropriately implemented with respect to future soils construction activities including remedial actions taken as a result of inadequate soil placement." App./Staff Exh. 1, ¶ 3; to the same effect, *see* Keppler, ff. Tr. 1864, at 8-9. Ms. Stamiris never joined in this stipulation; and, although the Board accepted the first two items of the stipulation recited above, we have never accepted the third item, except as a reflection of the then-current views of the Applicant and Staff (*see* Tr. 1172-75).

Through some superior efforts by NRC Staff inspectors (particularly I&E Inspectors Eugene J. Gallagher, Ross B. Landsman, and Ronald N. Gardner, and Resident Inspector Ronald J. Cook), and through the persistence of Ms. Stamiris, who made certain that these inspectors' views were explored at the hearings, the record was developed to an extent which necessitated our imposition of the interim conditions spelled out in LBP-82-35 (*see* discussion, *supra* p. 35, and *infra* Findings 14-15). Thereafter, following its successful effort to reopen the record, the Staff modified its earlier opinion by conditioning its reasonable assurance of the adequacy of QA/QC implementation upon CPC's adherence to the conditions brought about by LBP-82-35, as well as specified third-party overview efforts and enhanced Staff inspection efforts. Keppler, ff. Tr. 15,111, at 6; Keppler, ff. Tr. 15,114, at 6.[40] We express no opinion at this time whether we currently would have "reasonable assurance" with respect to implementation of the QA/QC program for construction, were the resumption of construction again to be contemplated. But, whatever our conclusion, we believe that the plant, if completed, likely would be measurably safer not only through the superior efforts of the Staff but also as a result of the persistence of Ms. Stamiris.

FINDINGS OF FACT AND CONCLUSIONS OF LAW

Findings of Fact

I. BACKGROUND, JURISDICTION AND PARTIES

1. This Partial Initial Decision treats certain issues in a consolidated proceeding involving (1) the application of Consumers Power Company (CPC or Applicant) for licenses to operate the Midland Plant,

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[40] Both CPC and the Staff favored the continued applicability of those conditions in their most-recent proposed findings. CPC Second Supplemental FOF, ¶ 670; NRC Further Supp. FOF, ¶¶ 11, 15-40.

Units 1 and 2 (OL proceeding), and (2) the Order under 10 C.F.R. § 2.204 for modification of licenses, dated December 6, 1979 (OM proceeding).

2. The Midland Plant consists of two pressurized water nuclear reactors designed by Babcock & Wilcox Co. (B&W), located on the Applicant's site on the south shore of the Tittabawasee River in Midland County, Michigan. The site is adjacent to the Dow Chemical Company's main industrial complex in the city of Midland. 37 Fed. Reg. 28,312 (Dec. 22, 1972). Each unit was designed to operate at a reactor core power level of 2452 megawatts thermal. Unit 2 was scheduled as the first to be completed. As a result of financial problems, CPC currently has suspended construction of both Units and does not contemplate the revival of construction in the near future. Nonetheless, CPC has stated that, despite the project shutdown, it intends "for the time being, to maintain the Construction Permits and Operating License applications for both units" so as to "maintain its options." Letter, CPC to Harold Denton, NRC, dated July 27, 1984 (file 1300, serial 31636); letter, CPC to J.G. Keppler, NRC, dated July 27, 1984, file 0.4.9, serial 31797; also telephone communication to Board from CPC counsel, on July 17, 1984; and letter from CPC counsel to Board and parties dated September 10, 1984.

3. The facility as designed was unique in that the heat generated was proposed to be used not only to produce electrical energy but also to produce steam for the nearby Dow plant. The facility's turbine generators were designed to produce 504 megawatts electrical (MWe) from Unit 1 and 852 MWe from Unit 2. The remaining heat from Unit 1 was planned to produce 460 kg/s (approximately 3.6×10^6 lb/hr) at 1200 kPa gauge (175 psig) and 50 kg/s (approximately 0.4×10^6 lb/hr) at 4100 kPa gauge (600 psig) of process steam for use at the Dow plant. The proposed process steam system was to have been a tertiary system utilizing heat extracted from the secondary steam system of the Midland plant. Staff Exh. 14 (SER, § 1.2, at p. 1-8). However, reflecting delays and cost increases in the project, there developed a contractual dispute between Dow and CPC, and ongoing litigation resulting therefrom, and Dow gave up its plans to utilize the steam which Unit 1 was designed to produce. *Dow Chemical Co. v. Consumers Power Co.*, Circuit Court for Midland County, Michigan, File No. 83-002232-CK-D, complaint initially filed July 14, 1983. Construction of both units has now been suspended as a result of CPC's financial problems.

4. Construction Permits CPPR-81 and CPPR-82, for Units 1 and 2, respectively, were issued by the Atomic Energy Commission on December 15, 1972 (37 Fed. Reg. 28,312 (Dec. 22, 1972)). The initial

part of the application for operating licenses (OL) was filed with the NRC on August 31, 1977, and was formally docketed on November 18, 1977. SER, § 1.1 at 1-1; Appendix A, at A-3; *see also* 43 Fed. Reg. 8870 (March 3, 1978). On May 4, 1978, following the filing by CPC and docketing by NRC of the remainder of the OL application, the NRC published a notice of the "Consideration of Issuance of Facility Operating Licenses; and Opportunity for Hearing." 43 Fed. Reg. 19,304. This notice commenced the first of the proceedings under consideration here.

5. An Atomic Safety and Licensing Board was established to rule on intervention petitions and thereafter to conduct the hearing. 43 Fed. Reg. 25,748 (June 14, 1978); Memorandum for the Record, dated August 16, 1978. The OL Board has been reconstituted several times throughout the proceeding, with the latest change being effective on March 1, 1982. 47 Fed. Reg. 9939 (March 8, 1982).

6. Timely intervention petitions were received from Ms. Mary P. Sinclair, on behalf of the Saginaw Valley Nuclear Study Group (Saginaw), and from the Attorney General of the State of Michigan. Prior to the first prehearing conference, a late-filed petition was received from Mr. Wendell H. Marshall, on behalf of the Mapleton Intervenors. We tentatively admitted Ms. Sinclair and Mr. Marshall as intervenors in their personal capacities (subject to the acceptance of contentions) but denied intervention to Saginaw and to the Mapleton Intervenors (although permitting those groups to file additional information which could qualify them to intervene). The Attorney General of the State of Michigan was admitted as an interested State, pursuant to 10 C.F.R. § 2.715(c). Memorandum and Order dated August 14, 1978 (unpublished); Memorandum and Order dated October 12, 1978 (unpublished). A Notice of Hearing was published on October 18, 1978. 43 Fed Reg. 48,089.

7. The special prehearing conference in the OL proceeding was held on December 16, 1978. Following that conference, we accepted several of Ms. Sinclair's OL contentions and reaffirmed our previous tentative admittance of Ms Sinclair as an intervening party. (Ms. Sinclair did not continue to seek admission of the Saginaw group.) We also accepted one of Mr. Marshall's contentions and admitted him as an Intervenor, although we reaffirmed our earlier ruling denying intervention to the Mapleton Intervenors. Special Prehearing Conference Order, dated February 23, 1979 (unpublished). Subsequently, we accepted a late-filed petition to intervene in the nonsoils-related aspects of the OL proceeding by Ms. Barbara Stamiris (a then-Intervenor in the OM proceeding). Prehearing Conference Order, LBP-82-63, 1 NRC 571, 585-93 (1982).

8. In July 1978, during the placement of concrete on some of the upper elevations of the diesel generator building (DGB), which was then approximately half constructed, the construction survey crews could not close a traverse in surveying (Tr. 2375 (Gallagher)). Upon further investigation, the Applicant determined that the half-constructed DGB had settled both differentially and excessively — indeed, to a greater extent than had been anticipated for the 40-year anticipated life of the plant (Gallagher, ff. Tr. 1754, Attachment 2). *See supra* p. 82. This excessive settlement of the DGB comprised the foundation for one of Ms. Sinclair's OL contentions which we admitted in our February 23, 1979 Special Prehearing Conference Order — as well as for the only contention of Mr. Marshall, which we also admitted in that Order. *See supra* Finding 7. This settlement of the DGB also formed the underlying reason giving rise to the NRC Staff's "Order Modifying Construction Permits," dated December 6, 1979 ("Modification Order" or "OM") (Stamiris Exh. 3, Attachment 15).

9. The Modification Order, issued by the NRC Staff through its Offices of Nuclear Reactor Regulation (NRR) and Inspection and Enforcement (I&E), would have suspended all soils-related and remedial work on the Midland facility until the related safety issues were resolved and a construction permit amendment for the soils remedial work was submitted by CPC and approved by the Staff. It provided that the Applicant or any other person whose interest was affected could request a hearing with respect to all or any part of the Order; and that, if a hearing were requested, the Order would become effective "following the hearing." On December 26, 1979, in accordance with Part V of the Order, CPC stayed the effectiveness of the Modification Order by requesting a hearing. A Notice of Hearing for the OM proceeding was published on March 20, 1980. In the Notice, the NRC designated the same Licensing Board to conduct the OM Hearing as was then designated for the OL proceeding. 45 Fed. Reg. 18,214 (March 20, 1980). This Board, like the OL Board, has been reconstituted several times, most recently on March 1, 1982, with the membership for each of the two Boards remaining the same on each occasion. *See* 47 Fed. Reg. 9939 (March 8, 1982).

10. Both the Modification Order and the Notice of Hearing set forth as issues for adjudication in the OM proceeding (1) whether the facts set forth in Part II of the Order are correct, and (2) whether that Order should be sustained. On April 26, 1980, CPC filed its answer to the Notice of Hearing, responding to the factual allegations set forth in the Modification Order and presenting its position with respect to whether the Modification Order should be sustained.

11. On April 30, 1980, the NRC Staff filed a "Motion for Issuance of Amended Notice of Hearing," which reflected that the earlier notice of opportunity for hearing had never been published in the *Federal Register.* In response to that motion, which was supported by CPC, we published an "Amended Notice of Hearing" on May 28, 1980, providing notice of opportunity for interested persons to participate in the OM proceeding. 45 Fed. Reg. 35,949. Numerous petitions for leave to intervene were timely filed. On July 24, 1980, in our Memorandum and Order Ruling upon Standing to Intervene (unpublished), we determined that nine petitioners had satisfied the "interest" and "aspect" requirements of 10 C.F.R. § 2.714(a)(2). We provided for the later filing of OM contentions and deferred ruling on the letter-petition of Wendell H. Marshall, representative of the Mapleton Intervenors.

12. At a special prehearing conference for the OM proceeding on September 10, 1980, we accepted certain contentions submitted, respectively, by Ms. Barbara Stamiris and Ms. Sharon K. Warren and admitted each as an Intervenor in the OM proceeding (Tr. 398). Thereafter, in our Prehearing Conference Order Ruling on Contentions and on Consolidation of Proceedings, dated October 24, 1980 (unpublished), we ruled on other contentions of Ms. Stamiris and Ms. Warren, respectively, accepting most of them (some in modified form). (Some of Ms. Stamiris' contentions were later amended through her Answer to Applicant's Interrogatories, dated April 20, 1981; and two of her contentions were withdrawn by letter dated June 1, 1981.) We rejected Mr. Marshall's only OM contention and hence denied intervention status in the OM proceeding to him as well as to the Mapleton Intervenors. We also denied intervention to the other petitioners. However, inasmuch as two (similar) OL contentions — one sponsored by Ms. Sinclair and the other by Mr. Marshall — overlapped the scope of contentions properly litigable in the OM proceeding, we granted the Applicant's motion to consolidate the OM proceeding with those issues relating to soil conditions and plant fill materials raised in the OL proceeding. By virtue of that consolidation, we permitted the Intervenors in the OM and OL proceedings, respectively, to participate in both proceedings (with OM Intervenors' rights in the OL proceeding limited to soil settlement questions). As noted earlier, Ms. Stamiris was subsequently admitted as an Intervenor in the nonsoils-related aspects of the OL proceeding (*see supra* Finding 7). We later accepted two additional OM contentions of Ms. Stamiris, arising out of the litigation between Dow and CPC (*see supra* Finding 3). LBP-84-20, 19 NRC 1285 (1984). Ms. Warren, the other OM Intervenor, withdrew from the OM proceeding effective February 16, 1981

(*see* Notice of Withdrawal, dated February 11, 1981), and she never sought intervention status in the OL proceeding.[41]

13. Hearings on soils-related OM-OL issues commenced on July 7, 1981, and have been held during the weeks of July 7 and 13, August 4 and 10, October 13, and December 1 and 14, 1981; February 2 and 16, August 12, November 15 and 22, and December 6, 1982; and February 14, April 27, May 2, June 1, 6 and 27, July 28, August 1, September 20, October 31, November 7 and December 3, 1983. (In addition, hearings on nonsoils-related OL issues were held during the weeks of March 8 and 28, 1983.) All hearing sessions were held in Midland, Michigan, except the hearing on December 3, 1983, which was held in Bethesda, Maryland. Limited appearance statements from members of the public were accepted at several hearing sessions.

14. Following the hearings in October 1981, we had proposed to issue a Partial Initial Decision on soils-related quality assurance (QA)/management attitude issues, prior to the close of the record on technical questions bearing upon the remedial corrective actions associated with the OM issues. Memorandum (Concerning Telephone Conference Call of September 25, 1981 and Applicant's Motion for Partial Decision), dated October 2, 1981 (unpublished). Parties submitted proposed findings of fact and conclusions of law on such QA/management attitude issues.[42] Subsequently, we reopened the record on related QA/management attitude issues; and, after the record was closed on February 19, 1982, parties submitted supplemental proposed findings and conclusions.[43] Thereafter, during the course of our preparation of a decision on those issues, we determined it to be necessary to issue an Order imposing interim conditions on further soils-related construction activities, pending completion of our Partial Initial Decision. We issued that Order on April 30, 1982. Memorandum and Order (Imposing Certain Interim Conditions Pending Issuance of Partial Initial Decision), LBP-82-35, 15 NRC 1060.

15. LBP-82-35, *supra*, required the Applicant, *inter alia*, to obtain explicit prior approval from the NRC Staff (to the extent such approval

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[41] In approving Ms. Warren's withdrawal, we asked the parties, in treating various OM issues, to include the substance of Ms. Warren's contentions (which was necessarily encompassed within the broader OM issues) (Tr. 906-07). Ms. Warren presented an oral limited appearance statement on July 7, 1981 (Tr. 1026).

[42] CPC Proposed Findings of Fact and Conclusions of Law (FOF), dated October 28, 1981; Wendell H. Marshall FOF, dated November 21, 1981; Stamiris Proposed FOF, dated December 11, 1981 (Tr. 5986); NRC Staff FOF, dated December 30, 1981; CPC Responses to Stamiris FOF and Staff FOF, each dated April 26, 1982.

[43] CPC Supplemental Proposed FOF, dated March 15, 1982; Intervenor's [Stamiris] Proposed Supplemental FOF, dated March 29, 1982; Staff Proposed Supplemental FOF, dated March 26, 1982; CPC Responses to Stamiris FOF and Staff FOF, each dated April 26, 1982.

had not already been obtained) before proceeding with further soils-related construction activities (as defined therein). Because LBP-82-35 halted further soils-related construction activities in the absence of NRC Staff approval, the effect of issuing LBP-82-35 was generally to sustain, pending issuance of our Partial Initial Decision on QA/management attitude issues, the requirements of the Modification Order except the requirement for submission and approval of amendments to the applications for construction permits, a procedural step which in our opinion was not necessary to attain the safety goals which we believed should be achieved.[44]

16. The conditions imposed on the Applicant by LBP-82-35 were motivated by QA (including quality control (QC)) considerations. They were intended to remain in effect for what we perceived as a relatively short period prior to the issuance of a Partial Initial Decision on QA/management attitude issues, which would have further reviewed the continuing necessity for such conditions or possibly others. Shortly after the issuance of LBP-82-35, however, events occurred which caused us ultimately to reopen the record on QA matters, at the Staff's request. The reopening is reflected by our Memorandum and Order dated July 7, 1982 (unpublished), in which we announced that we would defer the Partial Initial Decision until we had heard additional testimony on specified issues. The record was not thereafter closed until December 3, 1983 (Tr. 22,691) and proposed findings were subsequently submitted.[45] We are not resolving the QA/management attitude issues in this Decision; and, to the extent that further soils-related construction activities were to be undertaken, the interim conditions which we imposed through LBP-82-35 remain in effect.

17. Subsequent to LBP-82-35, *supra*, we concluded hearings on various technical issues associated with remedial soils activities, and proposed findings were submitted by the Applicant, Ms. Stamiris, and the NRC Staff.[46] Reflecting the probable lack of continuing materiality of the QA/management attitude issues in light of the shutdown of construction on the facility, but similarly reflecting the potential relevance of various programmatic technical findings should facility construction

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[44] Although LBP-82-35 set forth that it was an appealable order, neither the Applicant nor Staff filed any appeal. Ms. Stamiris filed what purported to be an appeal, but the Appeal Board construed the filing as a complaint against the NRC Staff's compliance with and implementation of our order, rather than the order itself. The Appeal Board dismissed Ms. Stamiris' appeal without prejudice to her right to present the same arguments to us, in the first instance. ALAB-684, 16 NRC 162 (1982).

[45] CPC Proposed Second Supplemental FOF on QA Issues, dated January 27, 1984; Stamiris' Second Supplemental FOF on QA and Management Attitude Issues, dated May 11, 1984; NRC Staff Further Supplemental FOF Concerning QA, dated May 25, 1984; CPC's Replies to Ms. Stamiris' and the Staff's FOF, each dated June 22, 1984.

[46] *See supra* note 3.

again be resumed, we have determined to issue this Partial Initial Decision on a number of the technical issues associated with remedial soils activities and encompassed by the foregoing proposed findings. For reasons described in the Opinion section of this Partial Initial Decision (*supra* p. 38), however, we are not at this time ruling on technical questions associated with the DGB and with differential settlement of the control tower relative to the main structure of the auxiliary building. Nor, for reasons set forth *supra* p. 38, are we ruling on certain questions bearing upon (1) the adequacy of soil spring constants, and (2) liquefaction and soils stability relative to the diesel fuel oil tanks. We are here covering various seismic matters (including general seismic standards applicable to the Midland site, standards for the proposed seismic margin review (other than certain aspects of soil spring constants), soil liquefaction (except with respect to the diesel fuel oil tanks), and the effect of dewatering), the structural adequacy of the auxiliary building (except with respect to the differential settlement matters mentioned above), and various issues related to the service water pump structure (SWPS), borated water storage tanks (BWSTs), the diesel fuel oil tanks (except as indicated above), underground piping, electrical duct banks and conduits, and the baffle and perimeter dikes adjacent to the cooling pond.

18. Some of the remedial soils activities discussed in this Decision were commenced prior to the close of the record in these proceedings. With limited exceptions (*see, e.g.*, Tr. 7788a and Tr. 7790), they were subject to the controls imposed by our April 30, 1982 Order (LBP-82-35) or, for certain earlier activities, the voluntary but somewhat narrower commitment of the Applicant in February 1980 not to proceed with further soils remedial actions without NRC Staff review and concurrence. One such earlier approved activity was the underpinning of the auxiliary building and feedwater isolation valve pits. The NRC Staff concurred with the construction of access shafts and a freezewall in preparation for this underpinning on November 24, 1981 (Staff Exh. 5); for activation of the freezewall on February 18, 1982 (Tr. 7838); and by letter dated December 9, 1982, from NRC Region III to CPC, the Staff authorized the commencement on a step-by-step basis of the actual underpinning under the turbine building (Tr. 11,007). Other soils activities were also authorized. During these hearings, we heard testimony from various witnesses on the progress of this work and on various events which have occurred during the course of construction, including actual or potential items of noncompliance. With the shutdown of construction of the facility, we do not at this time plan a thorough evaluation of the Applicant's construction performance, but here we will occasionally rely on

certain data generated by such construction activities, as reflected by the record before us. In this Decision, we are not taking into account the fact that construction of particular structures has commenced (or even been completed) in evaluating the technical adequacy of the Applicant's soils remedial measures.

II. SEISMIC MATTERS

A. Introduction

19. The construction permits for the Midland plant were issued in 1972 (*see supra* Finding 4), after publication of the proposed Appendix A to 10 C.F.R. Part 100, "Seismic and Geologic Siting Criteria for Nuclear Power Plants," but before its issuance and promulgation as a final rule, effective December 13, 1973. 36 Fed. Reg. 22,601 (Nov. 25, 1971); 38 Fed. Reg. 31,279 (Nov. 13, 1973); 10 C.F.R. Part 100, Appendix A. The Commission (AEC) set forth its expectation that, prior to their effective date, the proposed rules be used as guidance. 36 Fed. Reg. 22,601.

20. Appendix A, Part 100,

> describes the nature of investigations [currently] required to obtain the geologic and seismic data necessary to determine site suitability and to provide reasonable assurance that a nuclear power plant can be constructed and operated at a proposed site without undue risk to the health and safety of the public. It describes procedures for determining the quantitative vibratory ground motion design basis at a site due to earthquakes . . . .

10 C.F.R. § 100.10(c)(1).

21. The Design Basis Earthquake (DBE) approved for the Midland site at the CP stage was based on a Modified Mercalli Intensity (MMI) of VI, the size of the largest earthquake within about 150 miles of the plant site. Staff Safety Evaluation ("SER"), CP stage, dated November 12, 1970, at 13, 114, 116. The DBE was not associated with any tectonic province, since the Staff's CP review, which formed the basis for the CP authorization, predated both the issuance of the proposed rule and the effective date of the final 10 C.F.R. Part 100, Appendix A, which required a tectonic province determination. (Insofar as the formulation of a tectonic province was involved, the proposed Appendix A does not appear to have been used as guidance in any portion of the CP review or proceedings. *See supra* note 6.) The ground motions associated with the DBE were represented by a modified Housner design response spectrum anchored at 0.12*g* (where *g* = acceleration due to gravity at the earth's

surface). The Housner spectrum was modified by increasing its levels of response motions by an additional 50% in the frequency range between about 1.6 Hz and 5 Hz (or 0.6- and 0.2-seconds-period range). CP "SER" at 13; Thiruvengadam Affidavit[47] at 2; Kimball, ff. Tr. 4539, at 2; Tr. 6041, 6087 (Kennedy).

22. Following issuance of 10 C.F.R. Part 100, Appendix A, and during the OL review, the Staff had two concerns about the DBE accepted during the CP review. First, the Staff had come to accept the "Central Stable Region" as a tectonic province which would include the Midland site, and which has a controlling earthquake similar to the Anna, Ohio earthquake of March 9, 1937 of intensity MMI = VII-VIII (and a magnitude of m_{blg} = 5.3). Second, the Staff was concerned about the use of a modified Housner response spectrum anchored at 0.12*g* to represent the maximum vibratory ground motion for design purposes. The Staff, in fact, determined that the design response spectrum as used was no longer a conservative representation of the ground motion. SER, § 2.5.2.1, at p. 2-34; Kimball, ff. Tr. 4690, at 2, 4-5.

23. From investigations assertedly performed pursuant to 10 C.F.R. Part 100, Appendix A, the Applicant in 1977 proposed an SSE (as well as an operating basis earthquake (OBE)) based upon designation of the Michigan Basin as a tectonic province separated out of the larger Central Stable Region. Thiruvengadam Affidavit at 3; *see also* FSAR, § 2.5.2.3 (not part of the evidentiary record of this proceeding). (The OBE has not been at issue in these proceedings, and we make no findings concerning its adequacy.) For an SSE, the Applicant proposed an intensity of MMI = VI, representing the intensity of the controlling earthquake in the Michigan Basin, derived from the largest historically recorded earthquake therein. The Applicant further proposed that the SSE ground motions be represented by modified Housner response spectra anchored at 0.12*g*. These characteristics of the SSE proposed in the current version of the FSAR are identical to those of the DBE determined at the CP stage, and are at issue in these proceedings. Thus the term "FSAR spectra" (or spectrum) as used to this point in time, should be read as equivalent to the DBE spectra. Holt Exh. 10,[48] at 2; CP "SER" at 12-13, 116, 124.

24. If the OL application were to be pursued, the FSAR would need to be revised to reflect the SSE and its ground motion characteristics, as determined by the outcome of these proceedings, for purposes of

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[47] Affidavit of Thiru Thiruvengadam, dated March 6, 1981, submitted with Applicant's Motion to Defer Consideration of Seismic Issues Until the Operating Licensing Proceeding, dated March 18, 1981 (*see supra* p. 43); hereafter "Thiruvengadam Affidavit."

[48] *See supra* note 8.

design of the remedial structures and reevaluation of the seismic resistance of existing structures. As set forth *infra* Findings 27, 31 and 79, the Applicant was using (or was to use) a site-specific response spectra (SSRS) approach for these purposes, and we have found use of that approach to be reasonable and conservative. Thus, the DBE spectra served as the seismic design basis for the original safety-related structures, systems and components, but an SSE with SSRS ground motion characteristics would be considered as the seismic design basis in the final design analyses.

25. The Staff did not accept the proposed delineation of the Michigan Basin as a tectonic province and continued to be concerned about the adequacy of the DBE ground motion representations accepted at the CP stage. Tr. 867-68 (Hood); Holt Exh. 3; Thiruvengadam Affidavit at 3; SER, § 2.5.2.1, at p. 2-34, § 2.5.2.3, at p. 2-37.

26. While the December 9, 1979 Modification Order did not specifically address seismic issues, one of its major concerns was "the unresolved safety issue concerning the adequacy of the remedial action to correct the deficiencies in the soil construction under and around safety-related structures and systems . . ." (Modification Order at 4). Seismic design bases for the underpinning work clearly would have been included under the required acceptance criteria necessary for the Staff to evaluate the technical adequacy and proper implementation of the proposed remedial actions (*id.* at 3).

27. The Staff's recommendations of two acceptable methods to be used in resolving the OL concerns about the SSE and seismic design bases for the remedial actions (Findings 22, 25, 26, *supra*) were transmitted to the Applicant in a letter (Tedesco to Cook, October 14, 1980, Holt Exh. 3 ("Tedesco letter")). Both alternatives were based on an SSE for the Midland site similar to the Anna, Ohio earthquake of March 9, 1937, which is the largest historically reported earthquake in the Central Stable Region tectonic province. The first approach would have prescribed use of the *standardized response spectra* of Regulatory Guide 1.60[49] anchored at 0.19g, consistent with an intensity MMI = VII-VIII earthquake. The other acceptable approach, which had been discussed with the Applicant as early as July 1979 (Thiruvengadam Affidavit at 3),

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[49] Both the Staff and Applicant often refer to the Regulatory Guide 1.60 spectra as "site-independent," as if implying that the only distinction between them and site-specific response spectra is found in site conditions. They are more appropriately described as *standardized* response spectra, and are also magnitude-independent, epicentral-distance-independent, and source-characteristic-independent. Their construction also involved normalization of all constituent earthquake records within the ensemble used to a standard value (1.0*g*). Staff Brief at 10-11; Holt, ff. Tr. 4539, at 5-6; Tr. 4585-86 (Holt); Kimball, ff. Tr. 4690, at 8-9. It is the Board's understanding that the Housner spectrum is another, but generally lower, standardized response spectrum. *See* Figure 2, *supra* p. 66.

would have been to develop *site-specific spectra* by enveloping the 84th percentile spectral level of an ensemble of response spectra which were derived from actual, site-and-magnitude-matched accelerograms recorded at epicentral distances of 25 km or less. Site matching would be achieved through close similarity of materials properties beneath accelerograph station sites to materials properties beneath the Midland site. Magnitude matching was specified as equivalent to m_{blg} (central U.S.) $= 5.3 \pm 0.5$. Both approaches are discussed in the Standard Review Plan, §§ 2.5.2 and 3.7.1. Kimball, ff. Tr. 4690, at 5-6, *as corrected* at Tr. 4686; Holt Exh. 3.

28. A category of application of the "new" SSE would have been to the reevaluation of the seismic resistance of already-built structures, which are founded on plant fill and which were to be supported by the remedial work. This category needs to be distinguished because the construction of new foundations (underpinning) beneath fill-supported structures may alter seismic response of those structures to vibratory input motions. (The category results from a combination of the two other applications, i.e., reevaluation of already-built structures, components and systems using current seismic standards, and design of remedial structures or parts of structures, also to current seismic standards.) Thiruvengadam Affidavit at 7; Tr. 846, 857-59 (Statement of M. Miller, Applicant's counsel).

29. The main safety-related structures at the Midland facility are:

(a) containment buildings (founded on natural soils);

(b) auxiliary building:

main structure (located between containment buildings, founded on natural soils); railroad bay (located at north end, founded on plant fill); control tower (located at south end, founded on plant fill); electrical penetration areas (EPAs) (extend east and west from control tower, founded on plant fill);

(c) feedwater isolation valve pits (FIVPs) (structurally isolated, located adjacent to EPAs and containment buildings, founded on plant fill);

(d) service water pump structure (SWPS) (southern part founded on natural soils, northern overhang founded on soil fill);

(e) diesel generator building (DGB) (founded on plant fill);

(f) diesel fuel oil tanks (founded on plant fill);

(g) borated water storage tanks (BWSTs) (founded on plant fill).

Foundation underpinning structures were required to be constructed beneath the control tower and EPAs of the auxiliary building and the overhanging portion of the SWPS; and plant fill beneath the FIVPs was to have been replaced with concrete and compacted granular fill. New ring foundations, structurally attached to the old and to the integral valve pits, were required to be constructed for the BWSTs and tank 1 was to be relevelled. Surcharging with sand fill was employed by the Applicant to compact plant fill beneath the DGB, as well as beneath the valve-pit projections of the BWSTs which caused foundation damage from differential settlement during a preload test. Permanent dewatering of the plant fill was required beneath the railroad bay and the DGB, as well as in the area of a portion of the service water piping, to reduce the potential for liquefaction of the granular foundation soils under SSE loading conditions. SSER # 2, § 2.5.4.1.2, Tables 2.2 and 2.3, § 2.5.4.4.3, at p. 2-34, § 2.5.4.5.5, at 2-43, 2-44.

30. For the reasons set forth in the Opinion section of this Decision (*supra* p. 43), we are here making findings with respect to seismic criteria, including determination of the SSE, ground motions and associated response spectra, and the analysis model for each structure as modified by the remedial actions. We are not making findings at this time on whether the safety-related structures as built (including those with and those without modifications necessitated by the soils remedial actions) conform to the newly determined seismic criteria.

31. The Applicant used the SSRS approach offered in the Tedesco letter as an alternative for characterizing the SSE ground motions but without conceding that the seismic design basis of the Midland plant approved at the construction permit stage is inappropriate or that the Michigan Basin is not a separate tectonic province. Thiruvengadam Affidavit at 4.

32. Departures from the SSRS approach offered in the Tedesco letter that were used, or proposed by the Applicant, in addition to what tectonic province should be used, are the subject of later findings, below. These include such issues as the range of earthquake magnitudes to be employed and the appropriate statistical spectral level to represent the SSRS-derived maximum ground motions, as well as the magnitude of the controlling earthquake in the Central Stable Region tectonic province.

33. Because of the lack of agreement at the time between the Applicant and Staff on a seismic design criterion, the Applicant incorporated a "reasonable margin" over the FSAR SSE (DBE) seismic criteria for design of the remedial "fixes" (Thiruvengadam Affidavit at 6-7). This "margin" was established as 1.5 times the "FSAR design spectra," which was found generally to envelop the SSRS being proposed and committed to by the Applicant for reevaluation of existing structures as

part of the seismic margin review, as well as for design of the remedial "fixes." Tr. 5997-98 (Kennedy).

34. Because the SSRS approach proposed in the Tedesco letter appeared to be a probabilistic methodology (at least in part), the Board directed the Applicant and Staff (and permitted other parties) to file trial briefs discussing the compatibility of the approach with 10 C.F.R. Part 100, Appendix A, should the Applicant elect to use this approach. The Applicant and Staff responded. For reasons expressed in the Opinion section of this Decision (*supra* pp. 46-50), we find that the methodology used by the Applicant and the NRC Staff in developing the SSRS for the Midland site is compatible with 10 C.F.R. Part 100, Appendix A.

35. General elements of investigation for determining the SSE and its representative ground motions, in situations where no capable faults (or similar tectonic structures with which historical earthquake activity can be reasonably correlated) exist within the vicinity of the site, are (1) determination of the tectonic province in which the site is located, (2) determination of the size and ground motions of the controlling earthquake within that tectonic province, (3) determination of the size and ground motions, at the plant site, of earthquakes associated with distant tectonic structures and those associated with adjacent tectonic provinces, and (4) definition of the response spectra corresponding to the maximum vibratory ground accelerations at the various foundation levels of safety-related structures on the plant site. 10 C.F.R. Part 100, Appendix A.

36. The Applicant determined, and the Staff agreed, that, on the basis of extensive investigations by the Applicant, no capable faults, or similar tectonic structures with which earthquake activity can be reasonably correlated, exist in the vicinity of the site that would generate earthquakes whose motions would control seismic design of the Midland plant. Holt, ff. Tr. 4539, at 7; Tr. 4571-72, 4611-14, 4660-61 (Holt); Tr. 4729 (Kimball); SER, § 2.5.3, at 2-41 to 2-44.

B. Tectonic Province and Controlling Earthquake (SSE)

37. The Applicant maintained that the Michigan Basin met the requirements in Appendix A to Part 100 for definition as a tectonic province. It is a very large tectonic structure or "unit" itself (Holt, ff. Tr. 4539, at 11[50]; Tr. 4614 (Holt); *also see* Kimball, ff. Tr. 4690, at 3), dis-

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[50] Mr. Holt in his prepared testimony (ff. Tr. 4539, at 11) incorrectly described the Michigan Basin as being "nearly 200 miles in diameter." It is readily apparent on Holt Exhibit 9 and in his oral testimony (Tr. 4575-76, 4578) that he meant "nearly 200 miles in radius" or "nearly 400 miles in diameter." *See also supra* Figure 1.

tinguishable from the tectonic arches around its southern perimeter on the bases of structural relief, parallel and cross structures on the arches and seismicity differences (Holt Exh. 10; Holt, ff. Tr. 4539, at 11-12; Tr. 4562, 4577 (Holt)). It has a relative consistency of tectonic features within it, namely the northwest-southeast trending anticlines, monoclines, and possible related faults, known mainly in the deep subsurface from petroleum exploration in the State. The controlling earthquake, derived from two historical events in the southern part of the basin, would have an intensity MMI = VI or magnitude m_{blg} = 4.5. Tr. 4598, 4601 (Holt); *see also* FSAR, § 2.5.2.3 (not introduced into evidence).

38. As a result of its evaluation of relative seismic hazard analyses performed by the Applicant, the Staff withdrew from that part of its position expressed in the Tedesco letter that the Central Stable Region, with a controlling earthquake of intensity MMI = VII-VIII (or magnitude m_{blg} = 5.3), was the appropriate tectonic province for evaluating the seismic hazards of the Midland site. This change in position apparently came late in the preparation of the Staff's testimony. The Staff, however, still did not agree that the Michigan Basin, as proposed by the Applicant, was the appropriate tectonic province, but would extend it westward to include Michigan's Upper Peninsula, the northern part of Wisconsin, most of Minnesota, and maybe parts of North Dakota and southern Canada. The Staff's proposed tectonic province would include, as well, all of the Michigan Basin province proposed by the Applicant except for a small corner in southeastern Michigan. (This possible exclusion apparently was based on the north trending zone of small earthquakes and cross structures on the flank of the Findlay Arch that can be seen on Staff Exhibit 5 to extend toward the Michigan Basin from the vicinity of the Anna, Ohio earthquake zone. Tr. 4837 (Kimball) *referring back to* Tr. 4577-80 (Holt)). The effect of extending the tectonic province boundary to Minnesota would be to include a magnitude 5.0 earthquake which occurred there in 1860, and which would represent the controlling earthquake for the province.[31] The corresponding intensity of the controlling earthquake would be MMI = VI-VII, or VII, based on that event. Although the intensity of one or more earthquakes in the Keweenaw Peninsula of northern Michigan may have exceeded MMI = VII, the Staff's expert, Mr. Jeffrey K. Kimball, explained that the events there had anomalously high intensities because of their shallow depths of occurrence. Kimball, ff. Tr. 4690, at 2-5, 11, 20-23; Tr. 4697-98, 4713-14, 4769-83, 4787, 4794, 4837 (Kimball); Tr. 4602 (Holt).

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[31] The Staff also cited the occurrence of a magnitude 4.8 earthquake that occurred in Minnesota in 1975 (Kimball, ff. Tr. 4690, at 21).

39. The Applicant's witness, Mr. Richard J. Holt, was not aware of the change in the Staff's position when he prepared his written testimony prior to the hearings on October 13, 1981, judging from the content of that testimony and oral testimony at the hearing. During cross-examination by Staff counsel, Mr. Holt testified that, after reading the prepared testimony of the NRC witness, Mr. Kimball, he agreed with the use of seismicity as a tool (that the Staff had used in extending the province boundary westward) and he agreed that there have been no historic earthquakes of a magnitude greater than 5.0 in the area of the westward extension proposed by the Staff. While not specifically abandoning his proposed (Michigan Basin) tectonic province for the Midland site, Mr. Holt agreed that the choice of a magnitude 5.0, while "quite conservative," would be appropriate in this case and would correspond to the largest historical earthquake which should be associated with the seismotectonic province in which the Midland site resides. Holt, ff. Tr. 4539, at 11, 19-20; Tr. 4540-41, 4567-70, 4596-97, 4602-03 (Holt).

40. Two maps introduced by the Applicant showed somewhat different boundaries for the proposed Michigan Basin tectonic province, but the amount of disparity between the two representations appears to fall within the degree of acceptable uncertainty or "fuzziness" ascribed to those boundaries. Holt Exh. 9 and Exh. 10, Figure 5; Tr. 4561-65, 4576-80, 4597 (Holt); Tr. 4770, 4779, 4783-84 (Kimball). The larger representation on Holt Exhibit 9 apparently was the one intended by the Applicant to be used. Tr. 4781 (representation by Mr. P.A. Steptoe, Applicant's counsel). The Staff did not introduce map representations of the boundaries of its proposed tectonic province, or give it a name other than "the upper Midwestern U.S." (Tr. 4745, 4783, 4786, 4794 (Kimball)).

41. By reducing the Applicant's two cited map portrayals to a common scale and overlaying them, the Board has provided a single map here for convenience to show the proposed tectonic province boundaries, major tectonic structures, seismic source zones, and Central Stable Region sites used in the relative seismic hazard studies. Figure 1, *supra* p. 56. To this map the Board has added the delineation of what we understand from the verbal descriptions to be the boundaries of the Staff's proposed westward extension of the tectonic province and the area in southeastern Michigan that we would exclude based on the Staff's reservations about its inclusion. For ease in locating the places discussed in the testimony, we have also added a few place names mentioned therein. Tr. 4745-46, 4783, 4837 (referring back to Tr. 4577-80 (Holt)) (Kimball).

42. Both the Applicant and Staff argued (the Applicant more strongly) that the Central Stable Region could, or should, be subdivided. Both pointed out that it was based on the "veneer" of sedimentary rocks[52] deposited over the area about 200-600 million years ago and that it does not represent a region of uniform seismicity, in that the larger earthquakes (magnitude = 5.1-5.3) have occurred in isolated regions which generally show more frequent small earthquakes than other parts of the region. The Applicant's witness believed those larger earthquakes were generally associated with tectonic structures. Holt, ff. Tr. 4539, at 12-13; Tr. 4555-58, 4561-67, 4572, 4601, 4644-47 (Holt); Holt Exh. 10, Figs. 5-6; Kimball, ff. Tr. 4690, at 3-4, Figs. 4-5; Tr. 4717, 4744 (Kimball). The Board notes that these isolated areas of correlative, but not definitely associated structures and magnitude 5.1-5.3 earthquakes arguably could be cited as evidence of the relative consistency of geological structural features needed to characterize a tectonic province, even though they are widely separated.

43. While the Applicant provided geologic and tectonic justifications for its proposed tectonic province to demonstrate its compatibility with the requirements of Appendix A to Part 100 (Findings 37, 39, *supra*), the Staff relied upon its evaluation of the Applicant's probabilistic seismic hazard studies, almost exclusively, to justify its definition of the larger tectonic province. While the Staff's witness indicated that factors other than seismicity should be used in such definitions, e.g., tectonic flux measurements, past strain releases, tectonic structural fabric such as amount of folding or faulting, and consistency of structure and geologic features, he gave no indication that the Staff had, indeed, examined any of those characteristics,[53] only that nothing in the geology "flagged" the region as requiring a larger controlling earthquake than the maximum historic event within it. Furthermore, the Staff has not fully determined what the boundaries for its proposed tectonic province would be. Kimball, ff. Tr. 4539, at 4, 16-21; Tr. 4697-98, 4713-14, 4745, 4769-71, 4779-81, 4783, 4786, 4826-30 (Kimball); Staff Brief at 7.

44. For reasons stated earlier (*supra* p. 58), we reject the view that the agreement between the Applicant and Staff on the appropriate SSE and the representation of its ground motions by the SSRS permits us not to define the proper tectonic province in which the Midland site resides. We view the agreement between the Staff's and Applicant's positions as being material to determination of the SSE and acceptance of the SSRS

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[52] *See* note 57, *infra* p. 133.

[53] These characteristics are paraphrased from § 2.5.2 of the Standard Review Plan (NUREG-0800), which is quoted in the Staff Brief (at 7) as providing criteria for acceptance of a proposed new tectonic province.

rather than to definition of the tectonic province, a point on which they disagreed.

45. The Staff based its almost exclusive reliance on historic seismicity for proposing a new tectonic province on a theory with which the Applicant agreed. That theory held that past earthquake occurrence, or historic seismicity, provides one of the most, or the most, accurate means available for inferring geologic mechanisms causing earthquakes at depths in the earth's crust where earthquakes occur. The next step in the Staff's logic was to equate tectonic (or seismotectonic) provinces with seismic source zones. Kimball, ff. Tr. 4690, at 4, 20; Tr. 4697-98, 4713-14, 4745, 4747-50, 4830 (Kimball); Tr. 4559-61, 4567-68 (Holt).

46. The Board finds that reliance upon historic seismicity as a tool to help establish, or to verify a tectonic province and the size of its controlling earthquake, is consistent with both Staff practice and Appendix A to Part 100.[54] In practice the Staff has relied upon seismicity, at least in part, to subdivide the Central Stable Region farther south into eastern and western parts each with a different level of seismic hazard. Tr. 4807, 4831-32 (Kimball). (We assume that the Staff there considered the other characteristics specified in the Standard Review Plan (Finding 43, including note 53, *supra*) as criteria when making that subdivision.)

47. Reliance upon seismicity to help establish a tectonic province is also consistent with precedent established in the *Seabrook* proceeding. In *Seabrook*, a postulated seismic source zone (the "Boston-Ottawa belt" or trend) was divided into two parts, each with a different level of seismic hazard, but separated by a large tectonic feature (the Green Mountain Anticlinorium) which has been essentially aseismic in historic times, and where "as one moves away from the anticlinorium into either of the two adjacent zones, seismic activity begins to increase." It was not just the aseismic gap, but the correlation of differences in historic seismicity with a tectonic feature that formed the basis for the subdivision. *Public Service Co. of New Hampshire* (Seabrook Station, Units 1 and 2), ALAB-422, 6 NRC 33, 61 (1977).

48. This Board finds that the Staff's own past practice, Appendix A to Part 100, and the teaching of ALAB-422 do not support the definition, or subdivision, of a tectonic province solely on the basis of historic seismicity, even if that seismicity is viewed as somehow indicative of

[54] Appendix A, § V(a) of Part 100 requires that "[t]he design basis for the maximum vibratory ground motion . . . should be determined through evaluation of the seismology, geology, and the seismic and geologic history of the site and the surrounding region." Seismicity studies, whether probabilistic or deterministic in nature, are clearly part of the evaluation of the seismic history of the site and surrounding region.

otherwise poorly known tectonic conditions.[55] To support that theory, much more information about what the earthquakes reveal about tectonic conditions would be needed, other than just earthquake location, frequency of occurrence, and size. The Board was not convinced by the Staff's arguments and the Applicant's support of those arguments that occurrence of historic earthquakes, alone, can provide enough information on subsurface geologic or tectonic conditions to permit definition of a tectonic province based on that premise.[56]

49. An example of apparently inconsistent tectonic conditions within the Staff's proposed tectonic province is revealed by Staff Exhibit 5. On that map, northeast-trending tectonic structures prominently appear in the area of the Keweenaw Peninsula where the anomalously shallow historic earthquakes occurred, as well as in central Minnesota in the general region where we assume that the Staff's proposed controlling earthquake occurred. The northeast trend of tectonic structures in these two areas is orthogonal to the predominantly northwest trend of tectonic structures in the Michigan Basin that were cited by the Applicant as evidence of consistency of tectonic structure in its proposed province (*see supra* Finding 37). The Staff did not address this apparent tectonic inconsistency within its tectonic province that contains both sets of differently oriented tectonic structures, one set of which occurs in a region (the Keweenaw area) with anomalous historic earthquakes. In light of the definition of a tectonic province set forth in Appendix A to Part 100, we believe the Staff should have done so, especially since an uncited Staff discussion in the SER (§ 2.5.3.2.1, at 2-41, 2-42) of Applicant's studies of geology in the Midland region refers to a much subdued set of northeast-trending structures, orthogonal to the predominant trend, in the region. Kimball, ff. Tr. 4690, at 20-21; Tr. 4782-83, 4787 (Kimball).

50. The Staff's witness, Mr. Kimball (Tr. 4746-47, 4789), said that a problem of subdividing just the Michigan Basin from the Central Stable Region was the same as the problem perceived with retaining the Central Stable Region as a tectonic province — i.e., both would be large-

[55] The fact that these studies were probabilistic in nature was not material to our determination here. We simply were not convinced that the Staff had not just drawn lines around a cluster of historic earthquakes and called the area a "seismotectonic province" on that basis.

[56] Although agreeing in principle with the Staff's approach used in defining its proposed tectonic province, Mr. Holt stated elsewhere, "while I do not believe that tectonic provinces should be defined solely on the basis of historical seismicity or a probabilistic analysis of such seismicity, seismicity and analysis of seismicity can be used to test the validity of a defined tectonic province." Holt, ff. Tr. 4539, at 14.

ly based on "surficial Paleozoic geology."[57] However, like the Applicant, he was apparently willing to consider the position of the flank of the Findlay Arch, a feature of the "surficial Paleozoic geology," in the location of his proposed tectonic province boundary (Tr. 4837), and agreed that the Staff has used the Central Stable Region as a tectonic province (Tr. 4786). He also stated that there are some experts who would consider that portion of the Kankakee Arch that has had essentially no historic earthquakes to have a potential for earthquake activity (Tr. 4760). (For location of the Kankakee Arch, *see* Figure 1, *supra* p. 56.)

51. Mr. Kimball (Tr. 4791) also briefly noted that the historic earthquake activity in another basin, the Illinois Basin, which is also located within the Central Stable Region, was inconsistently higher than the historic activity in the Michigan Basin. We would assign little probative value to this argument against use of the Michigan Basin as a tectonic province because we do not know the causes of the earthquakes in either basin and do not assume that the causative tectonic mechanisms of earthquakes should be the same in all basins. Also, the Board notes that the Illinois Basin (*see* Staff Exh. 5) is adjacent to the very active New Madrid seismic zone where tectonic stresses are obviously high.

52. The Board finds that the Central Stable Region can be subdivided in the region surrounding the Midland plant site and that the Applicant has proposed a tectonic province, the Michigan Basin, that appears reasonably to meet the criteria for its establishment as prescribed by 10 C.F.R. Part 100, Appendix A (Findings 37, 43, *supra*). Because of agreement between the Staff's and Applicant's positions on the matter (Findings 38-39, *supra*) and for other reasons found below, the Board also finds that the appropriate magnitude of the controlling earthquake in the Michigan Basin tectonic province is $m_{blg} = 5.0$, rather than either the magnitude of 4.5 originally proposed in the FSAR, or the magnitude of 5.3 assigned to the controlling earthquake in remaining parts of the Central Stable Region.

53. The Board would accept either of the sets of boundaries for the Michigan Basin tectonic province that were provided by the Applicant (Holt Exh. 9 and Exh. 10, Fig. 5; Tr. 4562-62 (Holt)), except that we would exclude the southeastern corner of Michigan about which the Staff expressed reservations. Tr. 4837 (Kimball); *see also* our composite

[57] The Applicant's witness used this same argument as to why the Central Stable Region should be divided, going so far as to state that "defining the tectonic province based on the presence of a veneer of sedimentary rock is unreasonable" (Holt, ff. Tr. 4539, at 13). Thus the Board views as inconsistent both the Applicant's and Staff's arguments against using the veneer of sedimentary rocks as a basis for defining a tectonic province.

map in the Opinion section, Figure 1, *supra,* for what we understand to be the area that should be excluded.

54. The number of historic earthquakes that have occurred within the Michigan Basin is quite small. The Staff's witness, Mr. Kimball, estimated the number as "around ten" for the State of Michigan and referred to the Applicant's documents as a source of the actual numbers (Tr. 4755). By referring to Holt Exhibit 9, the Board counted twenty-two earthquake epicenters on or within the boundaries of the larger version of the tectonic province shown thereon, five of which would have occurred within the excluded southeastern portion. Thus the larger version of the Applicant's proposed tectonic province, as modified herein, would have experienced seventeen earthquakes in historic times. The smaller version (Fig. 5 of Holt Exh. 10) of the Michigan Basin, also excluding the southeastern corner, would contain only about nine historic earthquakes, by the Board's count.

55. Approximately fourteen more historic earthquakes (depending upon how many are counted in the Keweenaw Peninsula) are shown on Holt Exhibit 9 as having occurred within the region that the Staff would have included in its westward extension of the tectonic province, which extension alone would have about twice the area of either version of the Applicant's proposed tectonic province.

56. While the Board finds that the paucity of historic earthquakes in the Michigan Basin is, indeed, indicative of low seismic hazard, the data are so scant that the uncertainty that the maximum reported event represents a conservative controlling earthquake is large. *See* responses to Board questions on seismological and statistical uncertainties in this region. Tr. 4749-57 (Kimball), *especially* Tr. 4753-54, 4756-57.

57. Although we find that the Staff did not adequately support its proposed westward extension of the Michigan Basin tectonic province, it is clear that the Staff's proposed basis for that extension is essentially a perceived uniformity of seismic hazard across the entire region from Michigan to Minnesota. Tr. 4785-86, 4791-92 (Kimball).

58. Ground motions from two historic earthquakes larger than magnitude 5.0, that occurred outside the Michigan Basin tectonic province, were considered in the determination of maximum vibratory ground motions at the Midland site. These occurred near Timiskaming, in Canada, and near Anna, Ohio. *See supra* Figure 1; also *infra* Finding 62, regarding the location and possible recurrence of the New Madrid earthquake. The magnitude of the Timiskaming event was greater than 6.0. Tr. 4777 (Kimball). The Anna, Ohio earthquake, which is the controlling earthquake in the Central Stable Region, has been assigned a

magnitude of 5.3, although the Applicant claimed that a recent authoritative report indicated that it should be 5.0 instead of 5.3. Mr. Holt, however, was unable to justify adequately the differences between this report and an earlier report by the same author which assigned a magnitude of 5.3 to this same earthquake. Finding 22, *supra*; Kimball, ff. Tr. 4690, at 5; Holt, ff. Tr. 4539, at 7, 13 [illegible]; Tr. 4573-74, 4633-34 (Holt). On the basis of the evidence of record, the Board finds no reason to support a reduction or modification of the magnitude of the Anna, Ohio earthquake to below 5.3.

59. Questions concerning the Timiskaming earthquake were raised by the Board (Tr. 4765-69, 4770-72, 4776-81) mainly to be reassured that it had not been overlooked because of its occurrence outside the United States. While this event fell within the Applicant's "Western Quebec Seismic Zone," a fact not obvious during the hearing (*but see* the Board's overlay of the Applicant's seismic maps, Figure 1, *supra*), it was not specifically discussed in either the Applicant's[58] or Staff's prepared testimony. The Staff's expert subsequently testified that, using a magnitude of 6.2 for the Timiskaming earthquake, it would have to occur at least as close as 100 miles from the site to produce ground motion that would exceed the potential for coming close to the accepted (SSRS) spectrum. He further testified that while the boundary of the tectonic province containing the Midland site might extend northeastward to abut the province containing the Timiskaming earthquake, the boundary in that direction would in any case be more than 100 miles from the Midland site. Tr. 4808-09 (Kimball).

60. The Anna earthquake occurred about 205 miles south of the Midland site. The Applicant's witness testified that the closest approach of the boundary of the Michigan Basin tectonic province was about 150 to 170 miles from the site in that direction. However, in making that statement, he had not considered excluding the southeastern corner of Michigan, as was later suggested by the Staff and which exclusion the Board is accepting in this decision. Holt Exh. 10, at 2; Tr. 4571, 4578 (Holt). Even with the exclusion, the nearest approach of the tectonic province boundary, which the Board has drawn conservatively, would be no closer than about 70 miles (*see supra* Figure 1). While Mr. Holt had not actually performed the calculation, he estimated that a 5.3 magnitude Anna-type event would have to come closer than 100 miles from the site, possibly within 50 miles, before its motions would exceed motions

[58] While Mr. Holt did not testify on this subject, the Board assumes that the Applicant's witness would have associated the Timiskaming earthquake with his "Western Quebec Seismic Zone," had he had the opportunity to do so. The Board also notes that this zone appears to be the same as the Ottawa portion of the "Boston-Ottawa belt" discussed *supra*, in Finding 47.

of a magnitude 5.0 event at the site. Tr. 4575 (Holt). The Staff's actual calculations indicated that an Anna-type event would have to occur much closer, something like 25 miles, to the site, before its motions would exceed those of a magnitude 5.0 earthquake at the site. Tr. 4784 (Kimball).

61. The Board finds that the magnitude m_{blg} = 5.0 controlling earthquake for the tectonic province in which the site is located is the appropriate basis for the SSE at the Midland site. It would produce the maximum ground acceleration at the site because no capable faults or tectonic structures with which earthquakes may reasonably be correlated exist within 200 miles of the site, and because its ground accelerations would be greater at the site than those resulting from earthquakes in adjacent or nearby tectonic provinces, assuming those earthquakes occurred at a point on the tectonic province boundary nearest the site. Findings 36, 58-60, *supra*.

C. Construction of the SSRS

62. Representation of the ground motions associated with the SSE was evaluated by the Staff using the SSRS determinations made by the Applicant, but without including spectra from the Parkfield event, the only earthquake in the Applicant's SSRS ensemble with a magnitude greater than m_{blg} = 5.5. Thus for a magnitude 5.0 SSE, the "without Parkfield" site-specific spectra conservatively met the Staff's magnitude criterion specified in the Tedesco letter of ± 0.5 magnitude units. The low-frequency end of the SSRS was modified so as not to fall below the DBE spectrum and to account for the possible effects at the site of distant, very large earthquakes, such as a recurrence of the New Madrid earthquake. The Applicant's witness agreed that the Staff's use of the SSRS without the Parkfield records was an accurate, and conservative, representation for a magnitude 5.0 event at the Midland site. Kimball, ff. Tr. 4690, at 22-23; Tr. 4700 (Kimball); Holt, ff. Tr. 4539, at 8-9, 22-23; Holt Exh. 5, Table 2; Tr. 4541-42, 4570-71, 4586-88 (Holt).

63. Different representations of the SSE ground motions were derived for those safety-related structures founded on natural soils (glacial till and lacustrine clays) and for those founded on soil fill material, to comply with the requirement of Appendix A to Part 100 that SSE response spectra be determined at the elevations of the foundations of plant structures. Holt, ff. Tr. 4539, at 9-10; Kimball, ff. Tr. 4690, at 23-25; *see infra* Findings 66, 72-74.

64. During the hearings there was very little real controversy about the acceptability of the Applicant's SSRS and their applicability to the

Midland site. However, Mr. Holt's prepared testimony, especially that objecting to use of the magnitude 5.3 Anna-type earthquake and consequent inclusion of spectra from the Parkfield event (Holt, ff. Tr. 4539, at 7, 15-20; Holt Exhs. 7, 10, at 4-5, 9-10) must be read in light of Staff's subsequent conclusion, and this Board's concurrence, that a smaller SSE would be appropriate. Similarly, those parts of Mr. Kimball's prepared testimony on the Staff position that Parkfield spectra should be included (ff. Tr. 4690, at 12-16) should be read as if dependent upon a finding that the Central Stable Region with a magnitude 5.3 controlling earthquake would be the appropriate tectonic province for seismic design considerations at Midland. The Staff position that Parkfield records would be appropriate for inclusion in the SSRS ensemble for an Anna-type SSE (magnitude 5.3) was unchanged. Both witnesses agreed, eventually, that Parkfield spectra should not be used in construction of the SSRS for Midland because the magnitude of that event (between 5.6 and 5.9) was outside the magnitude range of 5.0 ± 0.5 m_{blg}. Tr. 4594-95 (Holt); Tr. 4723-24, 4727, 4735-36, 4814-17 (Kimball).

65. Aspects of the testimony concerning inclusion or exclusion of Parkfield data were material, however, to two issues on general criteria for construction of SSRS, i.e., selection of the appropriate statistical (percentile) spectral level within the ensemble of response spectra[59] for representing the SSE, and the inclusion of response spectra from accelerograms recorded at short distances from an earthquake (the so-called "at the site" requirement of § V(a)(1)(ii) of Appendix A to Part 100, applicable where the SSE is identified with the tectonic province in which the site is located).

66. Construction of the site-specific response spectra at the top of the natural soils ("original ground surface") for the Midland site involved calculation and statistical combination of individual spectra from records of forty-four horizontal components[60] of twenty-two acceler-

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[59] Appendix A to Part 100 (at § III(i)) defines a response spectrum as "a plot of the maximum responses (acceleration, velocity or displacement) of a family of idealized single-degree-of-freedom damped oscillators against natural frequencies (or periods) of the oscillators to a specified vibratory motion input at their supports." Essentially it shows how structures (the oscillators) with a given level of inherent damping but different natural (resonant) frequencies, would amplify the input motions of a postulated earthquake. Damping values and natural frequencies of structures depend upon their physical properties and dimensions, and their determination is another part of the seismic design process. For purposes of comparison, the response spectra generally have been displayed in this proceeding as calculated for 5% of critical damping, but response spectra for other damping values have been constructed and will be applied as appropriate to the individual structures. *See* Holt, ff. Tr. 4539, at 4; App. FOF, ¶ 2, n.5, *quoting Pacific Gas and Electric Co.* (Diablo Canyon Nuclear Power Plant, Units 1 and 2), ALAB-644, 13 NRC 903, 924 n.40 (1981); Holt Exh. 5, at 13.

[60] A strong-motion instrument station usually measures motions along three orthogonal axes, two horizontal and one vertical. Tr. 4582 (Holt). The horizontal components are those of greatest concern in seismic analysis and design practice.

ogram sets taken during ten earthquakes that occurred within 25 km (about 15.5 miles) of the individual recording stations. Five of the earthquakes occurred in California and five in Italy. The records were selected to include all those available worldwide from stations that have recorded earthquakes within the 25-km distance, and in the magnitude range equivalent to Central United States m_{blg} = 5.3 ± 0.5,[61] and founded on stiff soils having approximately the same shear-wave-velocity profiles and horizontal layering as those occurring beneath the Midland site. When the Parkfield event is excluded, the magnitude range of earthquakes actually used is 4.9-5.5. Holt Exh. 5, at 6-10, and Table 2; Tr. 4583-85 (Holt).

67. Mr. Holt in several places attacked the Staff's requirement, as expressed in the Tedesco letter, for using the 84th percentile level in statistically combining the individual spectra to arrive at the SSRS. He addressed this requirement as arbitrary and as not being required statistically. While he also asserted that justifications exist for spectral combination at some lower level, i.e., the mean, the 72nd or the 76th percentile, he presented no evidence or reasoning sufficient, in the Board's view, to support those assertions. Holt, ff. Tr. 4539, at 17-18, 20; Holt Exh. 3; Holt Exh. 10, at 9-10.

68. One of the Staff's principal reasons for requiring this particular spectral level (84th percentile) was that it was the level used in construction of the generalized response spectra found in Regulatory Guide 1.60 and, therefore, was appropriately conservative. Additionally the Staff pointed to the necessity of including records that account for uncertainty in the source properties of the design earthquake other than its magnitude, e.g., stress drop, fault rupture length, fault displacement, and rupture velocity. Kimball, ff. Tr. 4690, at 10-11, 15-16; Tr. 4735-36 (Kimball). Records containing the possible effects of such variables can appropriately influence the combined spectra when enveloped at the 84th percentile level. The effect of including a few such spectra, among a total of thirty or more, would be inappropriately minimized when combination is at the mean or median level. The Board finds that a purpose of utilizing many records, assuming they meet the site-and-magnitude matching and distance requirements, is to *include* the effects of these unknown parameters, not to average them out of the design spectrum.

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[61] The m_{blg} magnitude was devised by Dr. Otto Nuttli for use in the central United States. In the magnitude range around 5.0 to 5.5 it is approximately equivalent to the Richter magnitude, M_L, developed for California and also applicable in Europe. Thus M_L values in California and Italy can be used as equivalent to m_{blg} values in the central United States. *See* Tr. 4691-95, 4711-13, 4718-23 (Kimball) for clear and concise discussions of various earthquake magnitude relationships.

69. A distinction of considerable importance in constructing site-specific response spectra was drawn by the Staff's witness between "nearfield response spectra" and response spectra that include some nearfield records and are used to characterize the SSE where the SSE is identified with the tectonic province in which the site is located. "Nearfield response spectra" (which are also site-specific) represent ground motions at a given distance from a known nearby earthquake source such as a capable fault or zone of reservoir-induced seismicity. On the other hand, where neither tectonic structures with associated earthquake activity nor reservoir-induced earthquake activity are known to occur near the site, as at Midland, some nearfield records, if meeting the other matching criteria, would be included in the SSRS ensemble of records. The number of nearfield records to be included would be a specific consideration on a case-by-case basis. Indeed, nearfield records were included in the Applicant's construction of the SSRS for the Midland site, even without the Parkfield earthquake records, and the Staff's witness made the unrefuted statement that the Applicant's consultant had previously used Parkfield records in developing site-specific spectra for other central U.S. sites. Tr. 4727-34, 4799-4806, 4813-17 (Kimball); Tr. 4629-30, 4658 (lines 10-23), 4674-75, 4682-83 (Holt); *also see* col. 9 on Table 2 of Holt Exh. 5 for distances less than 10 to 15 km.

70. Use of earthquake records from California and Italy to construct the SSRS for the Midland site was justified on the basis that, out to about 25 km from an earthquake source, the attenuation in all three areas could be assumed to be roughly the same. Thus, if the other parameters (magnitude and site conditions) are matched to those of the plant site, source-to-site attenuation conditions do not significantly affect the records out to a distance of about 25 km. Tr. 4580-83 (Holt); Tr. 4691-95, 4803, 4805 (Kimball).

71. The SSE response spectra, or SSRS, as accepted here for the Midland site are higher than the modified Housner original design spectra except that they have been constrained not to fall below, and to be congruent with, the original spectra in the frequency range below 1 Hz. In the high-frequency range between 5 Hz and 25 Hz (where the original Housner spectra, "anchored" at 0.12*g*, had not been raised, or modified, at the CP stage), the SSE response spectrum (for 5% damping) exceeds the original design spectrum by 18% to 104%; that is, the SSRS is about double the original design spectrum from 5 Hz out to about 15 Hz.[62]

[62] In footnote 157 to its Proposed FOF, ¶ 77, the Applicant incorrectly reversed the meaning of its witness' statement on the relationship between the two spectra. While the question and answer may have *(Continued)*

The SSRS, or SSE response spectrum, is roughly equivalent to a Regulatory Guide 1.60 standardized response spectrum anchored at $0.12g$ to $0.13g$. Kimball, ff. Tr. 4690, at 10-11, 22-23, Fig. 1; Tr. 4787-88 (Kimball); SER at 2-34, 2-37, 2-38, Fig. 2.7; Tr. 4639-40 (Holt); Holt Exhs. 1 and 2; Holt Exh. 6, Figs. 1.1 and 1.2; Holt Exh. 11. Figure 2.7 of the SER, and Holt Exh. 1, with an overlay of Holt Exhibit 2 are reproduced here for convenience as Figures 2 and 3, *supra* pp. 66-67.

72. Appendix A to 10 C.F.R. Part 100, § V(a)(1)(iv), requires the development of response spectra at each of the various foundation locations of safety-related structures at the plant site. Because some of the main structures were founded entirely in plant fill and were not to be underpinned to the natural soils below, site-specific response spectra were constructed for the top of the plant fill. The effect of the layer of fill, which is about 30 feet thick and softer than the natural soils, would be to amplify certain ground motions, mainly those with a vibratory frequency between 1 Hz and 4 Hz, in the event of occurrence of an earthquake. These response spectra would have been applicable to the seismic reevaluation of the diesel generator building, the borated water storage tanks and the railroad bay area of the auxiliary building. Holt, ff. Tr. 4539, at 9-10; Holt Exhs. 1, 2, 11, and 8, at 1-7 and Fig. 7; Kimball ff. Tr. 4690, at 23-25; Tr. 5107, 5110-11 (Kimball); SER, Table 2.2, at p. 2-46.

73. The same general methodology that was used for calculating the SSRS at the top of the natural soils was employed to calculate the SSRS at the top of the plant fill, except that allowances were made for the softer materials and 30-foot thickness of the fill layer, placed on the stiffer natural soils. The ensemble of records used consisted of thirty-six components (from eighteen record sets) taken at ten sites during twelve earthquakes, eight of which occurred in California and four in Italy. The earthquakes ranged in magnitude from 4.9 to 5.6; epicentral distances ranged from 6 to 30.5 km, and the accelerograph stations were selected on the basis of the similarities of their soil properties and layering to those beneath the Midland plant site areas with the soil fill layer. Ten of the eighteen record sets taken at five sites had also been used in preparation of SSRS for the top of the natural soils. This overlap of sites and records used in the two compilations was cited as "reflecting the flexibility in the station characteristics that must be allowed during the selection

allowed this ambiguity (Tr. 4639-40), it is clear from Mr. Holt's other testimony, e.g., Holt Exhs. 2 and 11, that he was aware that the original design spectrum ("FSAR SSE accelerations") never exceeded the SSRS by any amount in the frequency range specified (5 Hz = 0.2 second-period and 15 Hz = 0.067 second-period).

process" (Holt Exh. 8, at 4). The Board assumes that this means that the materials and layering at those accelerograph sites were sufficiently similar to match either of the profiles to be modeled at the Midland site. Holt, ff. Tr. 4539, at 9-10; Holt Exh. 8, at 2-5, Table 1 (*cf.* Table 2 of Holt Exh. 5), Fig. 7; Kimball, ff. Tr. 4690, at 24-25.

74. An alternative approach to determine the SSRS at the top of the plant fill would be to compute amplification factors (and an amplification spectrum) for increasing the SSRS responses at the top of the natural soil. The Applicant accomplished this as a check against the top-of-fill SSRS that was calculated directly from the site-and-magnitude-matched ensemble of earthquake records. The SHAKE one-dimensional wave propagation computer code was applied to four different soil profiles to account for the heterogeneous nature of the plant fill, and amplification spectra were determined. Holt, ff. Tr. 4539, at 10; Holt Exh. 8, at B-1 to B-5; Kimball, ff. Tr. 4690, at 23-24.

75. The Staff employed Dr. Paul F. Hadala of the Army Corps of Engineers to review the Applicant's amplification spectra analyses. Dr. Hadala also performed his own analyses using the SHAKE computer code, but used what he believed to be more realistic soil and bedrock outcrop stiffnesses and earthquakes as input. He concluded that if one accepts the validity of the SSRS for the original ground surface then the directly computed SSRS for the top of plant fill is more conservative than the response spectrum derived from the theoretically calculated amplification factors. Hadala, ff. Tr. 5081, at 2-7; Kimball, ff. Tr. 4690, at 25.

76. The SSRS developed for the top of plant fill was modified in the low-frequency range (1 Hz and below) in a manner similar to that developed for the top of the natural soils, i.e., it was constrained so as not to fall below the original design spectrum for the Midland plant. Tr. 5108-14 (Kimball); Holt Exh. 11; *see also* Finding 71, *supra.* The Board accepts this SSRS (as shown on Holt Exhibit 11) with the understanding that, were the project reactivated, it would be used for seismic reevaluation of safety-related structures founded on, or in, the plant fill.

77. At the time when the Applicant undertook design of the underpinning structures and the new ring-beam foundation of the borated water storage tanks, and began seismic reevaluation of structures founded in soils, no agreement existed on the seismic criteria for those structures. In order to proceed, the Applicant incorporated what it believed to be a reasonable margin over the original DBE into the design or reevaluation of those structures. The Applicant directed its contractors to use 1.5 times the DBE (or "FSAR SSE") response spectra as the seismic

design basis for the remedial structures and for the various seismic reevaluations (but not for the seismic margin review). Subsequently, the Applicant committed to use of the SSRS, as accepted here, as a seismic design basis, but it continued to use the 1.5 times the DBE ("FSAR SSE") spectra in the actual remedial design work. The Applicant also had dynamic analyses performed which demonstrated that, for purposes of design of the remedial structures, the seismic design basis used exceeded the responses derived from the SSRS. Thiruvengadam Affidavit at 6-7; Tr. 5996-97, 5996-6005, 6027-28, 6040-43 (Kennedy).

78. In answers to questions about the adequacy of 1.5 times the DBE as a design basis, the Applicant's witness, Mr. Robert P. Kennedy, testified that in parts of at least one structure or substructure not founded on plant fill (the missile shield in the main portion of the auxiliary building) the SSRS responses were 1.7 times the DBE spectral responses, but that the SSRS responses would be used in the seismic reevaluation of the missile shield. Tr. 6002-03, 6029-32 (Kennedy). That reevaluation, as part of the seismic margin review, would have been considered in the later-scheduled OL portion of this proceeding, but is not material to issues dealt with in this Decision. SSER # 2, § 3.7.2.1, at 3-2.

79. Accordingly, the Board finds that the Applicant's use of the SSRS for seismic reevaluation of safety-related structures, systems and components of the plant, and its substitute use of 1.5 times the DBE ("FSAR SSE") response spectra in seismic design of the remedial structures, is reasonable and conservative.

D. Seismic Models and Soil Spring Constants

80. As provided in our May 5, 1981 Prehearing Conference Order, one of the issues considered in the soils hearings was the mathematical models to be used for dynamic analyses of structures as modified by the remedial soil settlement measures, including the bases for the derivation of the spring constants. The Applicant's consultant, Dr. Robert P. Kennedy of Structural Mechanics Associates, Inc. ("SMA"), testified on the dynamic mathematical models being used to perform the seismic evaluation of structures in conjunction with the foundation remedial work. Dr. Kennedy summarized the dynamic models developed for (1) the auxiliary building — control tower — electrical penetration area ("auxiliary building") which is supported on an interconnected foundation system; (2) the service water pump structure ("SWPS"); and (3) the borated water storage tank ("BWST"). The auxiliary building and SWPS models were developed by Bechtel Corporation, and important features of the models were reviewed by Dr. Kennedy and SMA. The BWST model was

developed by Dr. Kennedy and SMA. Kennedy, ff. Tr. 5995, at 1; Tr. 5998-6121, 6250-86 (Kennedy). The NRC Staff structural reviewer, Mr. Frank Rinaldi, and the Staff's consultants, Dr. Paul Hadala of the Corps of Engineers and Mr. John Matra of the Naval Surface Weapons Laboratory, presented the results of their review of the Applicant's dynamic models. Rinaldi/Matra, ff. Tr. 6129; Tr. 6121-36, 6252-86 (Rinaldi, Matra, Hadala).

81. Dynamic mathematical models are used to define the response characteristics of a structure subjected to a dynamic forcing function.[63] For the seismic evaluation of complex buildings, such as the auxiliary building or the SWPS, a two-step modeling procedure is commonly used. First, an overall dynamic response model of the complete structure is developed. This model must be adequate to determine the seismic-induced forces, shears, moments, displacements, and accelerations at all important locations throughout the structure, as well as to determine the seismic input to equipment mounted on the structure. Second, detailed static models for local regions of the complex structure are developed. These detailed static models are used to convert the overall seismic-induced dynamic responses (step one) to local forces and stresses for use in the seismic evaluation of the design of individual structural elements. The dynamic mathematical models presented by Dr. Kennedy are only intended for the first step; i.e., to determine adequately and conservatively the overall seismic-induced forces, shears, moments, displacements, and accelerations throughout the auxiliary building, SWPS, and BWST structures and foundations and to determine the seismic input to equipment mounted on these structures. Kennedy, ff. Tr. 5995, at 2-3; Tr. 6009-10, 6102-05 (Kennedy).[64]

82. Dr. Kennedy's testimony addressed various influences upon the overall dynamic response of a complex structural system to seismic input, but this Partial Initial Decision will summarize only the Applicant's treatment of soil-structure interaction and energy dissipation capability, which have special pertinence to this proceeding. A soil-structure interaction model must (1) feed the seismic input into the building models at the appropriate elevations and plan view locations (center of rigidity of the supporting soil); (2) account for the reduced

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[63] The mathematical representation of structures by dynamic models is not always necessary. For a very simple building, or for simple below-ground structures such as valve pits and retaining walls, an analyst can determine the natural frequency of vibration and thus the structural responses without constructing a dynamic model. Kennedy, ff. Tr. 5995, at 6.

[64] The Applicant described the detailed static (finite-element) models used in designing the remedial underpinning work in other testimony. *See* Burke, *et al.*, ff. Tr. 5509 (auxiliary building); Boos, *et al.*, ff. Tr. 9490 (SWPS); Poos/Hanson, ff. Tr. 7173 (BWST).

stiffness of the overall b 'ding system due to the flexibility of the supporting soil; and (3) cons.rvatively account for the radiation of energy (associated with build..ng response relative to the soil) from the building into the surrounding soil. Kennedy, ff. Tr. 5995, at 5.

83. The soil-structure interaction effect on complex buildings such as the auxiliary building is a complicated and controversial subject. A complete interaction analysis is beyond the current state of the art and cannot be performed for complex buildings. Dr. Kennedy testified that the soil-structure interaction models incorporated into the auxiliary building, SWPS, and BWST dynamic models for the foundation remedial work are very simple. They do not represent the most advanced state-of-the-art models, but they were developed in such a way as to provide high confidence that they will either accurately compute or conservatively overpredict the seismic response of the structures. Kennedy, ff. Tr. 5995, at 7-8; Tr. 6099-6102, 6105-08, 6118 (Kennedy).

84. Because of uncertainties in soil properties and in the mathematical modeling of soil-structure interaction, there is significant uncertainty in the "softening" effect of soil-structure interaction.[65] In order to cover this uncertainty, the Applicant and its consultant were to have varied the soil-structure interaction stiffnesses within the range from 0.5 to 1.5 times the "best estimate" soil-structure interaction stiffnesses. Dr. Kennedy testified that using this wide range of soil properties avoids the need for more sophisticated soil-structure interaction modeling. Kennedy, ff. Tr. 5995, at 9.[66]

85. Dr Paul Hadala of the Corps of Engineers evaluated for the NRC Staff the methods used by the Applicant in calculating soil spring constants and damping parameters for the auxiliary building, the SWPS, and the BWSTs. Dr. Hadala used a different method of calculation than did the Applicant. Dr. Hadala used field-measured seismic shear wave velocities in the plant fill and in the glacial till to derive a shear modulus. He then made a reduction based on the work of Seed and Idris to account for the fact that strain levels in earthquakes are larger than those in field seismic shear wave velocity tests. His result was in close agreement with the Applicant's best-estimate soil properties. Dr. Hadala testified that the methodology used by the Applicant and its consultant in determining soil spring constants and damping parameters is a sound one which provides conservative answers for estimating the transmission

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[65] The "softening" effect is the effect of soil-structure interaction on the natural frequencies and mode shapes of vibration of the structure.

[66] As we point out elsewhere in this Decision, *supra* pp. 70-71, the Applicant (through Bechtel) failed to include the ± 50% variation in soil modules in analyzing the auxiliary building and SWPS. Dr. Kennedy did include this variation in his BWST analysis. *See infra* Finding 88.

of energy away from the structure due to radiation damping and the contribution of the foundation soil to the stiffness of the system. Tr. 6130-31, 6278-79 (Hadala).

86. The Applicant's witnesses presented the dynamic models for the auxiliary building, SWPS and BWSTs. The auxiliary building is represented by a three-dimensional, lumped-mass stick model, with additional detail in the electrical penetration areas, which preserves the physical geometry of the various building components. The SWPS is represented by a three-dimensional lumped-mass stick model using beam elements. The model which has been submitted for the BWST, which was developed by Dr. Kennedy and SMA, and replaces a model which Bechtel had developed, is somewhat different.[67] The BWST is a vertical cylindrical tank which is supported by the soil beneath the tank and anchored to a ring foundation. The ring foundation must withstand the seismic-induced forces in the tank shell. These forces are nearly totally due to the water in the tank since the tank shell weight is negligible when compared to the weight of the borated water. Therefore, the primary seismic modeling concern is to model properly and conservatively the seismic forces induced by the water on the tank shell and thus also on the foundation. Dr. Kennedy testified that it is best to model the impulsive mode, the sloshing mode, and the vertical mode of fluid-structure interaction individually. The seismic forces imposed upon the tank shell and ring foundation are added by the square-root- sum-of-squares method. The impulsive mode is modeled by vertical stick elements between mass points distributed up the tank shell. A dynamic model is not required to evaluate the forces in the sloshing and vertical modes. The forces in these two modes can be determined by mathematical equations. Dr. Kennedy testified that the foundation ring does not affect seismic modeling except that the rings act as an anchor for vertical movement. Thus, the facts that the old foundation ring is out of plane and is cracked, and that another foundation ring will be added to the BWST foundation as a remedial measure, are irrelevant in the determination of seismic response of the BWST.[68] For details on all of these models, *see* Kennedy, ff. Tr. 5995, at 13-22, Figs. 2-12; Rinaldi/Matra, ff. Tr. 6129, at 3-5.

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[67] The foundation of the BWST has been designed based upon the Bechtel dynamic model. The Bechtel model predicts higher loads on the foundation than the Kennedy model by about 20% or a factor of 1.2. Because BWST foundation design loads are based upon the higher Bechtel model, extra conservatism is provided in the remedial work. Dr. Kennedy's mode was to be used in the seismic margin review and in checking of the forces on the tank for the SSRS. Tr. 5991-94, 6006-08 (Kennedy); Tr. 6279-80 (Rinaldi).

[68] Unlike Dr. Kennedy's model, which considers the tank to be supported by the soil at the base point of the tank, Bechtel's dynamic model includes the foundation ring. Dr. Kennedy explained that this is one of the reasons why his model is better and more accurate. Tr. 6044-52, 6059-63 (Kennedy).

Dr. Kennedy concluded that the dynamic models for the auxiliary building, SWPS and BWST are adequate for establishing the conservative seismic forces to be used in the design of the remedial work and in the seismic margin review. Kennedy, ff. Tr. 5595, at 19-22, Figs. 13-14, Attachment B.

87. In addition to the review of soil spring constants and damping parameters by Dr. Hadala, the NRC Staff's structural reviewer, Mr. Frank Rinaldi, and its consultant Mr. John Matra of the Naval Surface Weapons Laboratory reviewed the other aspects of Applicant's dynamic models. The NRC Staff found that the methodologies used by the Applicant and its consultant to develop and to review the dynamic mathematical models are within the state of the art, and that the auxiliary building and SWPS models adequately represented those structures within the state of the art. Rinaldi/Matra, ff. Tr. 6129, at 9, 11-14; Tr. 6131 (Hadala); Tr. 6131-34, 6258, 6266 (Rinaldi); Tr. 6134 (Matra). *But see* Finding 88, *infra*. Following its review of the dynamic model for the BWST, Mr. Rinaldi and Mr. Matra testified that the Applicant's dynamic analysis of the BWST was satisfactory. Rinaldi/Matra, ff. Tr. 7537, at 3.

88. By Board Notification BN 84-115, "Seismic and Structural Design Departures from Licensing and Design Criteria — Midland Plant," issued June 18, 1984, by the Staff, the Board and parties were advised of the Applicant's discovery during a design review that, in the original seismic design, Category I structures were analyzed using only the nominal soil dynamic modulus value without considering the ± 50% variation of that value as required by the FSAR. The design review, and BN 84-115, followed by several months the presentation of testimony on the seismic models. By letter dated August 2, 1984, the Staff supplemented BN 84-115 by identifying certain of its testimony and evidence which would be affected by the reported deficiencies (including testimony by Messrs. Rinaldi, Matra and Hadala). The impact of the design deficiency would be applicable to the seismic design of the underpinning structures (under the auxiliary building and the SWPS), and to the criteria to be established for subsequent seismic margin reviews of plant safety structures, i.e., the soil spring constants. The deficiency would not be applicable to the seismic design of the BWSTs, since Dr. Kennedy took into account the requisite variation in the nominal soil dynamic modulus value in deriving his new seismic model for the BWSTs. Tr. 6001-04 (Kennedy); *see also infra* Finding 192. Our conclusions with respect to the seismic models for the auxiliary building and SWPS — but not the BWSTs — are qualified to the extent they may be affected by the design deficiency.

89. The Licensing Board finds that the methodology used to develop the models for the auxiliary building, SWPS, and BWST was within the state of the art. The Board concludes that these models are adequate for the purpose of defining seismic design forces to be used in the design of foundation remedial work, for conservatively estimating the seismic-induced forces in these structures, and for defining the seismic input to equipment, systems, and components mounted on these structures. With respect to the auxiliary building and SWPS models, however, this conclusion is limited to the establishment and validity of the nominal values of the soil spring constants. Although the record establishes some measure of conservatism in the seismic design of the auxiliary building and SWPS by virtue of the exceedance of the SSRS by 1.5 x the DBE (FSAR SSE) response spectra actually used in the design of the underpinning, the record is not sufficient to permit a determination of whether the conservatism in calculation of seismic loads provided by use of the 1.5 x DBE (FSAR SSE) response spectra is sufficient to include the range of seismic loads that would result from the required variation of soil spring constants in those calculations.

E. Soil Liquefaction Potential

90. The potential for liquefaction[69] at a power plant site is a necessary part of the seismic evaluation prescribed by NRC regulations. *See* 10 C.F.R. Part 100, Appendix A, §§ V(d) and VI(a).[70] Its potential occurrence at Midland gave rise, *inter alia*, to the permanent dewatering system discussed *infra* in Findings 98-117. That potential became apparent when, following the discovery of excessive settlement of the partly built DGB in July of 1978 the Applicant undertook an extensive underground soils investigation program. One of the results of the borings and

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[69] Liquefaction of loose, cohesionless sands that are saturated with water is a phenomenon that may occur during strong earthquake shaking that results in loss of shear strength of the material. During the shaking, partial compaction may occur and the weight of the overburden and any overlying structures, if present, is transferred to the pore water which cannot escape rapidly enough to dissipate the elevated porewater pressures that result. Because the load, then, is borne largely or entirely by the water, which has no shear strength, the sand-water mixture behaves like a liquid. Woods, ff. Tr. 9745 at 3; *cf.* Woods, ff. Tr. 11,549, at 23 on a related phenomenon, seismic shakedown, in unsaturated loose sand.

[70] The adequacy of the seismic evaluation at Midland, and of the capacity of various structures to withstand liquefaction, was dealt with generally by Ms. Stamiris' Contentions 4.C and 4.D (which are quoted in full in findings on particular structures or dewatering, as well as in Appendix A). The only contention which specifically mentioned liquefaction was Warren Contention 2.B, which reads as follows:

> Given the facts alleged in Contention 2.A [concerning the adequacy of the permanent dewatering system], and considering also that the Saginaw Valley is built upon centuries of silt deposits, these highly permeable soils which underlie, in part, the diesel generator building and other class I structures may be adversely affected by increased water levels producing liquefaction of these soils.

soils testing was the identification, in isolated areas, of potentially liquefiable sands in the plant fill beneath certain safety-related structures and underground utilities at the Midland facility. These were the DGB, the EPAs and railroad bay/radwaste structure (RBA)[71] of the auxiliary building, the overhanging portion of the SWPS, and a portion of the service water piping. Underpinning the EPAs and the "cantilevered" part of the SWPS was to have eliminated the concern about potential liquefaction of their foundation soils, by extending their foundations down to dense natural soils beneath the plant fill. Other remedial action (e.g., dewatering or removal of loose sands) was needed to reduce or eliminate the liquefaction potential of plant fill soils beneath the DGB and the RBA, and beneath parts of the service water piping. While sands of questionable density were discovered in a few places in the natural soils, the evaluations of the Applicant and Staff showed that potential liquefaction of natural soils was not a problem beneath any safety-related structures or utilities. SSER # 2, § 2.5.4.5.5, at 2-42 to 2-43; Woods, ff. Tr. 9745, at 7-14, Figs. L-3, L-4, L-5 (locations of borings); Tr. 9786, 9793, 9802-03 (Kane). (With respect to borings under the diesel fuel oil tanks, we are making no findings, for reasons set forth *supra* pp. 38 and 103-04, and *infra* Finding 202.)

91. The Applicant and the Staff both conducted independent evaluations of the liquefaction potential of the loose sands encountered during the boring program. The U.S. Army Corps of Engineers, acting as a consultant to the Staff, performed a study of soils liquefaction potential and the permanent dewatering system proposed by the Applicant to eliminate liquefaction potential of loose sands under the DGB and RBA. The Applicant's witness on soils liquefaction was Dr. Richard D. Woods, a professor of civil engineering at the University of Michigan acting as a private consultant. The Staff's testimony on soils liquefaction was presented by Mr. Joseph Kane, a geotechnical engineer with the NRC Staff. SSER # 2, § 2.5.4.4.4, at p. 2-35 and § 2.5.4.5.5, at 2-42 to 2-44; Woods, ff. Tr. 9745; Tr. 9782, *et seq.* (Kane).

92. In their analyses of liquefaction potential, both the Applicant and the Corps of Engineers assumed a magnitude 6.0 earthquake and a peak acceleration of 0.19*g*. Dr. Woods explained that earthquake magnitude determines the number of cycles of stress reversal used in deriving liquefaction potential, and that a single cycle of peak motion would not

[71] The area committed to be dewatered included a small portion of the northeast corner of the radwaste building. The term RBA as used herein includes that corner of the radwaste building (*see* SSER # 2, Fig. 2.4, at p. 2-8).

be a concern. Both the earthquake magnitude used and the peak acceleration used are higher than corresponding values of the SSE (magnitude 5.0) and the peak acceleration (0.12*g*-0.13*g*) associated with the SSRS for the Midland site. Woods, ff. Tr. 9745, at 2; SSER # 2, § 2.5.4.5.5, at 2-43 and 2-44; Tr. 9749-52 (Woods).

93. Whether a specific sand body or layer will liquefy or not depends upon several factors. First, the sand must be loosely compacted, i.e., relatively low in density. Second, the sand must be low in cohesion, or cohesionless, i.e., it does not have a high proportion of clay or other binders. Third, the sand must be saturated; this occurs when the sand is below the water table and the pore spaces are filled with water. If not saturated, a loose, cohesionless sand body may undergo partial compaction during strong earthquake shaking, resulting in settlement ("seismic shakedown"), but not liquefaction (*see infra* Findings 114, 117). Other factors also influence the potential for liquefaction, such as the strength and duration (number of shaking cycles) of earthquake motions, an increase in either of which would increase liquefaction potential. Also, an increase in the effective confining pressure on a sand body (as from a greater depth of occurrence) decreases its liquefaction potential.[72] Manifestations of liquefaction of foundation soils include settlement and tilting of structures, cracking and lateral spreading of slopes and embankments, and disruptions of the ground surface. Woods, ff. Tr. 9745, at 3-7; Tr. 9785-86 (Kane); Woods, ff. Tr. 11,549, at 2-3.

94. Certain of the low-blowcount sand bodies encountered in the borings were not encountered in nearby borings and were surrounded above and below by nonliquefiable soils. These isolated small pockets were not regarded by the Applicant as significant threats to the integrity of safety-related structures. Woods, ff. Tr. 9745 at 11-13; Tr. 9747-48, 9753, 9761-62, 9765-66 (Woods). (With respect to borings used to evaluate the potential for liquefaction under the diesel fuel oil tanks (Tr. 9347-48 (Woods)), we are not making any findings, as a result of the discovery of information indicating those borings may be erroneous. *See*

[72] The Standard Penetration Test (SPT) is commonly employed when making borings to estimate relative density and liquefaction potential of soils. The test procedure consists of driving a standard sampling tube into soil at the bottom of the hole by dropping a "hammer" of specified weight from a specified height onto the drill stem to which the sampler in the hole is attached. The number of blows required to drive the sampler a specified distance is recorded. In general, a low blowcount indicates low relative density and a high liquefaction potential in sand. In his evaluation here, Dr. Woods' calculations resulted in a comparison between the *in situ* blowcount and the predicted blowcount at which liquefaction would not occur during a magnitude 6 earthquake, accounting for sample depth, relative density, and elevation of the water table. Curves were shown for the cyclic stress ratio at which liquefaction would not occur (safety factor of 1.0), and for a safety factor of 1.5 in that value. Woods, ff. Tr. 9745, at 3-7.

supra pp. 38, 103-04, and *infra* Finding 202.) In response to Board questions concerning the necessary lateral extent of sands in order for liquefaction to occur, Dr. Woods stated that, based on his examination of published records of liquefaction events, liquefaction has not occurred in areas where there have not been several acres of liquefiable material that is both in connection and fully saturated (Tr. 9769-72, *corrected* at Tr. 11,550-51 (Woods)). On the other hand, Mr. Kane believed that liquefaction could be a problem in saturated sands in areas under 1 acre. He indicated that, in the consideration of lateral restraint of a confined pocket of sand, it is necessary to consider the depth of the pocket and its location with respect to the foundation of the structure. For example, if it were located so as to be the layer most heavily stressed by the foundation pressures, and it lost its strength through liquefaction, there would be a risk of losing foundation support. Mr. Kane indicated further that dewatering the sands to below elevation 610 feet would resolve the Staff's concerns with respect to liquefaction. Tr. 9793-96, 9799-9800, 9810 (Kane).

95. Dewatering, however, was not to be employed to resolve potential liquefaction of those loose sands beneath service water piping and duct banks located in the vicinity of the SWPS. This was because of the proximity of that area to the cooling pond, the primary source of recharge of the ground water in the plant area. If the dewatering system were to fail, the water table could rise very rapidly in this area and the loose sands, which lie above 610-foot elevation, would become saturated. According to the Staff, it has been demonstrated that the water table, which would have been drawn down to elevation 595 feet, could reach an elevation of 610 feet in this recharge zone in approximately 3 days, which might not allow sufficient time to repair the dewatering system. Therefore the soil beneath the safety-related service water piping and duct banks near the SWPS was to have been removed and replaced with nonliquefiable material down to elevation 610 feet. Woods, ff. Tr. 9745, at 12-13; SSER # 2, § 2.5.4.4.5, at p. 2-36; Paris, ff. Tr. 9900, at B-3; Tr. 9902 (Paris).

96. The potentially liquefiable sands near the SWPS were not identified by the Applicant's representatives during a meeting held with the NRC Staff on March 3, 1982, the purpose of which was to obtain Staff approval of the Applicant's proposed site dewatering criteria, including limitation of ground water control to the areas near the DGB and RBA. The Staff had become aware of loose sands near the SWPS by July of 1980 through its review of the Applicant's logs of borings made in 1979. At the March 3, 1982 meeting, the Staff requested that the Applicant supply the NRC with copies of Bechtel's liquefaction analysis for soils

above elevation 610 feet. CPC subsequently did so. The analysis showed loose sand in the plant fill at locations other than the RBA and DGB, including that beneath the service water piping just north of the SWPS. The Applicant advised the Staff of CPC's intention to remove and replace the loose sand during a telephone call on March 12, 1982. Hood, ff. Tr. 12,144, with attachments; Tr. 12,145-47 (Hood); Tr. 9785-86, 12,168-70 (Kane); Tr. 12,186-99 (Budzik); Tr. 9901-03 (Paris). Because the issue of liquefaction potential in this area was resolved by the commitment to remove and replace the loose sands beneath the service water piping and duct banks north of the SWPS (Finding 95, *supra*), the controversy surrounding the March 3, 1982 meeting is not material to the technical aspects of liquefaction on which we are here ruling. The extent, if any, to which testimony on the March 3, 1982 meeting bears on management attitude was to have been addressed in a subsequent Decision in these proceedings.

97. The Applicant's evaluation of the bodies of loose sand present in the plant fill under the RBA and DGB indicated that almost all of them lie above 610-foot elevation. The few pockets that lie below that elevation are of such limited extent and deep enough that they do not present a liquefaction problem, even if saturated. Therefore, lowering the ground water table and maintaining it at a level below 610 feet beneath the RBA and DGB will ensure that there is no potential for liquefaction of soils to affect the integrity of either structure. The Staff reached the same conclusion based on its independent evaluation and review. SSER # 2, § 2.5.4.5.5, at 2-43 to 2-44; Woods, ff. Tr. 9745, at 8-9, 13, Figs. L-6 through L-9; Tr. 9784-86, 9810-11 (Kane). We agree.

F. Dewatering of Plant Soils

98. In order to reduce or eliminate the potential for liquefaction beneath the DGB and RBA, a permanent dewatering system was to be installed. Woods, ff. Tr. 9745, at 9, 13; Paris, ff. Tr. 9900, at 3-4, 39. This system was the subject of Stamiris Contention 4.D, which reads as follows:

> 4. Consumers Power Company performed and proposed remedial actions regarding soils settlement that are inadequate as presented because:
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> * * *
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> D. Permanent dewatering
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> 1) would change the water table, soil and seismic characteristics of the dewatered site from their originally approved PSAR characteristics — characteristics on which the safety and integrity of the plant were based.

thereby necessitating a reevaluation of these characteristics for affected Category I structures,

2) may cause an unacceptable degree of further settlement in safety-related structures due to the anticipated drawdown effect;

3) to the extent subject to failure or degradation, would allow inadequate time in which to initiate shutdown, thereby necessitating reassessment of these times.[73]

(1) *Sufficiency of Permanent Dewatering System (Stamiris Contention 4.D(3))*

99. Two witnesses described the design of the permanent dewatering system. Mr. William Paris, an engineering geologist with Bechtel testified for the Applicant, and Mr. Raymond O. Gonzales, a hydraulic engineer, testified for the NRC Staff. Other Staff witnesses, including Mr. Kane and Mr. Darl S. Hood, the Midland Project Manager, provided additional testimony pertinent to the effects of dewatering upon plant soils, and other aspects of the dewatering system. *See generally* Paris, ff. Tr. 9900; Tr. 10,012, *et seq.* (Gonzales); SSER # 2, §§ 2.4.6.2, 2.4.6.3, 2.4.6.4; Tr. 10,013, *et seq.* (Hood); Tr. 9812-51 (Kane).

100. The permanent dewatering system was designed to maintain the ground water table below 610-foot elevation beneath the DGB and RBA to eliminate or reduce the liquefaction potential of loose, noncohesive sands present in the plant fill beneath those structures (*see supra* Findings 97, 98). Although the system was not required to be designed to Seismic Category I standards, it was designed to lower the water table to elevation 595 feet. Hence, even in the event of total failure of all pumping capacity, the time required for the water table to rise to elevation 610 feet under the DGB or the RBA (about 40 days) would allow time to repair and restore the system. Paris, ff. Tr. 9900, at 4-5, 30-31; SSER # 2, § 2.4.6.2, at 2-1, 2-5.

101. The main source of water supply, or recharge, to the plant fill would be the cooling water pond, which was to have been maintained at a pool elevation of 627 feet. The main area of recharge would have been in the vicinity of the SWPS and adjacent circulating water intake structure, from where the water would flow through natural sand just below the plant fill. The sands in the plant fill are hydraulically connected to

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[73] Similar considerations were raised by Warren Contention 2.A, which reads as follows:

> Because of the known seepage of water from the cooling pond into the fill soils in the power block area, permanent dewatering procedures being proposed by Consumers Power Company are inadequate, particularly in the event of increased water seepage, flooding, failure of pumping systems and power outages. Under these conditions, Consumers cannot provide reasonable assurance that stated maximum levels can be maintained.

the underlying natural sand. Water from the dewatering system would have been pumped back to the cooling pond. Paris, ff. Tr. 9900, at 6-7, 10-13; SSER # 2, § 2.4.6.2, at 2-1.

102. The cooling pond and area of the power block to be dewatered are hydraulically isolated from aquifers of the regional ground water systems by a widespread underlying natural clay layer about 135 feet thick, and by the enclosing perimeter dike core, cutoff dikes and slurry trenches that were designed to extend down to the natural clay. The dikes and slurry trenches prevent hydraulic connection of the plant fill with laterally adjacent shallow sediments where ground water occurs under water-table conditions in the upper ground water system. An aquifer of a lower ground water system, located beneath the 135-foot-thick natural clay layer, is under artesian pressure with a hydrostatic head about equal to the water-table level of the upper ground water system. Observation wells drilled to the lower aquifer outside the perimeter dike showed no fluctuations with changes of water level inside the dike, indicating a lack of hydraulic connection between the upper and lower systems. The casings of these wells were sealed with grout to prevent a connection whereby water could rise from the lower aquifer and escape into the upper system. Water flow in the opposite direction would be prevented by the artesian pressure in the lower aquifer. Thus the potential sources of recharge of the ground water in the plant fill beneath the DGB and RBA are the cooling pond, leakage from pipes, and natural precipitation falling within the confines of the cutoff dikes and slurry trenches.[74] Paris ff. Tr. 9900, at 6-13; Tr. 9917-31, 9933-34, 9958-62 (Paris); Tr. 9835-37, 9841-43 (Kane); Tr. 10,017-20, 10,035-39, 10,045-51 (Gonzales).

103. Twenty interceptor and twenty backup interceptor wells located in two lines along the primary recharge area, and twenty-four area wells in the site area, form the main components of the permanent dewatering system. They are designed to lower the water table to elevation 595 feet, and to intercept recharge from the cooling pond and from natural precipitation or pipe leakage. While it is anticipated that only one line of interceptor wells and two of the area wells would need to remain in operation to maintain the ground water level at or below the design level, all of these wells were to be operational should the need for any of them arise.

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[74] Testimony was given that granular materials existed beneath the cutoff dike just west of the administration building, which permitted some inflow of water from the upper ground water system to the plant fill. However, because the degree of connection apparently was slight and the difference in head across the dike would be only about 3 feet, even with dewatering, no significant inflow from the upper system was considered likely. Tr. 9846-48 (Kane); Tr. 10,020-21, 10,035-39 (Gonzales); Tr. 10,022-24 (Hood).

Paris, ff. Tr. 9900, at 13-16, 31-32; SSER # 2, § 2.4.6.2, at 2-1 to 2-5, § 2.4.6.4, at p. 2-10.

104. Each of the pumping wells was equipped with a well screen/sand filter pack to reduce the quantity of soils fines removed from the sand through which the ground water would flow. Following well construction and initial development, each well had to meet a test limit of no more than 10 parts per million (ppm) of soils fines to be accepted (*cf.* SSER # 2, § 2.5.4.4.4, at p. 2-35). A lifetime limit of 1 cubic yard of soils fines was to have been specified for each well. If the limit had been reached during plant operation, that well would have been shut down and a new well would have been developed to replace it. Monthly testing to determine the quantity of fines being removed was to have been required. Paris, ff. Tr. 9900, at 18-19, 24-26, 36-38; Tr. 9814-15 (Kane).

105. Water quality samples were to have been taken annually to determine the concentration of compounds associated with encrustation. Acid treatment of the wells would have been employed, if needed, to remove encrusting minerals in order to prevent a decrease in dewatering efficiency that might result from encrustation of the well screens. Paris, ff. Tr. 9900, at 38-39; Tr. 10,065-67 (Gonzales).

106. Each primary interceptor well was to have been controlled by its own timer for cycling and a low-level cutoff switch to prevent pump damage if unexpected low flow were to occur. Timer settings were to have been determined on the basis of experience with the dewatering system and were to have been adjusted periodically to meet the limiting conditions of the operating technical specifications. The backup interceptor wells and the area wells were to have been automatically controlled by high-water-level and low-water-level switches. Electrical wiring was to have been designed so that a temporary outage of one or more wells would have no effect on the other wells. In the event of loss of power to the system, a separate diesel generator was to be provided to power the interceptor wells. Paris, ff. Tr. 9900, at 21-22; SSER # 2, § 2.4.6.4, at p. 2-10.

107. The first line of interceptor wells and the backup line were to be connected to different header lines so that if some problem developed in the header of the first line, the backup line would have been able to discharge excess ground water through its own header system. In addition, provision would have been made to attach flexible hoses to each well, thus bypassing the header system entirely, if so needed in the event of rupture of an underground header near a dewatering well. Paris, ff. Tr. 9900, at 32-33.

108. The Applicant committed to store on site one complete set of replacement parts for any repair, replacement, or installation which may

be required for a dewatering well during the operating life of the plant (Paris, ff. Tr. 9900, at 36). The Board (at Tr. 9979) questioned whether this was sufficient based on a pipe break scenario which postulates damage to two dewatering wells (*see* Paris, ff. Tr. 9900, at 33). Mr. Paris would recommend that more than one set of replacement parts be stored on site. Although the Staff would have no difficulty with the Board imposing such a requirement, it pointed out that this kind of requirement would not usually be a matter for technical specifications but, rather, would generally be covered by other procedures that the Applicant would maintain. Tr. 9979-80 (Paris); Tr. 10,102-03 (Hood). In view of this approach, and in consideration of the water-level monitoring requirements and the technical specification that the plant be shut down before the ground water rose to a level where a liquefaction hazard existed (Findings 109-110, 113, *infra*), we see no safety reason compelling imposition of a requirement for more than one set of dewatering well replacement parts on site.

109. Six permanent water-level monitoring wells were to have provided continuous recordings of water level during plant operation, and alarms to alert plant personnel to a significant rise in level at any of the wells. Of these six monitoring wells, two each were to have been located in the area of the DGB and the RBA. The remaining two were to have been located between each of those structures and the main recharge area. The Staff position was that the four permanent monitoring wells near the DGB and RBA would provide sufficient information on the ground water level at those structures, but would require additional monitoring of other wells to supplement, and check on, the recording wells. Paris, ff. Tr. 9900, at 22-23, 37, FSAR Fig. 2.4-46 (attached); SSER # 2, § 2.4.6.4, at p. 2-7 (*also see* Fig. 2.1 at 2-2 for plan location of all wells in the permanent dewatering system).

110. The Applicant and Staff each evaluated the impact of various pipe breaks on the ground water levels. A postulated break in the 66-inch cooling pond blowdown line near the service water pump structure would have minimal impact on the dewatering system because this is a low-pressure line and the dewatering system has sufficient capacity to remove all the released water from such a line break. Paris, ff. Tr. 9900 at 33-34; SSER # 2, § 2.4.6.3, at p. 2-7. A postulated break in the Unit 2 circulating water pipe near the DGB was considered. This is a 96-inch line located on natural material just to the east of the DGB. It was calculated that the ground water would rise over a period of about 3.3 days to about elevation 607 feet before the closest permanent area well would have been automatically activated. Operation of one area well would be sufficient to prevent ground water from rising significantly

above elevation 610 feet. While this 607-foot elevation would be just slightly above the 606.5-foot elevation at which plant shutdown would have been initiated, there still would have been time to shut down the plant before elevation 610 feet was reached. Moreover, the analysis was very conservative in that it assumed that 100% of the water flowed into the ground, that plant personnel did not notice the diversion of this water which normally would flow into the cooling pond, that the observation wells in the vicinity failed to alarm, and that all the water flowed towards the DGB. Paris, ff. Tr. 9900, at 34; SSER # 2, § 2.4.6.3, at p. 2-5; Tr. 9938-45 (Paris); Tr. 10,062 (Gonzales). Finally, the effect of a postulated break in the 20-inch condensate water pipe, which is located directly beneath the DGB, was evaluated. Using a simplified analysis, it was conservatively assumed that the entire contents of the condensate water tank (300,000 gallons) were spilled directly beneath the DGB, and that all the water would be contained in this area. It was determined that the ground water elevation would not rise above 610 feet, even if the area wells did not operate. However, there would have been an alarm if the level in the condensate tank dropped below 175,000 gallons. At that point another proposed technical specification would have required plant shutdown unless the low tank level could be mitigated in a given period of time. Tr. 9944-45, 9969-72 (Paris); Tr. 10,063-65 (Gonzales); Tr. 10,064-65 (Hood).

111. An evaluation of the impact of unusually heavy rainfall on the ground water level also was made. Such rainfall could be accommodated by the permanent dewatering system and would not result in the ground water level rising to elevation 610 feet. This evaluation was based on a prediction of the 100-year maximum rainfall. Tr. 9973-75 (Paris); Tr. 10,134 (Gonzales).

112. A recharge test of the dewatered portion of the site was requested by the Staff and conducted in 1982 by the Applicant. The purpose of the test was to verify the time it would take the ground water to rise from elevation 595 feet to elevation 610 feet, the elevation above which a potential soil liquefaction hazard would exist beneath the DGB and RBA as a result of ground water saturation of loose sands in the plant fill. The test was necessary to determine whether there would be sufficient time in the event of total failure of the dewatering system to repair or replace the system or safely shut down the plant. At the time of the recharge test, the cooling pond was full and the plant soils had been dewatered to elevation 595 feet, or considerably below, except for isolated perched water the drainage of which was retarded by impervious soil layers. All pumps were shut off and water levels were allowed to rise nor-

mally for a period of 60 days.[75] The water level rose beneath the DGB, in that time, to about 609-foot elevation (worked out to be about 52 days for a rise from 595- to 610-foot elevation). The rise in water level beneath the RBA was complicated by water leaking from a buried pipe that was not related to the test but which was accidentally ruptured during the period of the recharge test. It was nonetheless possible to estimate that about 40 days would be required for the ground water to rise from 595- to 610-foot elevation beneath the RBA in the event of complete failure of the dewatering system. The Staff estimated rates of water-level rise from the last 2 weeks of the recharge test as being 0.35 ft/day beneath the DGB and 0.41 ft/day beneath the RBA. SSER # 2, § 2.4.6.2, at 2-1 to 2-5; Paris, ff. Tr. 9900, at D1-D5, FSAR Fig. 2.4-58 (attached).

113. A permanent dewatering system technical specification was to have been provided detailing the measures to identify and verify a water-level rise above elevation 595 feet and to initiate repairs or, if the ground water level rose to elevation 606.5 feet, to initiate and coordinate plant shutdown. Based on the last 2 weeks of the recharge test, the Staff found that, with no wells operating, the rate of ground water rise beneath the RBA was about 0.41 ft/day. This was slightly faster than the 0.35 ft/day rate beneath the DGB. Using the faster rate, it would take about 8.5 days for the ground water level to rise from 606.5 feet to 610 feet, the design base elevation to mitigate soil liquefaction. It would have taken about 36 hours to bring the plant to cold shutdown. Thus, there would have been time to shut down the plant before the ground water reached an elevation that would present a liquefaction hazard. SSER # 2, § 2.4.6.2, at 2-4 to 2-5, § 2.4.6.4, at 2-7 to 2-10; Paris, ff. Tr. 9900, at 37; Tr. 9831-32 (Hood).

(2) Effects of Dewatering on Soils (Stamiris Contentions 4.D(1) and 4.D(2))

114. In addition to eliminating or reducing the potential for soil liquefaction, as discussed above, dewatering may have other effects on the engineering characteristics of site soils. Some of these effects may be advantageous while others may be adverse. Dewatering will increase the shear strength of soils which would increase their bearing capacity. Eliminating the lateral force exerted by ground water against underground walls of certain structures would be another advantage of dewatering.

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[75] Dewatering did not actually resume until about 4 weeks after the end of the recharge test. Tr. 9954-58 (Paris).

Potentially adverse effects of dewatering might come from the removal of soil fines, or from the loss of buoyancy of soil particles accompanying removal of the interstitial water and lead to increased compression of the soil. Seismic shakedown is a permanent vertical strain of loose sands related to their densification during earthquake shaking, and which might cause settlement of overlying structures. While not a consequence, strictly speaking, of dewatering, it is a lesser effect that must be considered in lieu of liquefaction of the same sands. The potential for seismic shakedown at the Midland site is governed by the same characteristics of loose sand in the plant fill that caused concern for liquefaction and engendered the need for dewatering (*see supra* Findings 90, 94, 98). Tr. 9212-16, 9814 (Kane); Woods, ff. Tr. 11,549 at 2-6; Hendron, ff. Tr. 8586 at 25, C-10 to C-12; Hendron, ff. Tr. 8675, at 1, 4-8; Tr. 8638-39, 8676 (Hendron).

115. What impact the removal of soil fines would have had on plant soils was not explored in the testimony because both the Applicant's and the Staff's experts agreed that proper discharge-well filter-pack design and construction would obviate the potential cause. The actual tests performed by pumping the dewatering wells and monitoring the content of fines in the discharged water demonstrated that the quantity of fines removed fell within the Staff's acceptance criterion by a considerable margin — less than 2 ppm observed, versus 10 ppm allowed. Monthly monitoring of the discharge from the dewatering wells was to be a requirement during operation of the plant (*supra* Finding 104), and would assure that continued operation of the dewatering system would not remove excessive quantities of soil fines. SSER # 2, § 2.5.4.4, at p. 2-35; Tr. 9814-15, 9828-30 (Kane); Paris, ff. Tr. 9990, at 18-19, 27, 37-38.

116. Dewatering would remove the effect of buoyancy from soil particles, and would hence increase the effective weight of the soil mass. This increase, in turn, would place greater loads on the foundation soils and lead to soil compression.[76] Tr. 9816 (Kane). The effects of the dewatering loads were seen in plots of measured settlement and parallel plots of water-table elevation. As the water table was lowered, the rate of soil settlement, as indicated by the slope of the settlement curve, increased. During the recharge test, some soil rebound was correlated

[76] Soil compression refers to the reduction in vertical height in a soil due to loading. Consolidation of soil is the inelastic portion that is not recovered upon removal of the load. Tr. 20,588 (Kane). The effect of dewatering on soil compression would influence settlement of structures founded on natural soils as well as plant fill. For example, the long-term settlement of the containment buildings, founded on natural soils, was estimated at 2.3 and 2.4 inches, of which 0.6 inch was attributable to the dewatering load (SSER # 2, § 2.5.4.5.2, at p. 2-41).

with the rise of ground water level. The effects were expected. For each of the safety-related structures and underground utilities at the Midland site,[77] the Applicant assessed the additional settlements that would be caused by dewatering, and the Staff was satisfied that they are adequately included in the predicted settlements that were to be used in the structural analyses. Tr. 9816, 9818, 20,535-37, 20,543-45, 20,578 (Kane); SSER # 2, § 2.5.4.5.2, at p. 2-41 (reactor containment buildings only); Staff Exh. 23 ("Diesel Generator Building Dewatering Settlement Report," accompanied by Affidavit of Ralph B. Peck, dated March 4, 1983). For general background, *see also* App. FOF, ¶¶ 122-125 and 137 (DGB), 226-227 (Aux. Bldg.), 261-262 (SWPS), 294 (BWST), 335 (piping), 410 (duct banks).

117. Seismically induced settlements of structures may occur as a result of "seismic shakedown" of loose cohesionless sands in the plant fill. The structures potentially affected would be the DGB and the RBA, as well as the diesel fuel oil storage tanks. The sand bodies subject to shakedown are those that would be potentially subject to liquefaction if not dewatered. The Applicant analyzed the potential additional settlement using conservative earthquake input, i.e., 0.19*g* peak acceleration and 10 cycles of shearing strain reversal, applied to each known sand body capable of affecting a safety-related structure. The seismically induced settlement was derived by summing the potential shakedown for each layer beneath each structure. Dr. A.J. Hendron presented testimony on his analyses of seismic shakedown potential at the DGB and Dr. R.D. Woods presented results of his analyses on the other safety-related structures and buried utilities potentially affected. Dr. Woods estimated that for an SSE of 0.12*g* (as accepted here for the Midland site, *see supra* Finding 71) the shakedown settlement would be about 50% of that determined by him (Woods, ff. Tr. 11,549, at 9). The Staff was in agreement with the magnitude of the settlements and concluded that they are reasonable and acceptable for use in design (Tr. 11,558-59 (Kane)). The seismic shakedown settlement for the DGB was 0.25 inch ± 0.15 inch (Hendron, ff. Tr. 8675, at 1, 8; Tr. 8682-83 (Hendron)) and about ¼ inch or less for the other affected structures (Woods, ff. Tr. 11,549, at 6-9). *See also* Wiedner, ff. Tr. 10,790, at 18-19; Shunmugavel, ff. Tr. 11,997; Shunmugavel, ff. Tr. 12,016, at 5-6.

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[77] While we do not here reach any conclusions on the acceptability of the DGB or its foundation soils, or on the prediction of differential settlement between the main structure of the auxiliary building and the control tower, no unresolved controversy over dewatering effects at those structures exists between the Applicant and Staff. Ms. Stamiris submitted no proposed findings with regard to the technical design of the dewatering system.

III. AUXILIARY BUILDING AND FEEDWATER ISOLATION VALVE PITS

118. Stamiris Contention 4.C(a) asserts:

> Remedial soil settlement actions are not based on adequate evaluation of dynamic responses regarding dewatering effects, differential soil settlement, and seismic effects for these structures:
>
> a. Auxiliary Building Electrical Penetration Areas [EPAs] and Feedwater Isolation Valve Pits [FIVPs].

Prehearing Conference Order, dated October 24, 1980, Appendix at 6-7, as supplemented by Ms. Stamiris' Answer to Applicant's Interrogatories, dated April 20, 1981.

119. The Applicant's testimony on remedial measures for the auxiliary building and FIVPs was presented by a panel consisting of Mr. Edmund M. Burke, Dr. W. Gene Corley, Dr. James P. Gould, Mr. Theodore E. Johnson, and Dr. Mete A. Sozen. Burke, *et al.*, ff. Tr. 5509. The Applicant's witness on seismic shakedown of sands in the plant fill beneath the RBA, control tower, EPAs and FIVPs was Mr. Palanichamy Shunmugavel. Shunmugavel, ff. Tr. 11,997. The Staff panel presenting testimony on the remedial underpinning of the auxiliary building was made up of Messrs. Darl Hood, Joseph Kane and Hari N. Singh. Hood, *et al.*, ff. Tr. 5839. Mr. Frank Rinaldi, of the NRC Staff, gave testimony on structural engineering evaluations of the auxiliary building underpinning design. Rinaldi, ff. Tr. 5944 and ff. Tr. 12,080.

120. The auxiliary building is a large, mainly reinforced concrete building located between the containment buildings to the east and west, and adjacent to the turbine building on the south. The main structure is founded on overconsolidated, hard lacustrine clay, a competent natural soil, at elevation 562 feet, about 73 feet below plant grade. The RBA projects northward about 28 feet and is founded on plant fill at elevation 630.5 feet, about 4 feet below plant grade. The control tower projects southward about 48 feet from the main structure, and the EPAs extend as wings about 90 feet to the east and to the west of the control tower. The control tower and EPAs are founded on plant fill at elevation 609 feet, about 25 feet below plant grade. The FIVPs are structurally isolated, but each is adjacent to the outer end of an EPA wing and to the respective containment building which each serves. The FIVPs are supported by plant fill at elevation 615 feet, about 20 feet below plant grade. The auxiliary building, its control tower and EPAs, as well as the FIVPs all contain safety-related equipment and are required to be designed to Seismic Category I standards. Burke, *et al.*, ff. Tr. 5509, at 7-9, Figs.

Aux-1 to Aux-5; Hood, *et al.*, ff. Tr. 5839, at 4-6, 7-8; Shunmugavel, ff. Tr. 11,997, at 2-3, Figs. 1, 2; SSER # 2, § 2.5.4.1.2, at 2-12, 2-13, Tables 2.2, 2.3.

121. The Applicant undertook a soils exploration program in 1978 following discovery of excessive settlement of the DGB. Three borings were taken in the vicinity of the RBA on the north side, and twelve borings were taken along the south side in the vicinity of the control tower, the EPAs and the FIVPs. Inadequately compacted soils that could lead to differential settlement were found in the backfill supporting the EPAs and the FIVPs. An early proposed remedial "fix," subsequently abandoned, would have supported the extreme ends of each EPA by caissons to control their differential settlement. Burke, *et al.*, ff. Tr. 5509, at 10-11, Figs. Aux-6 to Aux-8; Hood, *et al.*, ff. Tr. 5839, at 8-11, 13-14; Tr. 5856-57 (Kane); Tr. 5747-49 (Johnson). *See also* Staff FOF, ¶ 215.

122. In its evaluation of the proposed plan for caisson support of the extreme ends of the EPAs, the Staff determined that the plan did not adequately address the loads it would add to the control tower at the center. In the Staff's view, the added loads likely would have caused overstressing of the plant fill supporting the control tower under some loading conditions (e.g., dynamic bearing capacity). This problem was to have been solved by the eventually approved plan which required underpinning the control tower and EPAs with new foundation walls that would extend down to the hard lacustrine clay at elevations 562 feet and 571 feet, respectively. Hood, *et al.*, ff. Tr. 5839, at 13-14; Rinaldi, ff. Tr. 5944, at 4; Burke, *et al.*, ff. Tr. 5509, at 1, Figs. Aux-23, Aux-38; Tr. 5873-78 (Singh). The proposed remedy for the FIVPs, i.e., removal of supporting plant fill and replacement by competent nonliquefiable material, was not changed. SSER # 2, § 2.5.4.4.1, at p. 2-17; Burke, *et al.*, ff. Tr. 5509, at 13-14; *see infra* Finding 144, *re* proposed remedial action for the RBA.

123. The Staff's concern over the adequacy of the fill foundation soils supporting the control tower was engendered in part by the differential settlement of the south end of the control tower that had occurred, and by the location of cracks in the auxiliary building. The presence of a 1-foot void between a concrete mudmat and the underlying plant fill, encountered in one of the exploratory borings, also contributed to the Staff's concern over the adequacy of the plant fill beneath the control tower. While the measured differential settlement of the south end of the control tower had been slight (on the order of ¼ inch between

July of 1978[78] and August of 1981), the Staff believed it was reasonable to expect that it might have been as much as 0.5 to 1 inch, or more, since the beginning of construction. Cracks observed in the auxiliary building concrete, including some through-cracks, were regarded by the Staff as possible manifestations of distress. Tr. 5880-82 (Kane); SSER # 2, § 2.5.4.4.1, at p. 2-17, § 2.5.4.5.2, at p. 2-40; Burke, *et al.*, ff. Tr. 5509, at Fig. Aux-8-A; Hood, *et al.*, ff. Tr. 5839, at 9.

124. The Applicant, on the other hand, regarded the cracking in the auxiliary building as primarily caused by constrained volume changes in the concrete due to temperature changes and drying shrinkage during curing. The Applicant's witnesses recognized the possibility that there may have been some very slight structural deformation associated with rotation of the auxiliary building to the south during settlement. However, their analyses of the locations, patterns and widths of cracks did not indicate to them that the primary cause of cracking was differential settlement, nor that there was evidence of any structural distress, or even structural significance, to be found in the cracking. Burke, *et al.*, ff. Tr. 5509, at 11-12, Figs. Aux-9 to Aux-21, Appendix A.

125. As to the *cause* of the cracking in the auxiliary building, the Staff was unwilling to accept a determination that all of the cracks stemmed from shrinkage of the concrete. (*See* first conclusion, Burke, *et al.*, ff. Tr. 5509, at A-15.) The Staff required an evaluation of the *effect* of the cracks on the Seismic Category I structures supported fully or partially by plant fill, and found that the Applicant's analyses were acceptable. The results of the Applicant's analyses showed that existing cracks do not significantly affect the strength in tension, compression, and shear of properly reinforced concrete members. The results further showed that, provided the structure has been proportioned and detailed to resist design load combinations, reinforced concrete structures will develop their design strength, even if they have "precracks." Crack mapping, repair and monitoring programs were instituted to prevent degradation of the structures during construction of the underpinnings and during the operating lifetime of the plant if construction were to be completed. SSER # 2, § 3.8.3.5, at 3-27 to 3-29; Burke, *et al.*, ff. Tr. 5509, at 11-12, Figs. Aux-9 to Aux-21, Appendix A.

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[78] The Applicant stated that a Foundation Data Survey Program was established in May 1977, with the attachment, at that time, of a settlement marker to *one* corner of the auxiliary building (Burke, *et al.*, ff. Tr. 5509, at 10). Except for a general reference to the FSAR and to asserted use of the observation, the Board found no reference that provided or used the actual elevation data from the marker in the evidentiary record (*cf.* App. FOF, ¶ 216, Burke *et al.*, ff. Tr. 5509, at 56).

126. Underpinning the control tower and EPAs and replacement of the plant fill beneath the FIVPs were selected as the best remedial measures for assuring proper foundation support for the southern portions of the auxiliary building. If properly designed and executed, this approach would cause the foundation loads of these overhanging structures to be borne by the hard natural clay layer and eliminate those concerns about differential settlement arising from the unsatisfactorily compacted plant fill. Potentially, it also would have reduced, or effectively eliminated, stresses in the existing structures that might have been induced during underpinning construction or stresses possibly indicated by the presence of cracking ("precracking"). Burke, *et al.*, ff. Tr. 5509, at 12-14, 31-32, 37, 39-44, 50-53, 56-59, A-12 to A-15; SSER # 2, § 2.5.4.4.1, at 2-16 to 2-23.

127. Following a design audit conducted on September 14-15, 1983, the NRC Staff issued a Board Notification (BN 83-174) concerning soils remedial activities potentially at issue in these proceedings. The Staff cited three open items from the audit findings (items "d," "e," and "g") which it believed pertinent to soils remedial design issues. Open item "d" pertains to the Applicant's method of analyzing differential settlement between the main auxiliary building and the control tower and concerned the baseline length over which effects of a fixed differential and, hence, resultant structural stresses, were to be calculated. This item relates to the Stamiris Contention 4.C(a) allegation on inadequacy of the underpinning design to account for the effect of (future) differential settlement, as well as to the validity of acceptance criteria to be provided to the Staff (as cited in the Modification Order). Open items "e" and "g" call into question the permissible limits of upward movements on the structures during jacking operations, whether residual stresses in the building can be removed during jacking, and how the residual stresses would be treated in the final design analysis load combinations. Items "e" and "g" relate to questions of validity of acceptance criteria, but only indirectly, if at all, to the design adequacy aspects of Ms. Stamiris' contention. Nevertheless, the effect is the same, and the Board makes no findings at this time on any of the three open items referred to above from BN 83-174.[79] Board Notification Regarding Midland Auxiliary Building Underpinning (BN 83-174), dated November 21, 1983, transmitted to Board and Parties by memorandum from

[79] The other open items from the Staff's design audit, items a, b, c and f, were not identified as subjects of BN 83-174. BN 83-174 has been provided to all parties but has not been introduced into evidence at this time. We rejected Ms. Stamiris' motion to reopen the record on matters covered by BN 83-174 as premature — *see supra* p. 37.

Thomas M. Novak of the NRC Staff (hereinafter "BN 83-174"); Modification Order at 3.

128. During hearings on quality assurance/management attitude issues, Dr. Ross Landsman, a soils engineer with the NRC Staff, volunteered that in his opinion the design of the auxiliary building, and the SWPS, whereby the main part of the structure was founded on hard soil and another part was founded (at a higher elevation) on plant fill, constituted a design deficiency. *See* Figure 5, *supra* p. 97. He asserted that this design had an inherent potential for developing problems as a result of differential settlement. The "overhanging" part, resting on [a thick section of] backfill, could act as a cantilever projecting from the main structure if the backfill settled more than anticipated in the design. This would cause overstressing of the structure in the region where the two parts of the building connect. Dr. Landsman believed that, even if the backfill had been compacted as designed, the configuration would still have presented a problem at the Midland plant. However, similar design configurations have been accepted not only at the Midland plant (at the construction permit stage) but at other plants; the configuration violates no regulatory requirements and, if properly built, would be licensable. Dr. Landsman testified that differential settlement also was a problem at at least one of the other sites (South Texas), but he did not know if the differential settlement there was attributable to design of the foundations or to the compacted fill. Because this condition is what the underpinning was principally intended to remedy, the potential safety problems to which the cantilevered design might give rise would be adequately resolved for the Midland structures. We therefore need not determine whether or not the original design practice is generally acceptable. We are therefore not doing so — but see our recommendation in the Opinion section, *supra* pp. 93-94. Tr. 15,060, 16,316-17, 16,319, 16,392-99, 16,404-05, 16,505-09, 16,589-91, 16,816 (Landsman); Tr. 20,218-43, 20,281-88 (Thomas).

129. The underpinning wall for each electrical penetration area was to extend down to undisturbed lacustrine clay at about elevation 571 feet. Each wall would have a minimum thickness of 6 feet with an increased thickness at the base to provide greater soil bearing area. The thickness of the base would vary as the north face of each wall curves about the containment, leaving a 4-foot gap for compacted sand fill. Burke, *et al.*, ff. Tr. 5509, at 12, Figs. Aux-22 to Aux-29. (In its responsive FOF, ¶ 219, the Staff advised the Board that the Applicant was planning to use lean concrete instead of sand to fill the 4-foot gap left by the curving of the walls around each containment. The Applicant's Reply FOF, ¶ 219, indicated that any change would be submitted for Staff ap-

proval pursuant to the Work Authorization Procedure adopted as a result of LBP-82-35, *supra.*)

130. The underpinning wall for the control tower would extend down to undisturbed glacial till at elevation 562 feet and consist of 6-foot-wide by 3-foot-long piers (which provide support during construction operations) and closure portions which interconnect the individual piers to provide a continuous permanent underpinning wall. The piers and wall sections were to be belled out to 14 feet wide at the base to provide greater soil bearing area. The underpinning walls would have formed a box in conjunction with the existing south foundation wall of the main portion of the auxiliary building to which they were to be attached. The control tower underpinning walls would also have been attached to the underpinning walls of the electrical penetration areas. Burke, *et al.*, ff. Tr. 5509, at 12-13, Figs. Aux-22 to Aux-25.

131. The FIVPs were to be supported in a different manner than the control tower and EPAs. The existing backfill under the FIVPs was to be removed and replaced with well-compacted granular material to a suitable height below the existing valve pit mat. The new granular backfill was to be compacted to 95% maximum dry density as determined by ASTM Test D-1557 or ASTM Test D-2049, whichever results in the greater maximum dry density. A reinforced concrete slab would have been cast on top of the new fill and jacks placed between the slab and the original mat to precompress the new fill. After precompression of the fill was completed, the space between the slab and the original mat was to be filled with grout and concrete. A beam-and-tie system which provides temporary support for the FIVPs was installed for their support during the underpinning operation. *Id.* at 13-14, Figs. Aux-21, Aux-31; SSER # 2, § 2.5.4.4.1, at p. 2-17.

132. In order to accomplish the underpinning of the control tower and EPAs and the removal and replacement of the soil backfill under the FIVPs, access shafts were dug on the west and east ends of the affected area. These shafts were located immediately to the north of the turbine building and immediately to the west and east of the respective FIVPs. From these access shafts, tunnels were excavated which allowed workers to drift under the turbine building and, as the work progressed, under the EPAs, FIVPs and control tower. The work was to progress in a stepwise fashion, tunneling far enough to construct the first temporary supports, constructing those supports, tunneling far enough to accomplish the next part of the construction, constructing it and so on. Burke, *et al.*, ff. Tr. 5509, at 14-28, Figs. Aux-22 to Aux-26, Aux-30; SSER # 2, § 2.5.4.4.1, at 2-17 to 2-23; *see also* Tr. 5532-72 (Burke).

133. Because excavation under and alongside existing structures was necessary to accomplish underpinning efforts, the construction procedures to be used included measures to support the soil adjacent to all excavations and to provide temporary support for the affected structures during the construction process. In addition to the piers which were to become part of the foundation walls, and the beam-and-tie system to support the FIVPs, the EPAs were to be supported by a grillage system of beams and cross-beams supported at one end by steel posts resting on a projection of the containment structure and at the other end by a constructed pier (Pier M) bearing on the undisturbed natural soil (Tr. 5542-46 (Burke)). The procedures and sequence of construction of the underpinning operation for the auxiliary building and FIVPs are explained in detail by one of the Applicant's witnesses, Mr. Burke (at Tr. 5532-72), and in the prepared testimony of the Burke panel, ff. Tr. 5509, at 14-28. *See also* SSER # 2, Appendix I.

134. Temporary post-tensioning ties were installed to the upper part of the east-west wall of each EPA on either side of, and through, the control tower. These ties served to compensate for loads induced by loss of buoyancy under the EPAs resulting from construction dewatering of the foundation soils (*see infra* Finding 137). Burke, *et al.*, ff. Tr. 5509, at 16, Fig. Aux-27 (*cf.* SSER # 2, § 3.8.3.1, at p. 3-6).

135. During underpinning construction, the ground water level was lowered in the area of the southern end of the auxiliary building to about 565-foot elevation (30 feet below the permanent dewatering level). A freezewall or freeze-curtain dam, in conjunction with the existing west cutoff dike and the impermeable clay beneath the containment buildings, was created in order to maintain relatively dry working conditions. Burke, *et al.*, ff. Tr. 5509, at 16-18, 55; Fig. Aux-28; Tr. 5511-18 (Burke).

136. The freezewall was emplaced by drilling a line of closely spaced bore holes and circulating a coolant at low temperatures through pipes in the boreholes. The coolant froze water in the soil in a narrow strip along the line of boreholes and from elevation 610 feet down to the undisturbed natural soil (lacustrine clay). The frozen soil acted as a dam which minimized seepage of ground water into the excavations from surrounding areas. Breaks in the freezewall were left in the vicinity of buried utilities to prevent possible damage that might have resulted in heaving of the utility lines or ducts where they were crossed by the freezewall. Seepage through the freezewall at these breaks was to have been controlled by excavating and backfilling with impermeable materials and/or by temporary dewatering wells installed in their vicinity.

Burke, *et al.*, ff. Tr. 5509, at 16-18, 55, Fig. Aux-28; Tr. 5511-18 (Burke); SSER # 2, Appendix I at I-1 to I-2; Tr. 22,106-07 (Wheeler).

137. The Applicant took into account the loads resulting from the lowered ground water elevations to be maintained by permanent dewatering and by temporary (construction) dewatering in its design of the remedial soils measures for the control tower, electrical penetration areas and FIVPs. The NRC Staff verified that these loads were considered in the design of the remedial soils measures and that, with the exceptions noted in BN 83-174 in regard to differential settlement between the main part of auxiliary building and the control tower, the Applicant's design loads with respect to effects of dewatering were acceptable. Rinaldi, ff. Tr. 12,080 at 2-3; Tr. 12,101-03 (Rinaldi); Burke, *et al.*, ff. Tr. 5509, at 16-18, 55-57; Board Notification BN 83-174.

138. The natural clay soil which was to provide foundation support for the underpinning of the control tower, EPAs and FIVPs is the same as that supporting the containment buildings and main part of the auxiliary building. All parties and the Board in these proceedings often referred to all the natural soils at the Midland site simply as "till" or "glacial till," when, in fact, glacial till actually occurs only in limited areas of the plant site. The natural soil in the vicinity of the auxiliary building is a very stiff to hard clay of lacustrine origin which has been overconsolidated by glacial ice (probably many hundreds of feet thick) that produced a compressive burden on the clay greatly in excess of the foundation load that will be exerted by the Midland Plant structures. In determining settlement, an overconsolidated or precompressed clay will have no "virgin" compression and the elastic modulus (Young's Modulus) can be used to calculate the elastic recompression of the soil. Jacking loads were to have been maintained until pier settlements indicated that the full elastic recompression had been attained. Secondary, long-term settlements can be computed separately by extrapolating observed secondary compression or by using coefficients of secondary consolidation. The settlement calculated from secondary consolidation would be added to the initial settlement from elastic recompression to predict total settlement of the piers. Future settlement of structures resting on the piers would be predicted from the secondary consolidation of the clay, because of the preloading procedure. Tr. 5873-79 (Singh), *amending* Hood, *et al.*, ff. Tr. 5839, at 15-16; Burke, *et al.*, ff. Tr. 5509, at 50-51, 53-55, Table Aux-4; *see also* Staff FOF, ¶ 219 (and authorities there cited) for clarification of natural soils terminology.

139. Hydraulic jacks placed at the tops of the piers were to be used to impose predetermined pre-loads on the underpinning supporting soil

before the control tower and EPAs were finally fixed to the underpinning. After each increment of jacking, sets of steel plates and wedges adjacent to the jacks were to be driven tight to prevent settlement of the structure when jacking pressures were removed. The structural motions were to be monitored to assure that excessive stresses were not developed in the structure during the jacking process. Stresses in the piers were to be monitored by means of Carlson gages embedded in the top and bottom of the pier concrete or by load cells at the top of the pier. Pier vertical deflections were to be monitored to ensure that primary compression (elastic recompression) of the supporting clay was attained, and predicted future long-term settlements would be checked by extrapolation of the trend of the measured secondary settlements while the jacks were still active Burke, *et al.*, ff. Tr. 5509, at 22-34, 36-37, 53-55; SSER # 2, § 2.5.4.6.1, at 2-44 to 2-46, 2-48 to 2-50; § 3.8.3.a, at 3-6 to 3-9.

140. During underpinning construction the Applicant conducted a pier load test to evaluate the soil parameters and settlement response of the lacustrine clay. The test procedure, which was found acceptable to the Staff, was to load pier W-11 by jacking to 50% of the maximum load predicted throughout the operating life of the plant, unloading to 25%, and then raising to 130% of the maximum predicted load. After completion of the test the load was lowered to the design jacking load (SSER # 2, § 2.5.4.6.1.2, at p 2-51). The pier load test did not produce expected results in that the Carlson stress meters on the pier indicated that the load was not reaching the bottom of the pier (Tr. 14,370-71, 14,664 (Landsman)). Also, settlement of pier W-11 during (or subsequent to) the test was apparently more than predicted (Tr. 16,601-05 (Landsman)). As a result, the Applicant reevaluated the structure using an assumed settlement of twice the originally calculated amount, equivalent to an assumption of a soil modulus of one-half the originally estimated value. The purpose of the reanalysis, according to the Applicant, "was to ensure that even if the soils conditions were as poor as the tests indicated, the building would perform satisfactorily over the life of the plant" (Tr. 17,170 (Mooney)). This reanalysis was the subject of the NRC design audit that resulted in the issuance of BN 83-174 (*supra* Finding 127). The Board notes that the Applicant's assumption of a reduced elasticity modulus in its reanalysis was derived from an option provided to it by the Staff following unsatisfactory completion of the pier W-11 load test (Tr. 16,604-05 (Landsman)). The assumption of a reduced soil modulus does not equate to a reduction of bearing capacity by one-half, as alleged in Ms. Stamiris' FOF "13," item (1) at 5. *See*

Burke, *et al.*, ff. Tr. 5509, at 51-53, for a discussion of "ultimate bearing capacity" and the determination of the "bearing capacity factor" for the clay; and, *id.* at 53-55, for a discussion of the settlement estimates using the elastic method for estimating settlement of overconsolidated clay.

141. The Applicant took into account loads which would be imposed by postulated seismic events as well as flooding events in developing and evaluating the design of the remedial soils measures for the control tower, EPAs and FIVPs and, in so doing, complied with the requirements of SRP §§ 3.7.2, 3.8.3 and 3.8.5. Rinaldi, ff. Tr. 12,080, at 6-8; SSER # 2, § 3.8.3.1, at 3-10 to 3-11; Burke, *et al.*, ff. Tr. 5509 at 46, Appendix B. *See also supra* Findings 19-79, for general background on seismic issues. However, the seismic evaluation is subject to the resolution of the design deficiency identified in BN 84-115 (*see supra* Findings 88-89) and our findings on seismic design are limited by this open item.

142. Because the SSRS was not yet agreed upon when the design of the remedial soils measures was developed, the Applicant used loads equal to 1.5 times the loads which would result from the FSAR SSE in evaluating the design of the remedial soils measures for the control tower, electrical penetration areas and the FIVPs. Subsequent analysis by a consultant hired by the Applicant and an audit of the Applicant's design calculations by the NRC Staff determined that loads equal to 1.5 times FSAR SSE loads are conservative in relation to loads which would result from the now-agreed-upon SSRS. Tr. 6004-28, 6038-41 (Kennedy); Rinaldi, ff. Tr. 12,080, at 7-8; Tr. 12,130-31 (Rinaldi); *see also supra* Findings 77-79, on seismic issues.

143. The Applicant analyzed the potential for seismic shakedown of loose sands in the fill to affect the performance of Category I structures. However, because the replacement fill under the FIVPs was to be compacted to a 95% maximum dry density and all of the underpinning was to be founded on the natural hard clay, like the main portion of the auxiliary building, seismic shakedown is a potential concern only with respect to the RBA portion of the auxiliary building. The Applicant evaluated the seismic shakedown effects for the railroad bay and liquid radwaste areas and determined that, even in the event of an earthquake with peak ground acceleration of 0.19*g*, settlement of no more than approximately 0.25 inch would occur. This amount of settlement would not affect the integrity of the auxiliary building. Shunmugavel, ff. Tr. 11,997, at 3-7; Woods, ff. Tr. 11,547, at 6; Tr. 12,004-11 (Shunmugavel).

144. The Applicant and the Corps of Engineers, for the NRC Staff, conducted independent liquefaction analyses for the Midland site. Insofar as they apply to the underpinned auxiliary building and the FIVPs,

these studies indicated that a potential for liquefaction would remain in the plant fill soils only beneath the RBA portion of the auxiliary building. By lowering and maintaining the ground water elevation in this area to below elevation 610 feet, the Applicant's permanent dewatering system would eliminate concerns about soil liquefaction potential beneath the RBA. The natural hard clay beneath the auxiliary building is not liquefiable. Therefore the underpinning and excavation-and-backfill measures for the control tower, EPAs and FIVPs would eliminate any concern, if it existed, for potential soil liquefaction in these areas. In carrying out its liquefaction analysis, the Corps of Engineers postulated a seismic event with peak ground acceleration of 0.19*g*, which is more severe than the SSE for the Midland site determined during the course of these proceedings. SSER # 2, § 2.5.4.5.5, at 2-42 to 2-44; Woods, ff. Tr. 9745; *see also supra* Findings 90-93, 97 on soil liquefaction potential. The Board concludes that there is an acceptable margin of safety against liquefaction of soil beneath the RBA, provided the ground water in that area is maintained below elevation 610 feet.

145. Because of the possibility of structural movement as a result of the excavations alongside and under existing structures necessary for construction of the remedial soils measures for the control tower, EPAs and FIVPs, the Applicant installed extensive instrumentation to monitor any absolute or relative movement which might occur.[80] For a detailed description of the instrumentation, places of installation and movements measured, *see* Burke, *et al.*, ff. Tr. 5509 at 29-34, Fig. Aux-36; SSER # 2, § 2.5.4.6.1, at 2-44 to 2-49; Tr. 9400-05 (Krause).

146. The primary monitoring system consists of a network of state-of-the-art electronic measuring devices which were to be read by computer every hour and which were to be attended by a technician 24 hours a day. Tr. 9400-03 (Krause). At every point where an electronic device is installed there is also installed a mechanical gauge which does not depend on the electricity to operate. The mechanical gauges would be used to cross-check the electronic readings and would serve as a backup system in the event of a power outage. Tr. 9404-05 (Krause). All the instrumentation was installed away from the immediate area of any construction activities and all the measuring devices were in metal

[80] This monitoring of structures during underpinning construction activities addressed concerns expressed by the Board to the effect that:

> (1) the system for detecting structure movement be reliable as well as accurate so that large data gaps do not occur or instruments get covered up with sand; (2) the plan for arresting structural movement, if it should occur, is adequate; and (3) there is sufficient clearance between the turbine building and the auxiliary building, after taking into account any settlement of the buildings, so that the two buildings would not collide during an earthquake.

Tr. 7122-28.

cases so they should not become covered with sand or suffer degradation due to environmental conditions. Tr. 9405 (Krause). Together the mechanical and electronic devices would provide a reliable and accurate monitoring system for detecting any structural movement and provide reasonable assurance that no significant data gaps would occur. Tr. 9404-05 (Krause); R. Cook, *et al.*, ff. Tr. 11,391, at 3-4, Attach. 5, at 4. Also, extensometers were installed to monitor strains that might occur in certain walls, and a crack-monitoring program was initiated to monitor development of any new cracks or changes in the width of already-mapped cracks. Tr. 5521-26 (Burke); Tr. 9413-14 (Shunmugavel); Tr. 9549-50 (Shunmugavel, Boos, Burke).

147. The computer took hourly readings of all the instruments monitoring structural movement and was set to sound an alarm and immediately print out the data it had collected if an alert or action level were reached. In the event an action level were reached, the NRC Staff was to be notified. An NRC Staff test verified that the computer did sound an alarm and print out collected data when displacement exceeding the alert level was recorded by one of the instruments. Tr. 9400-04 (Krause); R. Cook, *et al.*, ff. Tr. 11,391, at 3-4; Tr. 11,396-97 (Landsman); Tr. 9412 (Boos).

148. The Applicant and the NRC Staff agreed on alert and action levels for structural movement which, if reached, would require that appropriate procedures be followed. The action levels for the auxiliary building were arrived at by analyzing the structure to determine what would constitute tolerable deflections. Once these were calculated and the action levels were set, with the concurrence of the NRC Staff, half the action level would generally be used as the alert level. The action levels for deflection of the auxiliary building are based on a very conservative analysis of what that structure could tolerate. R. Cook, *et al.*, ff. Tr. 11,391, Attach. 2 (Bechtel Specification C-200); SSER # 2, § 2.5.4.6.1.2, Table 2.7, at p. 2-49; Tr. 9413-14 (Shunmugavel).

149. Any movement the monitoring system detected would have been analyzed and appropriate steps would have been taken in response to that movement. In response to any movement trends in the monitoring record which suggest that an alert or action level might be reached, the applicant would have taken steps to arrest the movement before an alert or action level was reached. The primary method which would be used to arrest structural movement would be to jack additional loads into the existing piers and underpinning. However, there were contingency plans for installing additional temporary supports in those instances when the jacking would not be relied upon. If appropriate, all work would be stopped in the area of the movement. Tr. 9406-08

(Burke, Boos); 9634-37 (Poulos); Tr. 11,392 (Landsman); *see also* R. Cook, *et al.*, ff. Tr. 11,391, Attach. 2.

150. The Applicant performed an analysis of how much space is needed between the nonsafety-related turbine building and the safety-related auxiliary building at various elevations in order to ensure that these buildings do not come in contact with each other during an earthquake. Calculations of the maximum amount of deflection of each of these buildings during an earthquake determined that at all elevations there is significantly more space available between the building than the combined amount of deflection of both buildings. Instrumentation was installed by the Applicant to measure relative horizontal displacement between these two buildings to assure that settlement rotation during underpinning activities does not reduce the existing clearance to a point where the buildings would interact during an earthquake. Thus, there is reasonable assurance that the turbine building and the auxiliary building would not impact during an earthquake as large as the SSE determined during the course of these proceedings. Tr. 9416-22, 9621-23 (Shunmugavel); Tr. 9608-21, 9626-29 (Rinaldi); *see also* App. Exh. 27.

151. This Board finds that the concerns expressed in Stamiris Contention 4.C(a) have been adequately addressed, except with respect to the soil spring constants to be utilized in a seismic margin review. The Applicant at this time has adequately evaluated and taken into account during design of the soils remedial actions the responses regarding dewatering effects and (except as noted below) seismic effects, whether static or dynamic, for the auxiliary building electrical penetration areas and feedwater isolation valve pits. However, in the absence of a complete record on resolution of open issues described in Board Notifications BN 83-174 and BN 84-115, as discussed, *supra*, in Findings 127 and 88-89, we make no finding on the adequacy of the design of the remedial action to account for effects of differential settlement between the main portion of the auxiliary building and the control tower; our findings concerning the conservatism of the soil spring constants to be used in a seismic margin review of the auxiliary building structures are limited to the *nominal* value of such constants (and are subject to resolution of the reported design deficiency).

IV. SERVICE WATER PUMP STRUCTURE

152. The service water pump structure (SWPS), which houses the five pumps and support equipment for the service water system, is a Seismic Category I structure, located at the northwest bank of the return

leg of the cooling pond, adjacent to the circulating water intake structure (CWIS) and the Seismic Category I retaining wall of the cooling pond. It is a rectangular, reinforced concrete building with upper and lower sections of different dimensions. The lower section is approximately 72 feet long and 86 feet wide. Its base slab is supported on undisturbed glacial till at elevation 587 feet. The upper section is 106 feet long and 86 feet wide. This size difference results in an overhang at the north end of the upper section, resting on soil. Excavation of the natural clay material left a generally triangular (or trapezoidal) volume under the overhang to be backfilled. Thus the overhang was to be supported by this volume of fill as well as the unexcavated natural material above the undisturbed glacial till layer supporting the main part of the SWPS at elevation 587 feet. Boos, *et al.*, ff. Tr. 9490, at 1-3, Figs. SWP 2-4; Tr. 9728-29 (Hood); SER, § 1.12.7, at p. 1-23; Tr. 9536-41 (Boos); App. Exh. 28; SSER # 2, Fig. 2.8; *see* Figure 5, *supra* p. 97.

153. To evaluate the backfill under the overhang portion of the SWPS, eleven soil borings were taken — two inside the SWPS and nine in the surrounding area. These borings indicated that some localized areas of the soil backfill underneath and adjacent to the overhang portion of the SWPS had not been sufficiently compacted. The inadequately compacted fill revealed by the borings, however, has not caused the SWPS to undergo any unusual settlement, or to experience any significant structural distress. A Foundation Data Survey Program was established by the Applicant in May 1977 to monitor settlement of Seismic Category I buildings. Pursuant to this program, settlement markers were attached to the four corners of the SWPS by the Summer of 1978. In addition, six construction survey control points were installed a short time after concrete placement. Monitoring of the settlement markers and the survey control points has shown that the SWPS has been very stable, with a maximum north-south differential settlement of 0.25 inch. Settlements predicted by the Applicant after completion of the underpinning wall of the SWPS overhang, relative to the portion currently on the till, are 0.1 to 0.2 inch. The Staff considers these estimates of differential settlements for the underpinned SWPS reasonable and acceptable. Boos, *et al.*, ff. Tr. 9490, at 3-5; Tr. 9517-18 (Boos); SSER # 2, § 2.5.4.5.2, at p. 2-41; Tr. 9737-38 (Kane).

154. In December 1978, the Applicant instituted a crack-mapping program for all Seismic Category I buildings founded on plant fill. Several crack mappings of the SWPS were conducted pursuant to this program. The Applicant and Staff reached different conclusions on the reasons for cracks. Dr. W. Gene Corley, the Applicant's expert, concluded that the primary reason for the cracking was restrained volume changes

that occur during curing and drying of concrete. Although he could not completely rule out the possibility that stresses due to differential settlement contributed to some degree to the observed cracking, Dr. Corley indicated that the observed crack patterns do not support the conclusion that stress due to differential settlement was a primary cause of cracking. Dr. Corley observed no evidence of structural distress. On the other hand, the Staff noted the presence of some cracks at locations where one would expect them to occur if caused by differential settlement. Accordingly, in assessing the effects of cracks, the Staff directed its attention to determining whether the cracks significantly diminish the strength of the structure. The Applicant has shown that there is no such diminution in strength. A program for crack monitoring (and repair where appropriate) has been agreed to and found acceptable by the Staff. *See* discussion, *infra* Finding 163. The Staff concluded that, once concerns about future differential settlement were addressed by the remedial measures, it was no longer necessary to address further the reasons for the cracks. Dr. Corley agreed.

While the observed settlement of the SWPS and an analysis of the observed cracks in the SWPS indicate that the SWPS has not suffered significant structural distress to date, the Applicant elected to underpin the overhang portion of the SWPS in order to ensure long-term foundation stability and to allay concerns about future differential settlement due to the pockets of compressible backfill discovered under the overhang portion of the SWPS. Burke, *et al.*, ff. Tr. 5509, at 11; Corley, ff. Tr. 11,204, at 11-29 (crack mapping), 29-34 (crack significance), and 34-40 (crack monitoring); Tr. 9721 (Rinaldi); SSER # 2, § 2.5.4.4.1, at p. 2-23, § 3.8.3.5, at 3-27 to 3-29; Corley, ff. Tr. 11,206, at 1-3 and Attach. 1; Boos, *et al.*, ff. Tr. 9490, at 6; Tr. 18,483-84 (J. Cook); Tr. 2743-46 (Hood); Tr. 9738 (Kane).

155. The underpinning design for the SWPS consists of a continuous perimeter underpinning wall beneath the north end of the SWPS. The reinforced concrete wall was to form a box structure beneath the overhang, connected to the sides of the lower portion of the existing structure, and extending from the upper foundation slab to undisturbed glacial till at approximately elevation 587 feet. The completed underpinning wall would thus provide a structural foundation resting on undisturbed glacial till. *But see infra* Finding 158. In order to construct the underpinning for the SWPS, an access cofferdam was to be constructed to provide access for workers and equipment. It was to be excavated in two stages using soldier piles, tubular steel lagging and wales to ensure proper support for the adjacent soil. Initially it would be excavated, adjacent to the SWPS, to elevation 618 feet to permit installation through ap-

proach pits of the piers at the corners of the SWPS. Then the cofferdam would be lowered at the northwest corner to elevation 609 feet to provide access for excavation of a tunnel beneath the west wall of the SWPS. A tunnel was planned to provide access for constructing the west underpinning wall because of the location of the CWIS. All of the underpinning under the north and east walls of the SWPS would be constructed from elevation 618 feet by means of approach pits from the access cofferdam. Boos, *et al.*, ff. Tr. 9490, at 6-9; SSER # 2, § 3.8.3.2, at p. 3-15; Tr. 5534-36 (Burke).

156. Construction of the underpinning made it necessary to lower temporarily the ground water table, and construction dewatering wells were to be installed in the vicinity of the SWPS for this purpose. Operation of these wells would maintain the ground water level 2 feet below the lowest point of any existing excavation during the construction of the SWPS underpinning. To offset any loss of buoyancy force during the construction due to temporary dewatering, post-tensioning ties were installed along the tops of the east and west exterior walls of the SWPS in November 1981. These ties, which consist of two tendon groups on each side of the building, apply a compressive force of approximately 500 kips (kilo-pounds) to the upper portion of the east and west exterior walls. Boos, *et al.*, ff. Tr. 9490, at 8 and 10; SSER # 2, § 2.5.4.6.1.2, at p. 2-51; Tr. 9515-17 (Shunmugavel).

157. It was planned that the construction of the underpinning progress in stages. The principal consideration in the first stage of construction was to provide initial support for the north end of the SWPS in order to compensate for the possible loss of support under the base slab caused by the underpinning operations and further to counteract any loss of buoyancy force. After completion of the first stage, the rest of the piers would be constructed in a designated sequence. A typical pier would be 5 feet long, 4 feet wide and 30 feet deep. The piers along the north wall would be belled to 6 feet wide at the bottom. Shear keys and reinforcement would be used so that the individual piers, though cast separately, would form one continuous wall upon completion. Boos, *et al.*, ff. Tr. 9490, at 9-15 and Figs. SWP 11-13; SSER # 2, Fig. 2.9, at 2-27 to 2-30.

158. It was expected that all the piers would be founded on undisturbed glacial till which would have been inspected and accepted as adequate by a geotechnical engineer before each pier was cast. It is possible, however, that some pockets of alluvial sand might be encountered at the 587-foot elevation. If alluvial sand were encountered at the base of any of the piers, it would be removed if the pocket were shallow (less than 18 inches deep); however, if it were deep, it would have been accepted

as an adequate foundation material if undisturbed. The alluvial sand found so far has exhibited a higher median blowcount than the undisturbed glacial till and therefore would provide an adequate foundation. A lean concrete working mat was to be cast on top of the inspected and accepted soil to ensure that it remained undisturbed throughout the casting of the pier. A load test of pier 1E at the SWPS was to be performed as was done at the auxiliary building; i.e., using an initial loading of 130% of the maximum predicted bearing pressure, eventually reduced to the design jacking load. The Staff found this procedure acceptable. However, at the SWPS an additional pier would have been load-tested if the bearing level for any of the piers were on the dense sandy alluvium rather than the hard sandy clay fill.[81] Boos, *et al.*, ff. Tr. 9490, at 11-13 and 29-32; Tr. 9545-47 (Burke); SSER # 2, § 2.5.4.6.1.2, at p. 2-51 (pier foundation load tests).

159. Ms. Stamiris' Contention 4.C(b), as amended, expresses certain safety-related concerns with respect to the remedial measures the Applicant has proposed for ensuring adequate foundation conditions for the SWPS. The contention states:

> C. Remedial soil settlement actions are not based on adequate evaluation of dynamic responses regarding dewatering effects, differential soil settlement and seismic effects for these structures:
>
> * * *
>
> b. Service Water Intake Building [sic] and its Retaining Walls

Prehearing Conference Order, dated October 24, 1980, Appendix at 6-7, as modified by Ms. Stamiris' Answer to Applicant's Interrogatories, dated April 20, 1981. (Ms. Stamiris clarified (at Tr. 9500) that this contention refers to the SWPS rather than to the adjacent Circulating Water Intake Structure (CWIS), which is not safety-related.)

160. The Seismic Category I retaining wall in the vicinity of the SWPS is structurally isolated from the SWPS and would therefore not be affected by the underpinning of the overhang portion of the SWPS. The retaining wall was constructed in two sections which are structurally isolated from one another (though the sections would perform as a unit). One section is totally founded on undisturbed glacial till and the other is totally founded on plant fill. The retaining wall has exhibited only very small settlement to date and no compressible layers of soil

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[81] We were informed (App. Reply FOF, ¶ 258) that the Applicant was giving consideration to substituting a plate load test for the test described in SSER # 2 because of the poor experience with the pier load test encountered at the auxiliary building. Since such a change, along with other possible last-minute modifications, would have been subject to Staff approval under the Work Authorization Procedure, it is not a factor in our formulation of this Partial Initial Decision. *See* discussion at Tr. 14,379.

were found in the plant fill supporting one section of the retaining wall. Therefore the foundation of the retaining wall was not part of the problem involving plant fill and it was determined that no remedial soils measures were required. Tr. 9692-93, 9723-27 (Kane); Tr. 9726-27 (Hood).

161. In evaluating the design of the SWPS underpinning, the Applicant has taken into account the load resulting from the lowest ground water level possible as a result of the temporary dewatering necessary for the construction of that underpinning (587 feet), as well as the highest possible ground water level (627 feet) (estimated as equal to the highest water elevation predicated for the cooling pond). The NRC Staff reviewed the calculations the Applicant used to analyze the design, in light of the loads which would result from the lowest and highest possible ground water levels, and found that the design was acceptable and met all applicable requirements with regard to its capacity to withstand those loads. Tr. 9698-99 (Rinaldi).

162. The Applicant predicted that after completion of the underpinning there should be no more than 0.1 to 0.2 inch of differential settlement between the overhang portion of the SWPS and the portion currently founded on glacial till. The planned method of construction would achieve small values of differential settlement by jacking loads onto the underpinning until only secondary settlement remains, before final lock-off. The NRC Staff considered this estimate of differential settlement to be reasonable and acceptable. Moreover, the NRC Staff indicated that the Applicant had considered loads associated with both the predicted differential settlement and the predicted total settlement in analyzing the design of the underpinning for the SWPS. The Applicant assigned a load factor of 1.4 (equivalent to the load factor for deadweight loads) to differential settlement loads in accordance with the requirements of the Standard Review Plan. The NRC Staff found the Applicant's calculations to be acceptable and the design for the SWPS underpinning to be conservative with respect to its capacity to withstand any loads which would be imposed as a result of predicted differential settlement. Boos, *et al.*, ff. Tr. 9490, at 34-39; Tr. 9690-91 (Kane); Tr. 9697-99 (Rinaldi); SSER # 2, § 2.5.4.5.2, at p. 2-41.

163. To implement the crack-monitoring and repair program referenced supra in Finding 154, the Applicant installed instrumentation in the underpinning itself and in the SWPS. The instrumentation would have been used to monitor any building movement which might occur prior to or during construction, in order to determine if the SWPS were suffering any structural distress as a result of the underpinning

operation. Acceptance criteria for movement and strain limits were developed and incorporated into the Applicant's construction specifications as "alert" and "action" limits, each with specified consequences. In particular, if a new crack greater than 0.01 inch developed or if an existing crack exceeded 0.03 inch in width, an evaluation would have been undertaken to determine whether underpinning procedures should be altered or halted. Requirements for repair of certain cracks were also specified. If an "action" level were reached, a report would be required to be made to the Staff; in our view, the Staff should also have been authorized to require reports (if it deemed them useful) whenever an "alert" level was reached, and (insofar as construction might be resumed) we grant such authority. Furthermore, efforts have been made to anticipate and plan for contingencies which might cause structural movement or cracking. For example, the portion of the SWPS wall which comes into contact with cooling pond water was to be coated with waterproofing compounds. Precautions were also to be taken to assure against skin friction during the pier load testing. Boos, *et al.*, ff. Tr. 9490, at 15-20; Tr. 9549-55, 9570-74, 9584-91 (Boos, Burke, Shunmugavel); SSER # 2, § 2.5.4.6.1.2, at 2-50 to 2-51 and § 3.8.3.5, at p. 3-29; Tr. 9634-38, 9641 (Poulos); statement by Steptoe (Applicant's counsel) at Tr. 9592.

164. The Applicant took into account seismic effects in evaluating its design of the underpinning for the SWPS. The SWPS underpinning was required to be designed to meet loads associated with the site-specific response spectrum (SSRS). However, because the SSRS had not been agreed upon when the design was developed, the Applicant used loads equal to 1.5 times the FSAR SSE loads in developing and evaluating the design. Subsequent analysis has determined that loads equal to 1.5 times FSAR SSE loads exceed those which would result from the now-agreed-upon SSRS. The NRC Staff reviewed the Applicant's design calculations and was satisfied that the SWPS underpinning would be adequate to meet design conditions, including earthquake motions equal to those of the SSRS. As part of the seismic margin review, the entire SWPS, existing portion plus underpinning, would have been evaluated to determine whether the integrity of the structure would be affected by earthquake motions equal to those of the SSRS. Preliminary indications were that the SWPS would withstand an SSRS earthquake without impairing safety-related functions. SSER # 2, § 3.7.2, at 3-2 to 3-4, § 3.8.3.2, at 3-14 and 3-15; Tr. 6004 (Kennedy); Tr. 9568-69 (Shunmugavel); Tr. 9626-30, 9694-97, 9701, 9713-19 (Rinaldi); Boos, *et al.*, ff. Tr. 9490, ¶¶ 5.1 and 5.2, at 20 and 21, and ¶ 7.1.1.5, at 25 and 26. We note, however, that the seismic model which was to have been utilized

for the seismic margin review of the SWPS appears to be subject to the same design deficiency as has been discussed, *supra*, at pp. 70-71 and Finding 88. Our finding with respect to the SWPS seismic model is limited to the adequacy of the *nominal* values of the soil spring constants and is subject to resolution of the design deficiency noted above.

165. Because once the underpinning for the overhang portion of the SWPS was complete the entire SWPS would be founded on undisturbed glacial till, soil liquefaction and seismic shakedown are not factors which would affect the performance of the SWPS during a seismic event. (Findings on site-wide problems of liquefaction and dewatering are set forth in Findings 90 to 117, *supra*.) The Applicant also analyzed the possibility of an interaction between the SWPS and the nearby CWIS during postulated seismic events. The results of this analysis showed that there was sufficient space between the two buildings to ensure they would not collide during an SSRS earthquake. The space available between the SWPS and the CWIS is 1 inch, while the sum of the maximum displacements of the two buildings during a postulated FSAR SSE (DBE) is 0.3 inch and during a postulated SSRS earthquake is 0.5 inch. The Staff has expressed agreement with the Applicant's analysis of possible interactions between the SWPS and the CWIS but expected to reexamine this matter as part of the seismic margin review. SSER # 2, § 2.5.4.5.5, at 2-42 to 2-44; § 3.7.2.4, at 3-4 and 3-5; Tr. 9519-21, 9575-82 (Shunmugavel); Tr. 9626-30 (Rinaldi); Tr. 9730-35 (Kane).

166. The NRC Staff was in agreement with Ms. Stamiris' Contention 4.C(b) at the time it was submitted but later became satisfied with CPC's remedial measures for the SWPS based on information subsequently submitted by CPC (Tr. 9734 (Kane)). The Board agrees and concludes, based on Findings 159 to 165, *supra*, that the Applicant has adequately taken into account the dynamic responses of the remedial soils measures for the SWPS with regard to dewatering effects, differential soil settlement and seismic effects, in the design and evaluation of those remedial soils measures. Insofar as the seismic model of the SWPS is concerned, this conclusion is limited to the *nominal* values of the soil spring constants and is subject to resolution of the design deficiency noted *supra* in Findings 88 and 164. Further, the Board concludes that the Seismic Category I retaining wall, to which Contention 4.C(b) apparently also refers, would not be affected by remedial soils measures taken with respect to the SWPS, nor would any remedial soils measures be necessary with respect to it.

167. The Licensing Board also concludes that the Applicant has complied with all applicable requirements in designing the underpinning for the SWPS. The design is conservative with respect to the loads it would

have been expected to encounter and withstand and provides reasonable assurance that, if completed as designed, the underpinning would provide an adequate and stable foundation for the overhang portion of the SWPS. Our conclusions in regard to the SWPS remedial design are subject to the outcome of a seismic margin review (including resolution of the adequacy of the soil spring constants), as well as to satisfactory execution of the remedial measures. Although we are not now resolving the QA/QC and management attitude issues which bear upon such remedial measures, any possible granting of operating licenses would necessarily be contingent upon satisfactory evaluation of past practices and construction, including the matters which have been the subject of the independent overview commenced by Stone and Webster (but not completed at the time construction was suspended — *see* letter, J.G. Keppler (NRC) to CPC, dated November 13, 1984).

V. BORATED WATER STORAGE TANKS

168. Each unit of the Midland Plant has an identical 500,000-gallon, stainless steel, borated water storage tank (BWST), which was to have supplied borated water to the emergency core cooling system (and the reactor building spray system) during the injection phase of a loss-of-coolant accident. These Seismic Category I structures, which are located in the tank farm area on the north side of the containment and auxiliary buildings, are 32 feet high and 52 feet in diameter. Each tank foundation also includes a valve pit (larger for Unit 1 than for Unit 2) connected to the southeast side of each BWST, to provide access to the piping connections to the tank and house valves for the fill and drain lines. SSER # 2, § 3.8.3.3, at p. 3-16; Hendron, ff. Tr. 7186, at 5 and Fig. 1; Boos/Hanson, ff. Tr. 7173, at 1 and Figs. BWST-1 and BWST-2; Hood, *et al.*, ff. Tr. 7444, at 4-6.

169. Each BWST is a cylindrical structure with a flexible, flat bottom. The tank shell, roof, and part of the water in the tank are supported by a reinforced concrete ring wall. Compacted granular fill lies inside the ring wall with a 6-inch layer of oiled sand separating the tank bottom from the granular fill. There is a ½-inch-thick asphalt-impregnated fiberboard (Celotex) between the tank bottom and the ring wall. The material is compressible and tends to distribute the tank wall loading to the ring wall in a more uniform manner than if there were no compressible material at the interface. Approximately 25 feet of compacted fill lies under the foundation structure. The flexible tank bottom enables most of the vertical pressure created by the weight of the water to transfer directly to the soil within the ring wall This vertical pressure also causes a lateral

pressure in the sand which is resisted by the ring wall. Anchorage for resisting overturning loads caused by externally applied lateral forces is provided by forty 1½-inch-diameter anchor bolts which attach the tank to the ring foundation. Boos/Hanson, ff. Tr. 7173, at 1-2; Kennedy/Campbell, ff. Tr. 7345, at 2 and Attach. B, at 1-3; Tr. 7382-84 (Kennedy); Tr. 7550 (Rinaldi); Tr. 7954-56 (Boos); SSER # 2, § 3.8.3.3, at p. 3-16.

170. Plant grade around the BWSTs is approximately at elevation 634 feet. From that elevation down to between 595 and 605 feet, the foundation material is compacted backfill. Below elevation 595 to 605 feet, there are competent natural soils. An area of "less stiff" or soft backfill material occurs in the southwest side of the Unit 1 BWST. Hendron, ff. Tr. 7186, at 6; Tr. 7943-44 (Boos); App. Exh. 25.

171. Exploratory programs were conducted on the natural soils at the Midland site in 1968, 1969 and 1970. Following discovery of the settlement of the DGB, additional exploratory programs were carried out in the area of the BWSTs during 1978-79 and 1981, after compacted fill materials had been placed. The foundations for the two BWSTs were constructed between July 1978 and January 1979. Erection of the tanks was completed by December 1979. Hendron, ff. Tr. 7186, at 6-8.

172. The structural adequacy of the BWSTs was questioned by Stamiris Contention 4.C(c), which reads as follows:

> 4. Consumers Power Company performed and proposed remedial actions regarding soils settlement that are inadequate as presented because:
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> * * *
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> C. Remedial soil settlement actions are not based on adequate evaluation of dynamic responses regarding dewatering effects, differential soil settlement, and seismic effects for these structures:
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> * * *
>
> c. Borated Water Storage Tanks.

Prehearing Conference Order, dated October 24, 1980, Appendix at 5-7, as supplemented by Ms. Stamiris' Answer to Applicant's Interrogatories, dated April 20, 1981.[82]

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[82] Since the BWST valve pits were subject to surcharging (i.e., "pre-loading techniques"), Warren Contention 1 applies to the BWSTs. It reads:

> The composition of the fill soil used to prepare the site of the Midland Plant — Units 1 and 2 is not of sufficient quality to assure that pre-loading techniques have permanently corrected soil settlement problems. The NRC has indicated that *random* fill dirt was used for backfill. The components of random fill can include loose rock, broken concrete, sand, silt, ashes, etc. all of which cannot be compacted through pre-loading procedures.

Warren Contention 2.B is also applicable to the BWSTs; it states:

> Given the facts alleged in Contention 2.A [concerning an allegedly inadequate dewatering system], and considering also that the Saginaw Valley is built upon centuries of silt deposits, these

(Continued)

173. In October 1980, the Applicant conducted a proof load test of the BWSTs. It filled both tanks with water and, by means of surveys, monitored the behavior of the foundations and supporting fill materials. This proof test uncovered differential settlement between the valve pit and the ring wall foundation. As a result, on January 22, 1981, the Applicant reported a deficiency of the tank foundation to the NRC pursuant to 10 C.F.R. § 50.55(e). Structural analysis indicated that the allowable moment capacity for the dead load and the differential settlement condition was exceeded at several locations in the foundation structure. Examination at the locations where overstresses were calculated revealed visible cracking in the foundations of both BWSTs — a maximum crack width of 0.063 inch for Unit 1 and 0.035 inch for Unit 2 — at the juncture of each ring wall and the valve pit structures. Boos/Hanson, ff. Tr. 7173, at 1, 3; Hood, *et al.*, ff. Tr. 7444, at 9 and Attachs. 7-8.

174. The witnesses addressing the BWST problem provided divergent explanations for the cause of the BWST cracks. Mr. Alan J. Boos and Dr. Robert D. Hanson, on behalf of the Applicant, attributed the root cause of the cracks to a design error and not to soils compaction inadequacies. They explained that the original design of the BWST foundations included the load of two small tanks which were to be located on the top slab of each valve pit; but that, when the tanks were relocated to another area, the original design of the BWST foundations was not modified. During the proof load test, when each BWST was loaded with water, the weight of the water was transferred to the soil through the tank bottom and (partly) the ring foundations, causing greater settlement beneath the tank bottom and ring foundations than beneath the valve pits. They opined that, because of this uneven settlement, the valve pits rotated relative to the ring walls and induced bending moments which had not been considered in the original design. Boos/Hanson, ff. Tr. 7173, at 3; Tr. 7274-75, 7305 (Boos). Indeed, Mr. Boos deemed the failure to have considered bending moments in the original design as sufficient in itself to have produced a lesser degree of differential settlement, without regard to whether the small tanks had been left on the valve pits. Tr. 7260-63 (Boos).

175. Dr. Alfred J. Hendron, also testifying for the Applicant, likewise attributed the BWST cracks to design inadequacy, although he reached this conclusion on the basis of a different rationale. He explained that

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highly permeable soils which underlie, in part, the diesel generator building and other class I structures may be adversely affected by the increased water levels producing liquefaction of these soils. The following will also be affected:

1) borated water tanks . . . .

Prehearing Conference Order, dated October 24, 1980, Appendix at 9.

the primary settlements observed for the BWST (about 1.3 inches at the edge of the foundations) were not excessive, and that the structural cracks at the boundary between the valve pit and the ring wall indicated that the foundations were not really designed to take the distortions that they would get from the valve pits being very lightly loaded and the ring walls more heavily loaded. Tr. 7215 (Hendron). Mr. Boos concurred with Dr. Hendron's evaluation. Tr. 7216 (Boos).

176. In contrast, the NRC Staff attributed the primary cause of the BWST differential settlement, and the resultant cracking, to inadequately compacted backfill, rather than only to a design deficiency. SSER # 2, § 2.5.4.4.3 at p. 2-34; Tr. 7449 (Hood); Tr. 7451 (Kane). A Staff witness on this question, Mr. Joseph Kane, explained that the 1.3-inch settlement experienced at the Unit 1 BWST as a result of the proof load test was greater than he would have anticipated if the soil had been properly compacted. He also relied on an additional 1.1 inches of settlement of Unit 1 which had occurred prior to the proof load test, while the tank was empty, as well as results of the soils investigations, including the plate-load tests, as indications that the differential settlement stemmed from a soils-related problem. According to Mr. Kane, absent a soils problem the settlement prior to the load test would have been no more than about ¼ inch, roughly the amount of settlement actually experienced by Unit 2. Tr. 7494-96, 7510-11 (Kane); *see also* SSER # 2, § 2.5.4.5.2, at p. 2-41 (including FSAR references). Although not advanced for this purpose, the recognition by Mr. Boos (for the Applicant) of an area of "less stiff" soil in the vicinity of BWST 1 (Tr. 7944 (Boos)) supports the Staff view that soils problems were a prime cause of cracking in the BWSTs, at least at BWST 1.

177. Other Staff witnesses recognized that, in addition to soils problems, design problems represented another factor that might have contributed to the differential settlement and hence the cracking. Tr. 7481-82 (Singh); Tr. 16,589-91 (Landsman).

178. The most balanced — and, in our view (for reasons expressed *supra* at p. 102), the most persuasive — explanation of the BWST cracks was provided by another witness for the Applicant, Dr. Robert P. Kennedy, President of Structural Mechanical Associates, Inc. (SMA). In Dr. Kennedy's judgment, there were three causes of the cracking in the ring wall. First, from the settlement patterns, he believes the soils under the west end of BWST 1 had a pocket of softer material than under the east side of the tank or under BWST 2. *Cf.* Findings 170, 176, *supra*. The second cause was the design of the valve pits, which had low bearing pressures and hence to some extent acted like a snowshoe on snow and settled less than the rings. The resulting differential settlement

caused the largest stresses and the largest cracking in the vicinity of the valve pits. Finally, the ring walls were under-reinforced: had there been sufficient reinforcing steel in the ring walls, the load would have been spread and the differential settlement would not have occurred. Dr. Kennedy was unable to say which cause was the "primary" cause of the differential settlement, although he characterized the under-reinforcement of the ring walls as a "major cause." Tr. 7366-67 (Kennedy).

179. The Applicant and (subject to certain confirmatory items) the Staff have agreed upon a three-phase corrective action for the BWST foundation problems, consisting of (a) surcharging the valve pits and their surrounding areas with sand to reduce the residual differential settlement on the foundation; (b) constructing reinforcing ring beams around the periphery of the existing cracked beams; and (c) establishing a program for releveling the Unit 1 BWST. The first phase was completed by February 1982. The surcharge process served to consolidate the fill beneath the valve pit, thereby reducing the residual differential settlement over the 40-year life of the plant. Further, it had the additional effect of reducing ring wall distortion. A monitoring program was in place to monitor foundation settlement, concrete cracks and strain in the tanks during surcharge placement and removal. This monitoring did not reveal any unexpected changes or abnormal results. Boos/Hanson, ff. Tr. 7173, at 4-10, Fig. BWST-2 and Table 1; Tr. 7223 (Boos); Hood, *et al.*, ff. Tr. 7444, at 13-18; Tr. 7447-49 (Singh); Rinaldi/Matra, ff. Tr. 7537, at 9; Tr. 7538-45 (Rinaldi); SSER # 2, § 2.5.4.4.3, at p. 2-34.[83]

180. Under the BWST corrective actions, a new ring beam, constructed of reinforced concrete with a minimum compressive strength of 4000 psi, would be added to each BWST foundation. The modified beams are designed to resist all imposed loading from the tank, including future bending induced by the predicted residual differential settlement between the ring wall and the valve pit described *infra* in Finding 181. Shear connectors would transfer the shear force from the existing ring wall to the newly constructed ring beam. Although the stiffness of the existing ring wall was taken into account in the design of the remedial

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[83] After application of the surcharge, the Applicant noted a 5-mil crack in the valve pit wall which extended to the bottom of the roof slab of the valve pit. At the point where the crack touched the slab it was only 1 or 2 mils. The Applicant was unable to determine whether the crack occurred prior to, or as a result of, the surcharge. Tr. 7284-86 (Boos). However, since the crack underwent no change subsequent to its discovery, and due to its small magnitude it was deemed by the Applicant to be of no concern. Tr. 7286-90 (Boos). NRC Staff witness Darl Hood felt there was a "very high probability" that the Staff would have concurred with that finding. However, given the fact that a commitment had been made by the Applicant to inform the Staff of the propagation of cracks related to surcharging, he felt the crack should have been reported to the Staff. Tr. 7463-66 (Hood); and Hood, *et al.*, ff. Tr. 7444, Attach. 10.

measures, no credit was taken for any strength in the existing wall. Nevertheless, all cracks found in the existing ring exceeding 10 mils were to be repaired with compressive grout to avoid potential corrosion damage to the reinforcing steel in the existing ring. Boos/Hanson, ff. Tr. 7173, at 7-8, 12, 14, and Figs. BWST-4 and BWST-5; Tr. 7253-54 (Hanson); Tr. 7548 (Rinaldi).

181. Future settlement predictions used in designing the new ring beams were based on the data obtained from the full-scale load test of the existing foundation and soil, by extrapolating the settlement versus log-time curve for each settlement marker. Basing settlement predictions on the full-scale load test of the existing foundation is conservative because the modified BWST foundations will be stiffer and thus reduce future differential settlement. Moreover, the design procedure is conservative because no credit was taken for the substantial reduction in future differential settlement which predictably will be caused by the surcharge of the valve pits. Finally, the effect of soft soil under the southwest quadrant of the Unit 1 BWST has been considered in this design approach. The soil in that area has been compressed by the water load test and subsequent surcharge of the valve pit, and the extrapolation of settlement patterns used in designing the new ring beam implicitly takes this area into account. Boos/Hanson, ff. Tr. 7173, at 4, 7, 15; Tr. 7212-13, 7943-45 (Boos); Hood, *et al.*, ff. Tr. 7444, at 17.

182. The settlement values used by Bechtel in designing the new ring beams were independently confirmed by Dr. Hendron. Dr. Hendron also confirmed that the factor of safety against bearing capacity failure of the modified ring walls will be adequate and in excess of accepted normal practice for both long-term static, and for static-plus-earthquake, loadings. Dr. Hendron also derived the appropriate long-term soil stiffness values used in the static analyses of BWSTs. Although it was outside the scope of his prepared testimony, Dr. Hendron agreed with the range of short-term moduli used in the seismic analyses of the BWST foundations. Hendron, ff. Tr. 7186; Tr. 7207-08 (Hendron); Tr. 7214 (Boos).

183. The NRC Staff and its consultant, the Corps of Engineers, reviewed and approved the settlement values and other soil parameters used in the design of the ring beams. The NRC Staff's structural engineering witness, Mr. Frank Rinaldi, stated that the Applicant's proposal to add a new ring beam to the existing foundation was "in concept . . . structurally adequate," subject to a number of stated concerns. Hood, *et al.*, ff. Tr. 7444, at 14-16; Rinaldi/Matra, ff. Tr. 7537, at 9; Tr. 7538-45 (Rinaldi). By the conclusion of the evidentiary hearings on the BWSTs, these concerns had been reduced to three in number: (1)

whether Bechtel had used earthquake loads equal to 1.5 times the FSAR SSE along with ACI-349 as supplemented by Regulatory Guide 1.142 in evaluating the structural adequacy of the modified BWST foundations; (2) whether Bechtel had in fact checked all regions of the new ring beams for all the load combinations in ACI-349 as modified by Regulatory Guide 1.142; and (3) whether using 1.5 times FSAR SSE loads for the BWST gives greater loads than the SSRS. Each of these concerns was answered affirmatively by the Applicant's witnesses. *See* Tr. 7949-51 (Boos); Tr. 7278-80 (Hanson); Tr. 7388-89, 7395-98 (Kennedy). The NRC Staff ultimately resolved the first two concerns in a structural audit of Bechtel, as documented in SSER # 2, § 3.8.3.3, at 3-16 through 3-22. Final resolution of the third concern, as far as the Staff is concerned, awaits completion of a seismic margin review. However, the Staff finds "strong evidence" that the ring beam design based on 1.5 times FSAR SSE loads will be acceptable to it. *See* Rinaldi, ff. Tr. 12,080, at 8.

184. Upon completion of the reinforced ring beam,[84] the Unit 1 BWST would be releveled. Releveling of the empty tank was to include draining and venting the tank, mounting strain gages, raising the tank, leveling the existing ring wall, releveling the oil-sand layer below the bottom plate, installing asphalt-impregnated Celotex underneath the tanks and reattaching the tank to the foundation by anchor bolts. Analyses show that the Unit 2 BWST foundation has not undergone significant tilting or out-of-plane deflections and the metal tank can withstand future predicted settlement and the SSRS earthquake without being releveled. Tr. 7349 (Kennedy, Campbell); Tr. 7544-45 (Rinaldi); SSER # 2, § 3.8.3.3, at 3-21 to 3-22.

185. The BWST tanks (as distinguished from the BWST foundations) were evaluated by Dr. Kennedy and Mr. Robert D. Campbell of SMA for stresses incurred due to uneven support conditions resulting from differential settlement of the foundations. Examination of field measurement data established that the Unit 1 BWST tank had been exposed to more severe conditions and that verification of the integrity of that tank would unquestionably verify the integrity of the Unit 2 BWST. From the anchor bolt loading (determined by strain gaging the bolts) and the known weights of tank components, all loading conditions were known. The nonuniform support reactions and resulting tank wall

[84] From documents recently provided us and the parties (which are not in the evidentiary record), it appears that the ring beams were not completed at the time construction of the facility was suspended. I&E Rept. 84-25/26, Attachment 2 ("Soils Demobilization"), enclosure to letter from R.F. Warnick, NRC, to CPC, dated September 21, 1984.

stresses were computed utilizing a finite-element model and incorporating laboratory-determined properties of the Celotex on which the tank rests. The governing design codes are the ASME Boiler and Pressure Vessel Code, § III, Nuclear Power Plant Components, subsec. NC, 1974, supplemented by ASME Code Case 1607-1 to establish allowable stresses for conditions other than normal operation (infrequent events). Kennedy/Campbell, ff. Tr. 7345, at 2-3.[85]

186. The results showed that normal operating stress limits of the governing design code were met, *with two exceptions*. First, the most highly loaded *bolt chair top plate* did not meet *normal operating* stress limits, although it did meet the *emergency* event loading criteria for an ASME Code Class 1 plate-and-shell-type component support. A subsequent dye penetrant examination of the top plate welds verified that no cracking was present. Careful visual inspections by Dr. Kennedy and Mr. Campbell did not indicate any visible deformation to any bolt chairs. Kennedy/Campbell, ff. Tr. 7345, at 3.[86] The other exception was that *local tank wall compressive stresses* did not meet normal operating stress limits. Again, the emergency-event buckling criterion was used to verify freedom from buckling. A buckling factor of safety of 2.46 was also calculated to demonstrate that a large margin existed for tank buckling. *Id.* at 3-4.[87] A visual examination of the tanks performed by Mr. Campbell while they were under their most highly stressed conditions also verified that no buckling was present. Thus, Dr. Kennedy and Mr. Campbell concluded that the uneven support which resulted from soil settlement had not resulted in any damage to the tanks. They also testified that the Unit 1 tank after releveling and the Unit 2 tank without releveling could withstand the future differential settlement predicted by the Applicant together with the SSRS earthquake without exceeding the Code-allowable stress level. Therefore, the safe operating life of the tanks had not been reduced. *Id.* at 4; Tr. 7348, 7351, 7431-34 (Kennedy).

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[85] The ASME Code design rules do not specifically cover settlement-induced stresses. Therefore Dr. Kennedy and Mr. Campbell followed what they considered to be the intent of the Code in using the second level of stress in the Code ("service level C") applicable to plant emergency conditions or infrequent loading conditions, to assess the effect of settlement. At this level the Code recognizes that some permanent deformation is possible but that the equipment will remain serviceable. Kennedy/Campbell, ff. Tr. 7345, at 3; *see also* Tr. 7350-51, 7433-34 (Campbell, Kennedy).

[86] If there had been significant buckling, it could easily have been observed visually. Tr. 7429-30 (Kennedy). Ultrasonic and x-ray inspection methods are not applicable to this type of weld. Tr. 7430-31 (Campbell); *see also* Tr. 7568-69 (Rinaldi, Matra).

[87] The 2.46 buckling factor of safety was calculated by using a NASA-developed formula documented in NASA publication 8007, as opposed to the more conservative methods recommended by ASME Code. Using Code-recommended calculations, the BWST is 9% under Service Level C allowable stresses. However, Dr. Kennedy testified that the NASA formula is more appropriate for the nonuniform axial loading of the BWST than the method recommended by the Code, which assumes uniform axial compression. Tr. 7370-81 (Kennedy).

187. The NRC Staff reviewed the Applicant's evaluation of the current condition of the tanks and also concluded that the nonuniform support condition did not impose any unacceptable stresses on the tank components. Rinaldi/Matra, ff. Tr. 7537, at 5; Tr. 7565-69 (Rinaldi, Matra).

188. Subsequent to the construction of the new ring beam, two observation pits were to be provided for each BWST foundation at the high-stress locations. The new ring beams were to undergo monitoring for a period of at least 6 months after the tanks were initially filled with water. Upon completion of a 6-month monitoring period, a report evaluating the effect of any existing cracks would be submitted to the NRC. However, if during the monitoring period any crack were to reach 0.03 inch or larger, the tanks would be emptied and the condition evaluated. Boos/Hanson, ff. Tr. 7173, at 20-21 and Fig. BWST-2; Tr. 7562 (Rinaldi); SSER # 2, § 3.8.3.3, at p. 3-22. The Applicant has committed to providing a technical specification for long-term settlement monitoring should the plant be operated, and to providing FSAR documentation of the as-built conditions for the new ring beam foundations and releveling operations, once they are completed. During the operating life of the plant, the Applicant would utilize strain-gage monitoring in the area of interest, the transition zone where the high stresses occur, to demonstrate that the ring beam foundation is performing adequately. SSER # 2, § 2.5.4.4.3, at p. 2-35; Tr. 7176-78, 7320-21 (Boos); Tr. 7178-79 (Hanson).[88]

189. Although Ms. Stamiris' Contention 4.C(c) raised legitimate questions about the effects on the BWSTs of dewatering, differential soil settlement and seismic loads, the Applicant has now adequately analyzed these effects in connection with its plans for the remedial surcharging of the valve pits, construction of new ring beams, and releveling BWST-1, measures which it has proposed and the Staff has accepted. The addition of new ring beams to the BWST foundations is based on a conservative prediction of future settlement which has been independently confirmed by Dr. Hendron and reviewed and approved by the NRC Staff and the Corps of Engineers. The prediction is conservative because it takes no credit for the effect of the water load test and of the surcharging of the valve pits, which will reduce future differential settlements. It is also conservative because the BWST foundations, as modified by the new

[88] Mr. Boos testified that in terms of developing a technique for future monitoring of the concrete foundation, the area of interest was small enough that traditional optical survey methods for determining displacements in the ring foundation would not suffice to detect the rotation of the concrete member, which is a reflection of the induced bending moments and stresses (Tr. 7176).

ring beam, will be stiffer than the old foundation and thus undergo less differential settlement than extrapolations of past settlement would indicate. The BWST tanks themselves have been shown to be unharmed by past differential settlement and able to withstand predicted future differential settlements without exceeding normal operating-service-level stresses.

190. In its prediction of future differential settlement for the BWSTs, the Applicant took into account possible dewatering effects. Rinaldi/Matra, ff. Tr. 7537, at 12; Boos/Hanson, ff. Tr. 7173, at Fig. BWST-8; Rinaldi, ff. Tr. 12,080, at 3.

191. The Applicant has also adequately analyzed the effect of potential seismic activity in developing its remedial soil measures for the BWSTs. The new ring beam interface shear connectors and new ring foundation are designed to resist resulting stress requirements without exceeding the allowable stress values and load combinations identified in ACI 318 and ACI 349-76, as supplemented by Reg. Guide 1.142. These criteria meet with Staff approval since they conform with requirements set forth in SRP § 3.8.4. Boos/Hanson, ff. Tr. 7173, at 11-12; SSER # 2, § 3.8.3.3, at 3-18 through 3-21.

192. At the time the remedial steps for the BWSTs were being initiated, the site-specific response spectra (SSRS) had not yet been developed. The Applicant, in order to proceed with the design of its proposed new foundation ring beams, adopted the load formula of 1.5 multiplied by the FSAR SSE. Dr. Kennedy testified that this procedure would result in higher stresses than the SSRS, which is equivalent to about 1.3 times the FSAR SSE. SSER # 2, § 3.7.2 at 3-2 to 3-3; Rinaldi, ff. Tr. 12,080, at 8; Tr. 6001-02, 7389 (Kennedy). In Finding 89, *supra*, the Board notes its approval of the seismic model of the BWST developed by Dr. Kennedy and accepted by the NRC Staff.

193. Although in our May 5, 1981 Prehearing Conference Order we deferred until subsequent stages of the OL proceeding the question of whether the structures as built conform to newly determined seismic criteria, preliminary evidence indicates that the BWST, as modified, would in fact meet such criteria. Dr. Kennedy testified that there is a substantial margin for the design of the tank and the foundation, taking into account both the predicted future differential settlement of the foundation and the SSRS. The Staff has not yet formally reviewed the results of the seismic margin review but, based on preliminary information provided by the Applicant, also reports "strong evidence" that the BWSTs comply with design and acceptance criteria acceptable to the Staff. Tr. 7395-99 (Kennedy); Rinaldi, ff. Tr. 12,080, at 8.

194. Dr. Richard Woods, a consultant for Bechtel appearing as a witness for the Applicant, evaluated the potential for seismic shakedown settlement at the Midland site. Although pockets of sand which have a potential for shakedown settlement exist at several site locations, Dr. Woods testified that the soil under the BWSTs exhibited no potential for such settlement. Moreover, the sand within the ring foundation has been compacted to a relative density greater than 80% for which no significant seismic shakedown settlement will occur. Woods, ff. Tr. 11,549, at 3-6. The Applicant has shown and the Staff agrees that the materials underneath the BWSTs are not subject to liquefaction. Woods, ff. Tr. 9745; SSER # 2, § 2.5.4.5.5, at 2-43 and 2-44. Intervenor Sharon Warren's Contention 2.B expressed concern for liquefaction adversely affecting the BWSTs. Mr. Kane testified that the Staff is satisfied that liquefaction is not a problem for the BWST structures. Tr. 9817. The Board agrees.

195. The Board concludes that the concerns set forth by Ms. Stamiris in Contention 4.C(c) have been adequately addressed in the remedial soil measures being taken for the BWSTs. The Applicant has shown and the Staff has verified that the remedial measures, assuming they are successfully completed, will provide reasonable assurance that the BWSTs will perform their intended safety functions throughout the operating life of the plant. Moreover, Staff-approved methods of monitoring the BWSTs for settlement, concrete cracking and strain provide additional assurance that any unanticipated future differential settlement would be detected and corrected before presenting any undue risk to the public health and safety. The details of the monitoring remain an open question, pending submission by the Applicant and approval by the Staff of a technical specification governing such monitoring. Our reasonable assurance finding is subject to the submission by the Applicant and approval by the Staff of an appropriate technical specification governing long-term settlement monitoring, together with additional FSAR documentation, as set forth in SSER # 2, § 2.5.4.4.3, at p. 2-35; § 2.5.4.6.3, at p. 2-52; and Table 2.8, at p. 2-53.[89]

VI. DIESEL FUEL OIL TANKS

196. There are four Seismic Category I steel diesel fuel oil storage tanks at the Midland Nuclear Power Plant site. They are located to the southeast of the DGB and are buried approximately 6 feet underground.

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[89] These conclusions are also dispositive of Warren Contention 1, insofar as it relates to the BWSTs.

The function of the emergency diesel fuel system is to supply fuel to the onsite diesel generators in case of loss of offsite power. Eight diesel fuel oil lines provide fuel oil supply and return between the diesel generators and the four buried diesel fuel oil storage tanks.

The diesel fuel oil storage tanks were designed and fabricated to the requirements of ASME Code, § III, Class 3 (1974). Their 3-foot-thick concrete foundations, which rest predominantly on a supporting base of medium stiff to medium dense sandy clay backfill material, were designed and fabricated to the requirements of ASME Code, § III, Class 3 (1974) and also, ACI 318-71. The tiedown is designed to the AISC-1971. The Staff has determined that the load combinations and acceptance criteria utilized by the Applicant in designing the four storage tanks meet the Staff's design requirements. Rinaldi/Matra, ff. Tr. 7537, at 10, 12, Attach. 4; Tr. 12,071-73 (Kane); Landers, *et al.*, ff. Tr. 7619, at 5-7; SER, § 1.12.10, at p. 1-25 (Staff Exh. 14).

197. Stamiris Contention 4.C(d), as amended, states as follows:

> 4. Consumers Power Company performed and proposed remedial actions regarding soils settlement that are inadequate as presented because:
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> * * *
>
> C. Remedial soil settlement actions are not based on adequate evaluation of dynamic responses regarding dewatering effects, differential soil settlement, and seismic effects for these structures:
>
> * * *
>
> d. Diesel Fuel Oil Storage Tanks.

Prehearing Conference Order, dated October 24, 1980, Appendix at 6-7, as supplemented by Ms. Stamiris' Answer to Applicant's Interrogatories, dated April 20, 1981. In addition, one of the contentions of Ms. Warren which the parties addressed (*see* Finding 41), claims that the diesel fuel oil tanks will be affected by liquefaction resulting from an allegedly inadequate dewatering system.[90]

198. The Applicant undertook a program of measurement, analysis and monitoring to assure that the tanks could perform their intended functions throughout the operating life of the plant. The tanks had been installed approximately 2 years after the fill was placed, and therefore

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[90] Warren Contention 2.B(2) states:

> Given the facts alleged in Contention 2.A [concerning an allegedly inadequate dewatering system], and considering also that the Saginaw Valley is built upon centuries of silt deposits, these highly permeable soils which underlie, in part, the diesel generator building and other class I structures may be adversely affected by increased water levels producing liquefaction of these soils. The following will also be affected:
>
> * * *
>
> 2) diesel fuel oil tanks.

Prehearing Conference Order dated October 24, 1980, Appendix at 9.

were isolated from the effects of the fill's initial settlement. In 1979, the Applicant surcharged the four tanks by filling them with water and monitored settlement for about an 8-month period. The Applicant's witnesses (Messrs. Donald Landers, Donald Lewis and James Meisenheimer) testified that the diesel fuel oil storage tanks will settle with the surrounding soil, as will the connecting pipes. Thus, the differential settlement between the pipes and the tanks would be small, and the nozzle loads due to settlement, insignificant. Rinaldi/Matra, ff. Tr. 7537, at 10; Rinaldi, ff. Tr. 12,080, at 5-6; Landers, *et al.*, ff. Tr. 7619, at 11.

199. NRC Staff witness Joseph Kane testified that, at the time of the hearing, the Staff was not concerned about the foundation stability of the four diesel fuel oil storage tanks. He stated that a total maximum settlement of a half an inch was the largest settlement recorded for the diesel fuel oil storage tanks. Following surcharging in 1979, the tanks experienced a maximum settlement of a quarter of an inch. An additional quarter-inch settlement occurred in late 1980 as a result of temporary dewatering conditions; however, when the ground water table was allowed to rebound, settlement rebounded one-tenth of an inch, to a total settlement of four-tenths of an inch. For the expected operating life of the plant, additional settlement of approximately half an inch was estimated. The NRC Staff, in recognizing and accepting the settlement values relating to the storage tanks, concluded that the results of the analysis and monitoring program performed by the Applicant indicated that the Staff did not anticipate any significant problem for these tanks or their pedestals resulting from differential settlement, and there was no reason for any structural concerns relating to the effects of differential soil settlement on the diesel fuel oil storage tanks. Tr. 12,071-73, 12,090-91 (Kane); Landers, *et al.*, ff. Tr. 7619, at 11; Rinaldi, ff. Tr. 12,080, at 5-6; Rinaldi/Matra, ff. Tr. 7537, at 12; SER, § 1.12.9, at p. 1-25. The Staff has recently raised questions, however, as to the continuing viability of its earlier conclusions on the stability of soils beneath the diesel fuel oil tanks. Kane Affidavit, dated December 21, 1984, submitted to Board and parties by letter dated December 21, 1984 (*see supra* pp. 38-39, 103-04).

200. The Applicant analyzed and evaluated the effects of dewatering, seismic events, and differential soil settlement on the diesel fuel oil storage tanks. It analyzed and monitored the tanks for possible effects caused by differential settlement of the soil supporting them. It found the tanks to be in an acceptable and functionally capable condition, leading the Staff to express its belief that, subject to an audit of the information, and to the outcome of the seismic margin review, any structural concerns regarding the fuel oil tanks which are represented in

Stamiris Contention 4.C(d) are without merit. The effect of dewatering on settlement of the tanks was taken into account. As stated *supra* in Finding 199, following dewatering, the tanks reached a maximum settlement of half an inch. When the ground water table was allowed to rebound to the full-scale recharge test, rebound settlement of one-tenth of an inch occurred. The Staff found these settlement values acceptable. Landers, *et al.*, ff. Tr. 7619, at 11, 35; Rinaldi, ff. Tr. 12,080, at 5-6; Rinaldi/Matra, ff. Tr. 7537, at 12; Tr. 12,071-73, 12,090-91 (Kane).

201. The Applicant also analyzed the fuel storage tanks for seismic-induced loads in conjunction with normal, thermal and differential settlement loads. In addition, it provided a reinforced concrete cover to resist the impact of postulated tornado missiles. As noted *supra* in Finding 196, the Staff determined that the load combinations and acceptance criteria used by Applicant to design and fabricate the tanks meet the Staff's design criteria. (Although the tanks were designed for the original seismic loads of the FSAR SSE (DBE), in the seismic margin review they were to be reevaluated using the site-specific response spectra.) Dr. Richard Woods evaluated the potential for seismic shakedown of loose sands at the Midland Plant. His analysis revealed that sands for which there is a potential of shakedown settlement, exist in a number of site locations. One boring performed in the diesel fuel oil storage tank area revealed the existence of loose sand. Dr. Woods testified that the maximum shakedown settlement which would occur based on evaluation of loose sands in this boring amounts to about 0.10 inch. These settlements do not present any hazard to the diesel fuel oil storage tanks. Rinaldi, ff. Tr. 12,080, at 6-8; Rinaldi/Matra, ff. Tr. 7537, at 10; Woods, ff. Tr. 11,549, at 7; Tr. 11,557-58 (Kane). However, information uncovered recently casts doubt on any conclusions based on borings beneath the diesel fuel oil tanks. *See supra* pp. 38-39, 103-04. We are making no findings at this time on the stability of soils beneath the diesel fuel oil tanks.

202. Dr. Woods also presented testimony regarding the potential for liquefaction at the buried diesel fuel oil storage tanks. He explained that during the initial liquefaction boring study, a loose sand pocket was discovered in one of the borings close to the storage tanks. Using an earthquake producing a peak ground acceleration of 0.19*g* and what he regarded as conservative assumptions (based on certain borings), Dr. Woods had concluded, and the Staff was satisfied, that no danger of liquefaction exists for the tanks. Tr. 9747-49 (Woods); Woods, ff. Tr. 9745, at 13-14, and Fig. L-3; Tr. 12,071-73 (Kane). However, the Board has recently been advised that the logs of borings relied upon to establish the conservatism of Dr. Woods' conclusions were erroneous and that

the analyses of liquefaction under the diesel fuel oil tanks must be regarded as inconclusive (*supra* pp. 38-39, 103-04). For these reasons, we are making no findings at this time with respect to liquefaction under the diesel fuel oil tanks.

203. The Board concludes that the outstanding open items regarding soils stability and liquefaction are significant enough to preclude our reaching any final conclusions with respect to Ms. Stamiris' Contention 4.C(d) or, to the extent it relates to liquefaction under the diesel fuel oil tanks, Warren Contention 2.B(2). We also are reaching no "reasonable assurance" conclusions with respect to those tanks.

VII. UNDERGROUND PIPING

A. Introduction

204. Two of Ms. Stamiris' OM contentions (Nos. 4.A(4) and 4.C(f)) relate to the technical (as distinguished from QA/QC) aspects of underground piping. They read:

> 4. Consumers Power Company performed and proposed remedial actions regarding soils settlement that are inadequate as presented because:
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> * * *
>
> A. Preloading of the diesel generator building
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> * * *
>
> 4) may adversely affect underlying piping, conduits or nearby structures;[91]
>
> * * *
>
> C. Remedial soil settlement actions are not based on adequate evaluation of dynamic responses regarding dewatering effects, differential soil settlement, and seismic effects for these structures:
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> * * *
>
> f. Related Underground Piping and Conduit.[92]

Prehearing Conference Order, dated October 24, 1980, Appendix at 5-6, as supplemented by Ms. Stamiris' Answer to Applicant's Interrogatories, dated April 20, 1981. In addition, one of the contentions of Ms. Warren which the parties addressed (*see supra* note 41) questioned the stress produced by surcharging of the DGB on, *inter alia*, circulating water lines and fuel oil lines.[93]

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[91] *See infra* Findings 293-305, for a discussion of the portions of Ms. Stamiris' contentions dealing with underground conduit. We are not dealing in this decision with the effect of the DGB surcharge on nearby structures.

[92] *See supra* note 91.

[93] That contention (Number 3) states:

> Pre-loading procedures undertaken by Consumers Power have induced stresses on the diesel generating building structure and have reduced the ability of this structure to perform its essen-

(Continued)

205. A concern for foundation stability of underground piping at the Midland Plant arose because the plant fill supporting these pipes was found to be inadequately compacted and settling under its own weight. Consequently, piping buried in the plant fill was settling with the fill. Observed settlements have not been uniform because of the highly variable soil fill conditions, differences in actual loadings, and also due to the varying foundation elevations of structures connected with underground piping. SER, § 1.12.10, at p. 1-25 (Staff Exh. 14); Kane, ff. Tr. 7752, at 1-2.

206. There are two categorizations for underground piping systems and components at the Midland facility: Seismic Category I and Nonseismic Category I. SER, § 1.12.10, at 1-25 to 1-26, and § 3.9.3.1, at 3-28 to 3-30; SSER # 2, Table 3.1, at p. 3-33 (Staff Exh. 14, Supp. 2). The Applicant and Staff have included in the first category those systems and components which they regard as "important to safety" and which are designed to withstand the effects of the earthquake forces applicable at the Midland site.[94] Those systems and components are reviewed to assure through analysis and, where appropriate, remedial measures and/or monitoring that they will perform their intended safety functions throughout the plant's projected service life. *See, e.g.*, Tr. 7763 (Kane); Tr. 7931-32 (Chen). In contrast, the Nonseismic Category I items are reviewed to the extent necessary to assure that postulated failures would not have an adverse impact on nearby Seismic Category I structures or piping. SER, § 2.4.6.3, at 2-28 to 2-29; SSER # 2, § 2.4.6.3, at 2-5 to 2-6, and § 3.9.3.1.2, at p. 3-34; Tr. 3646-47, 3649 (Kane); Tr. 7825-26 (Hood).

B. Seismic Category I Underground Piping

(1) General

207. There are five types of buried Seismic Category I piping at the Midland Plant, ranging in size from 1 inch to 36 inches in diameter. These types are (1) service water system (SWS) lines; (2) diesel fuel oil

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tial functions under that stress. Those remedial actions that have been taken have produced uneven settlement and caused inordinate stress on the structure and circulating water lines, fuel oil lines, and electrical conduit.

Prehearing Conference Order, dated October 24, 1980, Appendix at 9-10.

[94] We understand the Applicant and Staff to utilize the term "important to safety" as it appears in 10 C.F.R. Part 100, Appendix A. We are here using it similarly but are expressing no opinion as to the exact scope of such terminology. *See* BN 84-011, provided by the Staff to the Board and parties by Memorandum dated January 18, 1984; *see also* Tr. 3646-47 (Kane). *See also* note 12, *supra* p. 52.

For a discussion of the earthquake forces applicable to Seismic Category I items, *see supra* Findings 19-79.

lines; (3) borated water lines; (4) control room pressurization lines; and (5) penetration pressurization lines. SSER # 2, Table 3.1, at p. 3-33 (Staff Exh. 14, Supp. 2).

208. The smaller underground pipelines are seamless, while the 18-inch and larger-diameter pipes are seam-welded. These larger-diameter pipes are fabricated in nominal lengths ranging approximately from 4 to 40 feet, which are fitted together and welded. The welds are inspected and hydrostatically tested to assure integrity. Landers, *et al.*, ff. Tr. 7619, at 7.

209. All of the underground Seismic Category I pipelines at the Midland site rest on compacted backfill material. As a result of its discovery of insufficiently compacted fill material at a number of onsite locations and its investigation (in part through borings) of such fill conditions, the Applicant ascertained that the consistency of the fill at the location of buried piping can vary considerably in a vertical direction within a boring, and also laterally as evidenced by closely spaced borings. Settlements of buried piping were primarily a result of fill settling under its own weight; the piping itself adds little, if any, weight to the fill and hence has little impact on settlements. The Applicant also undertook internal profiling of some of the buried pipes to establish pipe deflection (settlement) profiles. The results of the profiling indicate that the pipe invert elevations[95] have maximum deviations from 6 to 21 inches below the originally intended elevations, with the majority in the range of 9-11 inches. In contrast, field installation procedures for the installation of the piping provided for a placement tolerance of ± 2 inches from the design invert of elevation. Even if credit is taken for placement tolerances, deviations in pipe elevations from design values of at least 4 to 19 inches occurred. Landers, *et al.*, ff. Tr. 7619, at 7-9, 13-14; SSER # 2.5.4.4.5, at p. 2-35, and § 1.12.10, at p. 1-25; Tr. 7658 (Meisenheimer); Tr. 7693 (Lewis); Tr. 7807 (Kane).

210. Inspection records would suggest that Seismic Category I piping was installed within the ± 2-inch placement tolerance, inasmuch as no construction nonconformances related to this requirement were reported. However, lacking any profiles to verify post-installation locations, it is not known how much of the deviation in invert elevations is due to soil settlement alone. Although some of the deviation is likely the result of fabrication and installation, the Applicant and NRC Staff conducted

[95] As we understand it, "invert elevation" refers to the elevation at the bottom of the pipe below the pipe's central axis.

their analyses of underground piping on the assumption that all variations in design elevation are due to settlement. Chen/Hood, ff. Tr. 7762, at 6; Tr. 7693-95 (Lewis). One Staff witness questioned the conservatism of that approach (Tr. 7766 (Chen)). Others expressed reasons for requiring such post-placement profiles (Tr. 7904 (Kane, Hood)). In the Board's view, the analyses of piping would have been more accurate if post-placement pipe profiles had been prepared. In addition, such profiles could assist in the monitoring of future settlement (Tr. 7624 (Kane)). For that reason, we are providing that, if further placement or replacement of underground Seismic Category I piping were to be carried out, the Applicant must prepare as-built pipe profiles to verify the post-installation location of the pipes.

211. The Applicant compared depth profiles along pipelines with subsurface conditions projected from adjacent exploration borings. Its direct testimony indicated that it could establish no correlation between lower profile areas and softer underlying fill areas or between higher profiles and stiffer underlying fill soils. Nor, according to its direct testimony, did the Applicant observe abrupt differential variations in pipeline profiles in areas where closely spaced borings indicate stiffer soils and softer soils adjacent to one another. Landers, *et al.*, ff. Tr. 7619, at 9. On the other hand, the Staff, in reviewing pipe settlement profiles, did detect such a correlation. It observed a general pattern where the major settlement of pipes occurred under the greatest surcharge loading. But one instance where the piping experienced smaller settlement in the surcharge area could be explained by recognizing that other pipes encased in concrete had put a discontinuity into the foundation support beneath the higher placed piping. Tr. 7902-03 (Kane). On cross-examination, one of the Applicant's witnesses acknowledged such a correlation (Tr. 7658 (Meisenheimer)). The Staff also explained that one reason it had requested development of soil profiles along the alignment of the underground piping was to identify the softer soil areas as evidenced by the low blowcounts recorded in the soil borings that had been completed. It used this information to determine where settlement markers should be installed. Tr. 9053, 9088, 9090 (Kane).

212. Records of the monitored settlement within the fill have been utilized to predict future settlement for buried pipes. A series of markers (Borros anchors) have been installed at nine locations in the vicinity of buried piping not influenced by surcharge loadings. Settlement readings for anchors that have been established at depths of 7 to 12 feet below the surface were used in the analysis, because this depth is representative of the depth of most buried pipes or utilities. Soil conditions at these locations are representative of the variable soil conditions encountered

throughout the fill. SSER # 2, at p. 2-36; Landers, *et al.*, ff. Tr. 7619, at 9.

213. Borros anchors BA 13, BA 14, and BA 34 were installed in December 1978. Settlement data have been taken on these anchors for over 5 years. Borros anchors BA 100 through BA 106 were installed in September 1979, and over 4½ years of settlement data exist for these anchors. As of the close of the record on underground piping, the plots of settlement versus log-time for each of these anchors formed straight lines which extrapolate to 2.0 to 2.5 inches of additional settlement occurring over the next 40 years. Based on these projections, the Applicant and the NRC Staff have concluded that a conservative estimate of future maximum settlement of buried piping or utilities is for not more than 3 inches of additional settlement to occur at any pipe location, provided only limited loads are placed over the piping. This estimate includes allowances for settlement due to both seismic shakedown and dewatering. SSER # 2, § 2.5.4.4.5, at p. 2-36; SER, § 1.12.10, at p. 1-25; Kane, ff. Tr. 7752, at 6; Landers, *et al.*, ff. Tr. 7619, at 10; Shunmugavel, ff. Tr. 12,016, at 6. As indicated in Findings 259, 262, *infra*, the 3-inch settlement estimate is to be considered as an acceptance criterion. The Applicant committed to providing a technical specification that would include control measures restricting placement of heavy loads over buried piping and conduits. In addition, were the plant to be operated, the technical specifications should include alert and action limits based on the foregoing acceptance criterion for settlement.

(2) Assurance of Serviceability

214. The various Seismic Category I underground pipes have been reviewed by the Applicant and Staff to assure their continued serviceability over the life of the facility. Remedial activities for each pipe depend upon the type of pipe, the conditions and timing in which it was initially installed, and the settlement and other measurements described previously. Among the remedial actions included for piping are replacement, rebedment, and reinstallation, which are defined as follows:

Replacement — the removal of existing buried pipe and the installation of new pipe.

Rebedding — the exposure of the existing buried pipe, removal of underlying soil, placement of new underlying fly ash concrete fill, realignment of the existing pipe, repairs to the pipe coating, and backfill around and over the pipe.

Reinstallation — the replacing and/or rebedding of piping.

Lewis, ff. Tr. 8868, at 9. We turn first to the criteria utilized to evaluate underground piping and then to the remedial actions which were planned to be utilized for each category of piping.

(a) Criteria

(i) Stress Analyses and Design Criteria

215. Section 3.9.3 of the Standard Review Plan (SRP) defines the design criteria and load combinations to be employed in the design of ASME Code Class 1, 2 and 3 items. Stresses in piping as a result of soil settlement are not addressed either by the SRP or the 1971 Edition of the ASME Code (with Addenda through Summer 1973), which generally governs the Midland facility. However, the 1977 Edition of the ASME Code addresses single deflection of a pipe through a discussion of "single nonrepeated anchor movement." SSER # 2, § 3.9.3.1.3, at p. 3-35; Tr. 7811 (Chen); Tr. 7815 (Hood); Landers, *et al.*, ff. Tr. 7619, at 23; *see also* 10 C.F.R. § 50.55a(d)(2).

216. To augment the SRP and the ASME Code, the Applicant initially proposed a design criterion of $3S_c$ (three times the allowable basic material stresses at minimum (cold) temperature, in psi) for its evaluation of the buried pipe. SSER # 2, § 3.9.3.1.3, at p. 3-35. Stress analyses based on the assumption that existing deviations from design configurations are due solely to differential settlement yielded stresses which in some cases exceeded the $3S_c$ criterion. *Ibid.*; Landers, *et al.*, ff. Tr. 7619, at 23-24; Chen/Hood, ff. Tr. 7762, at 8. Subsequently, to provide a greater margin of safety, the Applicant proposed a combination of the $3S_c$ criterion, additional design criteria, remedial action and monitoring to assure the safety and serviceability of the Seismic Category I underground piping. SSER # 2, § 3.9.3.1.3, at p. 3-35; Chen/Hood, ff. Tr. 7762, at 8-9.

(ii) Strength Criteria

217. These criteria are intended to provide assurance that the overall cross-sections of piping are capable of resisting the forces and movement due to all loads imposed upon the piping over the life of the plant. These loads include pressure, thermal expansion, overburden and traffic, soils settlement and seismic loads. SSER # 2, § 3.9.3.1.3, at 3-35 to 3-36; Chen/Hood ff. Tr. 7762, at 7.

218. For settlement stresses only, the $3S_c$ criterion is an acceptable strength criterion (SSER # 2, § 3.9.3.1.3, at p. 3-36). In cases where the $3S_c$ criterion could not be satisfied, however, the Applicant and the

NRC staff considered the effects of load combinations that could lead to catastrophic effects in a short amount of time in comparison to the proposed monitoring frequency. In particular, the Staff and the Applicant considered and made provisions for adequate margins of safety for the effects of settlement in conjunction with 1.5 x FSAR SSE ground motion forces (i.e., using an input of 0.18*g* ground motion). The 1.5 x FSAR response spectra envelopes the site-specific response spectra (SSRS) for purposes of the BC-TOP-4A analyses of buried piping. Tr. 8941-44 (Lewis).

219. With respect only to underground SWS piping to be reinstalled, the Applicant performed a dynamic seismic analysis based on the FSAR SSE earthquake (0.12*g* ground motion). The Applicant committed to run a check analysis using BC-TOP-4A techniques and 1.5 x FSAR SSE as input (Tr. 8942-43 (Lewis); Lewis, ff. Tr. 8868, Table 4, Enclosure 2, at Sheet 3, n.2). The Applicant was given permission to supplement the record to explain how the underground SWS piping to be installed meets current criteria (Tr. 8944). By affidavit dated January 21, 1983 (Enclosure E to Applicant's letter to Board dated February 3, 1983), Dr. Thiru Thiruvengadam of CPC demonstrated that input spectra used in the dynamic seismic analysis of the SWS piping to be reinstalled (which had earlier been analyzed against the FSAR SSE) in fact exceeds the current SSRS criteria. On November 2, 1983, the Staff filed an affidavit of Dr. Paul Chen indicating concurrence with Dr. Thiruvengadam's affidavit. (No other party has commented on either affidavit.)

220. In addition, overburden and vehicular load effects were assessed relative to the margins of safety for existing Code criteria (SSER # 2, § 3.9.3.1.3, at p. 3-36).

221. The following strength criteria have been found acceptable by the NRC Staff:

Criterion 1: $S_{ss} \leq 3S_c$

where S_{ss} = stresses due to differential soil settlement only.

In cases where Criterion 1 could not be satisfied, the following three criteria must be met:

Criterion 2: The total ovality due to a 1.5 x FSAR SSE plus soils settlement must be less than the maximum allowable ovality permitted for the diameter-to-wall thickness ratio of the pipe.

Criterion 3: $S_{SL} + S_{o/b} \leq 1.5\ S_h$

where S_{SL} = stress due to sustained loads, as defined in the ASME Code;

$S_{o/b}$ = stress due to overburden loads;

S_h = basic material stress allowable at operating temperature, in psi.

Criterion 4: $S_{OL} \leq 1.8\ s_h$

where S_{OL} = stress due to occasional loads, as defined in the ASME Code, but also including bending or other stresses due to traffic loads.

(iii) Buckling Criteria

222. The buckling criteria discussed herein are intended to provide assurance that local buckling (which could lead to cracking in the pipe) and gross collapse (which could lead to loss of function of the pipe) would not occur throughout the life of the plant. Buckling data were obtained from theoretical and experimental sources available in the current technical literature. These data were reviewed in depth by the Staff and adapted for specifying buckling criteria for underground piping. For this type of piping, the criteria are expressed specifically in terms of ovality and strain criteria. Ovality of a pipe is defined as:

Ovality = $(D_{max} - D_{min})/D$

where D = outside diameter of unovalized pipe

D_{max} = maximum outside diameter of ovalized pipe

D_{min} = minimum outside diameter of ovalized pipe

Based on these data, the allowable ovality adopted for the underground piping over the life of the plant is 4% for pipe with a diameter-to-wall thickness (*D/t*) ratio of 69 and a factor of safety of 1.5. SSER # 2,

§ 3.9.3.1.3, at 3-36 and 3-37; Chen/Hood, ff. Tr. 7762, at 7; *see also* Landers, *et al.*, ff. Tr. 7619, at 16, 19, 21-25.

223. Where monitoring of pipe ovality was to be specified, the ovality would be determined by measuring pipe strains. A specific strain-to-ovality relationship was developed by the Applicant and approved by the Staff. *See* Lewis, ff. Tr. 8868, at 3 and Fig. 1; *see also* SSER # 2, § 3.9.3.1.3, at p. 3-37; Landers, *et al.*, ff. Tr. 7619, at 24-26. For pipes with a *D/t* ratio of less than 69, the permissible maximum ovality under this relationship is actually greater than 4%, but the Applicant agreed to the 4% limit. SSER # 2, § 3.9.3.1.3, at p. 3-37.

(iv) Minimum Rattlespace Criteria

224. A "rattlespace" is the gap opening between the exterior of a pipe and the wall of a building or other structure which the pipe penetrates. The minimum rattlespace criteria discussed herein are intended to provide assurance that both local and gross overstressing of the piping and gross overstressing or distortion of piping components or attached equipment would not occur due to loads which may be imposed or are postulated to occur during the life of the plant. Tr. 7892 (Hood); SSER # 2, § 3.9.3.1.3, at p. 3-36.

225. The clearance conditions of the piping at building or other structural penetrations are in part dependent on the proposed remedial actions for the associated piping in the plant fill (*see infra* Findings 227-250) and on the configuration of the piping at the penetrations. These conditions are therefore quite variable and have required case-by-case study for their resolution. SSER # 2, § 3.9.3.1.3 at 3-37 and 3-38.

226. In general, assurance that minimum rattlespace will be adequate over the projected life of the plant was provided by the analytical method set forth in § 3.9.3.1.3 of SSER # 2 with respect to the 36-inch SWPS pipe penetrations. This criterion requires that the minimum rattlespace shall be greater than or equal to 0.5 inch at all locations after taking into account variations in calculated pipe displacement resulting from predicted future settlement (*see supra* Finding 213) or the effects of a 1.5 x FSAR or an SSRS SSE (*see supra* Finding 219, and *infra* Finding 240). SSER # 2, § 3.9.3.1.3, at p. 3-38.

(b) Remedial Actions

(i) Service Water Piping

227. The SWS piping includes twenty-two lines, consisting of eight lines of 8-inch diameter, two 10-inch-diameter lines, eight 26-inch-diameter lines, and four 36-inch-diameter lines. These lines, constructed of ASME Code Class 3 SA-106 and SA-155 carbon steel piping, were to be used to supply water to various systems as needed under normal and accident conditions. SSER # 2, Table 3.1 and § 3.9.3.1.1, at p. 3-33.

228. All of the 26- and 36-inch-diameter SWS piping at the Midland plant (*see supra* Finding 22.) was subjected to extensive profile and pipe ovalization measurement programs in November 1981. Profile data were obtained at 5-foot intervals along the pipe lengths and at welds, and are accurate to 1/16 inch. These 1981 data, which supersede the previously obtained 1979 data, which were accurate only to 1/4 inch, were furnished to the Staff in 1982. The data show that the piping was, on the average, approximately 5 inches below its design elevation, with deviations of up to 8 to 12 inches. The 1981 data also show that, in general, pipe ovalizations were between 1 and 1.5%, with a maximum of 3%. SSER # 2, § 3.9.3.1.1, at p. 3-33; *see also* Landers, *et al.*, ff. Tr. 7619, at 13-14.[96]

229. All the 8- and 10-inch SWS piping is located in the vicinity of the DGB. These lines were installed before the soils settlement problem was recognized, and they were in place during the DGB surcharge program. The lines were profiled in 1979, and the data indicated that they were, on the average, 6 to 8 inches below their design elevation, with a maximum deviation of up to 21 inches. SSER # 2, § 3.9.3.1.1, at 3-33 to 3-34.

230. The two longest SWS lines that exhibited the greatest deviations are located north of the DGB between the DGB and the turbine building. These lines were rebedded after the removal of the DGB surcharge. In addition, pipe diameter verification has been conducted on 4-foot lines. The verification indicated that these lines are acceptable in accordance with American Waterworks Association (AWWA) requirements (i.e., less than 5% ovality). These rebedded and diameter-verified lines have been disconnected at the bolted connections at or near their DGB penetrations and have been recentered in their rattlespace annuli. SSER # 2, § 3.9.3.1.1, at p. 3-34.

231. The Applicant and Staff did not agree on the adequacy of the 36-inch-diameter SWS piping, but the Applicant, as discussed below,

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[96] *See* SSER # 2, § 2.5.4.4.5, Fig. 2.11, for a diagram of the various SWS pipes.

agreed to replace this pipe. Following hearings in April 1982, it was determined that it was also necessary to rebed a portion of the buried 26-inch-diameter SWS piping as part of a fill replacement program to resolve potential liquefaction concerns in the area north and west of the SWPS. Because all the 36-inch-diameter SWS pipe is located in this area of potential liquefaction, it too was to be rebedded during replacement. Lewis, ff. Tr. 8868, at 8; *see also* Enclosure 2 to Applicant's letter dated March 16, 1982, serial 16269, attached as Reference 2 to the Lewis testimony.

232. The reinstallation program for SWS piping proposed by the Applicant and accepted by the NRC Staff included the reinstallation of the buried 36-inch-diameter SWS piping in the vicinity of the SWPS and the rebedding of the two buried 26-inch-diameter service water lines immediately north of the circulating water intake structure. The 36-inch lines which were to be replaced were the service water supply and return lines at the point of entry to and from the SWPS. Lewis, ff. Tr. 8868, at 10. The 26-inch pipes which were to be rebedded were service water supply and return lines to and from the DGB and turbine building. The lines proposed to be rebedded extended from the 36-inch lines to a point even with the southwest edge of the CWIS. *Id.* at 11.

233. The new fill material used in the reinstallation program to replace the potentially liquifiable fill in the area north of the SWPS and CWIS was to be a type of low-strength fly ash concrete similar to the material known by the brand name "K-KRETE." The properties of this new fill material would have been similar to those set forth in Table 3 to the testimony of Applicant's witness Donald F. Lewis (ff. Tr. 8868). These properties were to be verified by testing (*id.* at 11). This material was to be placed to a level of 1 foot above the top of the pipe. SSER # 2, § 2.5.4.4.5, at p. 2-36.

234. The existing 36-inch-diameter buried pipe would have been replaced with 36-inch-diameter welded ASME SA-672, Grade B-70, Class 20 pipe. The 0.625-inch nominal wall thickness would result in a *D/t* ratio of 57.6, considerably and acceptably reducing the potential for local buckling. SSER # 2, § 3.9.3.1.3, at p. 3-38; Lewis, ff. Tr. 8868, at 11.

235. The 36-inch pipe would be encased in a 6-inch-thick layer of a compressible polyethylene material known as "Ethafoam," which would create a transition that would eliminate concentrated shear strain to the piping caused by differential settlement (SSER # 2, § 2.5.4.4.5, at 2-36 to 2-37; § 3.9.3.1.3, at p. 3-39; Affidavit of Palanichamy Shunmugavel on Ethafoam, dated August 2, 1983, at 8). By so doing, the Ethafoam would minimize the effects of differential settlement.

236. The reinstallation of the designated SWS lines would have been coordinated with the SWPS underpinning. The excavation required to expose these lines and replace unsuitable fill would be contiguous with the excavation for the SWPS underpinning. Underground pipelines that would be exposed during excavation work would be left in place, and temporarily supported and protected to preclude damage. Precautions would include, as necessary, such measures as:

a. shoring and bracing supporting fill;
b. complete temporary support;
c. staking utility locations prior to excavation; and
d. hand excavation near utilities.

A list of structures, facilities, and utilities that might have been encountered or affected by the excavation is included in Table 5 to the testimony of Applicant's witness Donald F. Lewis. Lewis, ff. Tr. 8868, at 14 and Table 5.

237. Fill material within limits agreed to by the Applicant and the NRC Staff (*id.*, Table 4) would be excavated down to elevation 610 feet and replaced with a suitable material to minimize settlement and prevent liquefaction. Predicted future settlement, considering replacement of loose or soft fill material, was not expected to exceed 1.5 inches, a figure less than the 3.0 inches of settlement estimated for the existing fill. SSER # 2, at 2-36, 3-39; Lewis, ff. Tr. 6686, at 11.

238. The 26-inch pipe to be rebedded was, at a minimum, to have been exposed from the point where it connects to the 36-inch line to a point approximately even with the southwest edge of the CWIS. The existing 36-inch pipe to be replaced would have been cut from the point where it connects to the 26-inch pipe and at a point inside the SWPS near the penetration. Any 36-inch pipe which has already been replaced and temporarily covered would again have been exposed.[97] The soil beneath all the pipes, within the limits referenced *supra* in Finding 237, would have been removed and replaced with the fly ash concrete discussed *supra* in Finding 233. Before being rebedded, the pipe was to have been inspected to verify the integrity of the pipe and the external corrosion coating, and then encased in compressible material where applicable. Lewis, ff. Tr. 8868, at 15.

239. All pipe would have been fabricated and installed in accordance with design drawings and specifications and in accordance with the Work Authorization Procedure established as a result of our April 30, 1982

[97] Because of the Applicant's need for the 36-inch pipe in meeting its startup test schedules, portions of this pipe might have been replaced, and then temporarily backfilled for frost protection. *See* Lewis, ff. Tr. 8868, at 15.

Order, LBP-82-35, *supra*.[98] All material used to replace unsuitable fill and to backfill the excavation was planned to be placed in accordance with design drawings and specifications. Lewis, ff. Tr. 8868, at 15.

240. The Applicant has performed dynamic seismic analyses of the buried SWS piping which has been or will be reinstalled. These analyses, performed using Bechtel Associates' ME-010 computer code, analyzed the piping for appropriate ASME load combinations and certain single nonrepeated anchor movement. ASME Code Equations 8, 9, and 10 and Code Case 1606-1, which were utilized by Applicant in the analyses, address stresses due to design and peak pressure, weight and sustained loads (including overburden), seismic inertial loads, thermal expansion and seismic anchor movements. The ME-101 analysis[99] incorporated the FSAR SSE as input. As indicated *supra* in Finding 219, even though the FSAR SSE (0.12*g* ground motion) was used in this analysis, the input spectra are more conservative than the SSRS; moreover, a check analysis using approved BC-TOP-4A techniques and 1.5 x FSAR SSE as input was to be carried out. Lewis, ff. Tr. 8868, at 12-14 and Table 4; Affidavit of Thiru R. Thiruvengadam dated January 21, 1983 (Enclosure E to Applicant's Letter to Board, dated February 3, 1983). Finally, the Applicant had planned to include Seismic Category I underground piping in its seismic margin review. *See* Letter from Philip P. Steptoe (Applicant's counsel) to Board, dated February 3, 1983, Enclosure A.

(ii) Diesel Fuel Oil Lines

241. The diesel fuel oil lines include four 1½-inch-diameter pipes and four 2-inch-diameter ASME Code[100] Class 3 carbon steel pipes. These lines were to provide fuel oil supply and return between the emergency diesel generators and four buried fuel oil storage tanks located east of the condensate storage tanks. SSER # 2, Table 3.1 and § 3.9.3.1.1, at 3-33 and 3-34; Landers, *et al.*, ff. Tr. 7619, at 5, 7.

242. These lines were initially installed in June 1980, after completion of the DGB surcharge program. They were attached to unistrut support frames embedded in concrete piers, which are located at approximately 20-foot intervals. Both piping and supports are covered with approximately 2 feet of compacted fill and were to be provided with tornado-missile protection. SSER # 2, § 3.9.3.1.1, at p. 3-34.

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[98] *See also* Bird/Wheeler, ff. Tr. 11,408, at 9.

[99] Bechtel computer program ME-101 is described in FSAR § 3.9.1.2.

[100] ASME Boiler and Pressure Vessel Code, § III (1980 Ed., with Addenda through Winter 1981).

243. The maximum settlement stress of the diesel fuel piping has been calculated assuming that the maximum value of 3 inches of predicted settlement was apportioned over a span of pipe corresponding to the maximum spacing between pipe footings. The highest calculated stress value was 18 ksi. This value is well within the allowable stress of 45 ksi for these lines under the 1977 ASME Code. Further, the pipes would settle with the diesel fuel oil storage tanks, and thus the differential settlement between the pipes and tanks would be small. Landers, *et al.*, ff. Tr. 7619, at 11.

244. Subject to the outcome of a seismic margin review (*see* Finding 240), the Licensing Board finds that this flexible small-diameter pipe in the diesel fuel lines could safely accommodate future plant fill settlement.[101]

(iii) Borated Water Piping

245. The borated water lines include four 18-inch pipes constructed of ASME SA-358, Grade 304 stainless steel and installed in accordance with ASME Code Class 2. They were to provide water from the borated water storage tanks (BWST) for normal functions, emergency volume and reactivity control and for such postulated accidents as a pipe break in the reactor coolant system. SSER # 2, Table 3.1 and § 3.9.3.1.1, at p. 3-34; Landers, *et al.*, ff. Tr. 7619, at 5-6, 7.

246. Profile data obtained in 1979 and 1981 show that these lines are below their design elevation by up to 2 inches, the maximum deviation allowed for under the construction tolerances. However, with the exception of the portions of the lines discussed below, the differential settlement effects for these lines have been evaluated, and the NRC Staff has found the effects of past and projected future settlement to be acceptable. SSER # 2, § 3.9.3.a.1, at p. 3-34.

247. The portions of the four 18-inch-diameter borated water lines from the BWST valve pits to the dike wall around the outdoor tanks were to be rebedded. These lines have been cut loose from the valve pits to isolate them from settlement caused by the surcharge of the valve pits, and have been refitted and recentered in the valve pit penetrations. Stress analyses based on the profile data for these lines

[101] By copy of a letter from the Staff to CPC, dated June 20, 1984, we were informed that the Applicant had sought, and the Staff had approved, the removal and replacement of at least some (and possibly all) of the diesel fuel oil lines. As long as procedures prescribed by LBP-82-35, *supra*, were followed, and as long as SSRS criteria govern the analysis of new piping, we find no objection to this change of plans for corrective action.

satisfy the $3S_c$ criterion accepted by the Staff. However, monitoring programs were to be implemented at the ends of the piping to address rattlespace concerns. Pipe strain only would have been monitored at the valve pit penetrations. Pipe strain and minimum rattlespace dimension would have been monitored at the auxiliary building penetrations. The maximum additional ovality and minimum rattlespace dimension were to be limited to 4% and 0.5 inch, respectively, throughout the projected life of the plant. The current minimum rattlespace dimension at any penetration is 1-7/8 inches. SSER # 2, § 3.9.3.1.4, at p. 3-40; Landers, *et al.*, ff. Tr. 7619, at 12.

248. Subject to the outcome of a seismic margin review (*see* Finding 240), the Board agrees with the Applicant and Staff that the foregoing partial rebedding and recentering of borated water lines in conjunction with the proposed monitoring program for the BWSTs and the auxiliary building (including the rattlespace monitoring described above) would provide sufficient assurance of the continued serviceability of this piping.

(iv) Control Room Pressurization Lines

249. Piping in the control room pressurization system includes one 4-inch ASME Code Class 3 carbon steel pipe and one 1-inch ASME stainless steel pipe. This system would supply overpressurization air to the main control room from two tanks buried to the east of the auxiliary building, during postulated accidents such as releases of hazardous gases from offsite storage areas. SSER # 2, § 3.9.3.1.1, at p. 3-34, and Table 3.1; Landers, *et al.*, ff. Tr. 7619, at 6, 7; *see also* SSER # 2, § 2.6.4.4.5, Fig. 2.11.

250. These lines were installed in 1981, after major fill settlements had occurred and in a manner equivalent to that utilized for the rebedding of other piping. The future differential settlement effects were expected to be negligible. SSER # 2, § 3.9.3.1.1, at p. 3-34; Landers, *et al.*, ff. Tr. 7619, at 33. Therefore, subject to the outcome of a seismic margin review (*see* Finding 240), the Licensing Board finds that there would be reasonable assurance of continued serviceability of the pipes in this system.

(v) Penetration Pressurization Lines

251. The fifth type of Seismic Category I piping includes two 1-inch-diameter ASME Code Class 2 carbon steel penetration pressurization lines. These lines had not been installed as of November 1982 (the

month during which the latest hearings on underground piping were held). SSER # 2, § 3.9.3.1.1, at p. 3-34 and Table 3.1.

252. The majority of fill settlement would already have occurred before these pipes were to be installed. The effects of differential settlement therefore should be negligible. SSER # 2, § 3.9.3.1.1, at p. 3-34. Moreover, installation of these pipes would be governed by procedures instituted as a result of our April 30, 1982 Order, LBP-82-35, *supra*. Accordingly, and subject to the outcome of a seismic margin review (*see* Finding 240), we agree with the Applicant and Staff that there is reasonable assurance of the continued serviceability of these penetration pressurization lines.

(vi) The Monitoring Program

253. Effective monitoring of Seismic Category I piping, particularly SWS piping, is a necessary step for assuring that such piping would remain serviceable for the life of the facility in the face of the differential soil settlement conditions which have been present in the past and the lack of sufficient records to ascertain the exact amount of settlement caused by soil settlement and imperfect installation, respectively. *See supra* Finding 210. Both strain gage monitoring and vertical settlement monitoring were to be employed.

STRAIN-GAGE MONITORING

254. To ascertain the effect of future soil settlement, externally mounted strain gage instruments would be located at various points along the SWS system. The SWS piping was to be monitored by strain gages because it is the most critical piping in terms of its response to soil settlement, and because of the necessity of the strain measurements in computing ovality. SSER # 2, § 3.9.3.1.3, at p. 3-39; Landers, *et al.*, ff. Tr. 7619, at 33; Tr. 7673 (Lewis). The strain gages would be located at positions along the piping where the greatest settlement, and hence the most stress, would likely occur. The Applicant took the position that the maximum differential settlement along the longitudinal axis of buried piping would occur at anchor points, and that the maximum critical differential settlement expected along buried piping would be the difference between the future projected settlement of the building entered at the anchor locations and the maximum estimated settlement of the fill in which the pipeline is buried. Landers, *et al.*, ff. Tr. 7619, at 10. On the other hand, the Staff took the position that, due to the variable soil

properties, maximum differential settlement could occur at any point along the length of the piping — and particularly where local soft spots are adjacent to high spots, as where conduit is located beneath the pipe. Tr. 7765-66, 7864-65 (Chen). Since the Staff conservatively required strain and settlement monitors wherever it believed there could be a potential problem (based on its review of soil profiles prepared along the line of the underground piping), and because the Applicant agreed to those locations, the question is moot as to precisely where one would expect to find the maximum differential settlement. Tr. 9086, 9088-91 (Kane); SSER # 2, § 2.5.4.6.2, at p. 2-52, and § 3.9.3.1.3, at 3-39 to 3-40.

255. A curve derived theoretically would be used to determine the equivalent strains for the allowable ovality and the actual ovality data measured on the Midland 26-inch-diameter SWS piping. Allowable ovality for the pipe is 4%, which is equivalent to 0.0048 inch/inch strain and which includes an appropriate safety factor, as discussed *supra* in Findings 222-223. Using the curve, the ovalization data measured in the 26-inch-diameter pipe would be transformed to an equivalent strain. This equivalent strain value would then be subtracted from the allowable strain to determine the future maxima for the strain monitoring stations. Lewis, ff. Tr. 8868, at 4 and Fig. 1; Tr. 7637 (Lewis).

256. Table 1 to the Lewis testimony shows the measured ovality, corresponding meridional strain, and future allowable strain for all strain monitoring stations on the buried Midland Seismic Category I piping, as well as the number of gages for each station. The method used to calculate the future allowable strain would allow the pipe strain resulting from soil settlement occurring before the 1981 data to be accounted for at each station. The number of gages was determined by reviewing the pipe elevation profiles for abrupt inflection points and critical buckling zones. Each such station would include at least two gages, thus providing redundancy. The strain gages would be mounted 1 pipe diameter apart along the top line of each pipe. Lewis, ff. Tr. 8868, at 4, Fig. 1 and Table 1; Tr. 7736-37 (Lewis); Tr. 9023-25 (Kane, Chen).

257. The strain gages would be used, and would be necessary, throughout the life of the plant (as much as 40 years). Although the gages represent the "state of the art" in such equipment, existing records verify their effectiveness only for periods up to about 20 years. Moreover, within the scope of such records, problems have been raised concerning the reliability of those gages and the length of time they may be expected to provide reliable information. For example, certain gages failed to give accurate readings after about 3-5 years for reasons such as relaxation of the wire in the gages or movement of the anchors. For that

reason, the use of strain gages necessitates an adequate monitoring program for the gages themselves, which would extend throughout the period (i.e., plant life) when strain gages would be used and, as necessary or appropriate, requiring repair or replacement of the gages. (For further details, *see infra* Finding 263.) Tr. 7704-05, 7738-39 (Lewis); Tr. 7763-64, 7880-82 (Kane).

VERTICAL SETTLEMENT MONITORING

258. Vertical settlement markers were added to various monitoring stations to supplement the pipe strain gage measurements. These markers have been installed where loosely compacted soil may exist, based on borings taken throughout the plant site fill material, and where high future differential settlement could potentially occur due to underlying utilities. Figure 2 to the testimony of Mr. Lewis is a monitoring station location diagram for both strain gage monitors and settlement markers. Figure 3 to the Lewis testimony shows a typical pipe settlement marker which would be attached directly to the pipe. Lewis, ff. Tr. 8868, at 5, and Figs. 2 and 3; SSER # 2, § 2.5.4.6.2, at p. 2-52. We understand the locations of these markers to incorporate the locations determined by the Staff to be necessary, as set forth *supra* in Finding 211.

259. The vertical settlement measurements were to be based upon the initial installation survey of the markers. This survey would establish an elevation datum. Subsequent surveys would be compared against this datum to calculate the pipe movements. The differential vertical displacement from the initial datum to the current survey measurement would be used for comparison to the acceptance criterion discussed *infra* in Finding 262. This acceptance criterion is based on the prediction of 3 inches of predicted maximum future settlement (*supra* Finding 213). Lewis, ff. Tr. 8868, at 5.

260. The vertical settlement markers measure the absolute pipe settlement at each monitoring station, rather than the differential settlement between stations. If settlement at any one station reaches or exceeds the acceptance criterion discussed *infra* in Finding 262, an investigation would be called for under the proposed technical specifications. In addition, where any station reaches or exceeds an "alert level" of 75% of the 3-inch acceptance criterion, the NRC Staff is to be notified. *Ibid.* The combination of strain gages and settlement markers at each monitoring station, together with the foregoing alert-level reporting requirement, would ensure that differential settlement would be detected and proper actions taken before stresses exceed the allowable limits. Lewis, ff. Tr. 8868, at 5-6; Tr. 8869-72 (Lewis).

STRAIN AND SETTLEMENT MONITORING FREQUENCY

261. The proposed measuring frequency for the monitoring stations was the same for both strain gages and vertical settlement markers. Monitoring would commence after the gages and markers were installed and operational. The monitoring schedule that was proposed by the Applicant is as follows:

> 1. At least once each 30 days during the first 6 months of unit operation. The frequency will continue until observed settlement has stabilized at less than or equal to 0.10 inches from the previous reading.
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> 2. When observed settlement stabilizes as discussed in (1), above, the monitoring frequency will decrease to at least once each 90 days during the first 5 years of plant operation for all stations. After the fifth year, the Applicant will file a report with the NRC on the need to continue monitoring of the field stations. This report will be based upon the evaluation of time history plots of the collected data.
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> 3. After the fifth year of plant operation, anchor stations will be monitored on a yearly basis for the remaining plant operating life.
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> 4. In the event of an unusual event, the Applicant will immediately monitor all stations.
>
> 5. In the event of a reportable occurrence, the Applicant will increase monitoring frequency as is determined necessary by the Applicant and the NRC.

Lewis, ff. Tr. 8868, at 6-7; Tr. 8873-75 (Lewis); SSER # 2, § 2.5.4.6.2, at p. 2-52.

PROPOSED TECHNICAL SPECIFICATION ACCEPTANCE CRITERIA AND ACTIONS

262. Under the Applicant's proposed technical specifications, if either the future allowable strain specified in Table 1 to the Lewis testimony or 75% of the 3-inch vertical settlement criterion were reached, this would constitute a reportable occurrence. Increased monitoring frequency would thereafter be required, the NRC would be notified of the occurrence and an engineering evaluation of the situation would be initiated. Supplemental reports to the NRC would follow the initial notification to describe the final resolution and actions. Such actions might include excavation of piping in the affected zone for visual examination and possible replacement or sleeving. Strain gages determined to be providing faulty data would be recalibrated or replaced within 90 days during the first 5 years of monitoring. Lewis, ff. Tr. 8868, at 5.

263. Based on our earlier findings, should plant operation be contemplated, the following guidelines should also be factored into license or permit requirements to be imposed by the Staff:

1. No monitoring schedule is proposed for the period between the commencement of monitoring (i.e., after gages and markers are installed and operational) and the commencement of unit operation. Since the degree of pipe settlement at any period of time is relevant, and since settlement resulting from defective installation, if any, would likely occur at an early date, the Applicant and Staff should agree upon an appropriate monitoring schedule for pipe settlement during the period between the commencement of monitoring and the initiation of unit operation.
2. To accommodate the usage of strain gages beyond the first five years of monitoring and throughout plant life, if necessary, the requirement for repair or replacement of gages which are determined to be providing faulty data (*see supra* Finding 262) should be supplemented by extending it for the life of the plant, on a schedule to be determined by the Staff.
3. The monitoring schedule proposed for the period of "plant" operation does not appear to take into account any extended period of time between the startup of Units 2 and 1, respectively. Nothing herein is to be taken to preclude the Staff, in the event a second unit were to be operated, from imposing additional monitoring requirements following the startup of the first unit, if appropriate.

RATTLESPACE MONITORING

264. To assure continuing adequate rattlespace clearance, the Applicant proposed monitoring the clearances of piping penetrations into buildings, but only where the pipes involved had not been rebedded and re-analyzed. As required by the minimum rattlespace criteria discussed *supra* in Findings 224-226, the soil settlement, seismic, and thermal displacements would be combined and compared to the available annular space to ensure at least a 0.5-inch safety margin. The Applicant proposed that the designated rattlespaces be monitored on a yearly basis for the first 5 years of plant operation, and that a determination then be made as to the necessity of continued monitoring Lewis, ff. Tr. 8868, at 5; App. FOF, ¶ 380; *see also* FSAR, § 16, at p. 3/4.13-18. On the other hand, the Staff believes that the question of exactly which pipes should be monitored for rattlespace can be resolved as part of the Staff's review

of CPC's proposed technical specifications (Staff FOF, ¶ 395) and the Applicant offers no objection to this proposal (App. Reply FOF, ¶ 395). To the extent that the plants are to become operational, we will permit the Applicant and Staff to resolve this matter in the manner suggested by the Staff. In addition, with respect to the frequency of rattlespace monitoring, the technical specification should provide for annual monitoring throughout plant life, subject to modification after 5 years if requested by the Applicant and approved by the Staff (subject to normal requirements for effectuating a technical specification modification).

(vii) Laydown Loads and Safety-Grade Utilities

265. Load limits have been specified to prevent a surcharging effect resulting from laydown loads of long-term storage over buried safety-grade piping and conduits. Exclusion zones would be used to designate the affected safety-grade utility and the maximum allowable loads and time limits. The Applicant proposed technical specification limits based on an allowable surcharge settlement of 0.5 inch at a depth of 7 feet below the ground surface — the average buried pipe depth. Lewis, ff. Tr. 8868, at 7-8 and Table 2.

266. Based on questions raised by the Staff as to this proposal (Tr. 8999, 9011-12 (Kane), we express no opinion at this time concerning the adequacy of the proposed technical specification limits. Should plant operation ever again be contemplated, the precise technical specification limits may be worked out by the Applicant and Staff during the Staff's review of proposed technical specifications, but the specifications must provide an adequate margin of safety for the heaviest loads postulated to occur over buried piping and conduits (in terms of both weight and, if appropriate, time within which loads might remain in place). Tr. 7909-11 (Kane). The control procedure to administer these technical specifications would be handled in conjunction with the plant operating procedures for controlling heavy loads inside the plant. Lewis, ff. Tr. 8868, at 8.

(viii) Freezewall Concerns

267. The Applicant committed to providing a plan for addressing a Staff concern about differential settlement that arises from a modification to Applicant's originally proposed freezewall crossing design. The freezewall is a temporary underground barrier of frozen earth created for construction purposes to minimize ground water flowing into the areas

where underpinning excavations for the control tower, electrical penetration areas, and the feedwater isolation valve pit are taking place. There is a potential for differential settlements where piping or conduit crosses the freezewall. The Applicant had planned to submit information that describes the crossing modification, details on surcharging the piping and conduit foundations during ground freezing, and the monitoring records on heave and/or settlement. Details on backfilling the excavations at the freezewall crossings would also have been provided by the Applicant. SSER # 2, § 2.5.4.4.5, at p. 2-36.

(c) Corrosion

268. As indicated earlier (Findings 245 and 249), there are two types of Seismic Category I underground piping which are composed of stainless steel: the borated water lines and one of the control room pressurization lines. The remainder of such piping is composed of carbon steel. *See also* Tr. 7832 (Hood). The Applicant initially relied to some extent on the use of these materials to resist potential corrosion. Tr. 7859-60 (Hood, *citing* § 9.21 of the FSAR, Rev. 30, dated October 1980, at 9.2-7). Nonetheless, pitting corrosion was discovered with respect to a portion of certain nonsafety stainless steel piping (Stamiris Exh. 35;[102] Tr. 7683-86 (Lewis); Tr. 7827-28 (Hood); Lewis, ff. Tr. 8868, at 16-17.

269. At the Board's request the Staff presented an expert witness on corrosion. That witness was Dr. John R. Weeks, a Senior Metallurgist at Brookhaven National Laboratory, where he has been employed since 1953. His responsibilities include experimental investigations on the mechanisms of stress corrosion cracking and pitting corrosion of stainless steels. Weeks, ff. Tr. 9147. Dr. Weeks, who prepared and sponsored the section of the Staff's Safety Evaluation (SSER # 2, § 3.12) dealing with corrosion of underground piping, addressed potential corrosion in both stainless steel and carbon steel piping. Tr. 9148 (Weeks).

270. All carbon steel piping used in the service water and diesel fuel lines was to be protected from corrosion by a combination of a primer paint and a protective wrapping to provide electrical insulation as well as a physical barrier between the piping and the corrosive environment. There were procedures for both shop coating of piping and field coating of field welds to ensure that this piping would be protected from external corrosion. In addition, the piping has been 100% inspected by Bechtel

[102] Stamiris Exh. 35 was admitted subject to the qualification that certain handwritten notes on the face of the document, which had not been authenticated, were not to be regarded as evidence (Tr. 7836-37).

for defects in the coating. Bechtel inspectors have determined that the coatings are acceptable. SSER # 2, § 3.12.1, at p. 3-42; Tr. 8877, 8882-84 (Lewis); *see also* Tr. 9394-95 (Weeks).

271. The buried pipe wrapping material consists of reinforced fiberglass followed by a layer of coal-tar-saturated felt paper wrap for the shop-coated material, and by a field-installed tape coat for the field-coated material. Both techniques are standard commercial practices for protecting carbon steel piping from ground water attack. Field installation and backfill techniques were carefully specified to minimize damage to the coatings. These procedures were also monitored by the Bechtel quality assurance department. Contrary to the claim of Ms. Stamiris, the pipe wrapping materials would not be subject to degradation due to differential settlement bending, inasmuch as they are inherently flexible and should not fail as a result of the amount of strain that might occur in the piping. Moreover, an independent check of the condition of the pipe wrappings would be possible when the 36-inch pipes are excavated and replaced before startup of the plant. *See* SSER # 2, § 3.12.1, at p. 3-42; Tr. 9146-49, 9159-60, 9209-12 (Weeks). The Board directs that this check be undertaken, to the extent that excavation were to occur following issuance of this Decision.[103]

272. The entire Midland site was to be protected by a galvanic protection system designed to maintain all buried piping to a potential of 0.85 V negative to the copper/copper sulfate reference electrode. This is a standard industry practice intended to ensure that, should any defects develop in the protective coating of these pipes, localized corrosion would not occur. This galvanic protection system consists of an array of buried electrodes charged from a central rectifier, as well as zinc protective anodes that can be used both for controlling corrosion and for monitoring the effectiveness of the applied galvanic current protection system. SSER # 2, § 3.12.1, at p. 3-42; Tr. 9168 (Weeks); Tr. 9222-34 (Woodby).

273. The galvanic protection system, as originally installed, included approximately 120 buried anodes. At the request of the site geotechnical engineer, concrete was used as backfill material for the installation of approximately fourteen anodes located near the BWSTs and to the south of the DGB. This practice was discontinued soon after it started, however, and no further anodes were encased in concrete, because of a concern

[103] Since both the Applicant and Staff assert that we should give credit to this possibility of checking of the condition of the pipe wrappings, we are doing so but are directing that it be undertaken to the extent it is still feasible to do so.

that the concrete would insulate the anodes and diminish their effectiveness. In further response to this concern, the concrete-embedded anodes were tested and shown to be performing within acceptable limits. Tr. 9223-25, 9256 (Woodby).

274. Well-founded concerns do exist, however, about the ability of concrete-encased anodes to function in the future. One reason that the concrete-encased anodes have functioned well is the high porosity of the concrete (Tr. 9304 (R. Cook)). Should the concrete become dry, however, it would act as an insulator, thereby defeating the purpose of the anodes (Tr. 9225, 9256-57 (Woodby)). The satisfactory performance of the concrete-encased anodes can also be attributed to the fact that the resistivities of the soil and concrete are about equal. If the site were to be flooded with water of higher conductivity, the concrete-encased anodes might not be as effective. Tr. 9303 (Weeks). For these reasons, the Applicant had planned to abandon the concrete-encased anodes, even though they had been shown to operate properly. The Applicant would have replaced them with anodes placed in a material called "coke breeze," a byproduct of burning coal which would allow for adequate compaction and proper conductivity. Tr. 9226-27 (Woodby). Moreover, the Applicant had been upgrading the galvanic protection system by installing about 190 new anodes in addition to the approximately 106 that would continue to be in operation (Tr. 9223-27 (Woodby)).

275. The galvanic protection system has been in operation since November 1980. Readings were being taken from voltmeters located on the rectifiers of the system approximately every other day, and the entire system was inspected twice monthly. Tr. 9160 (Weeks); 9230-31, 9254-55 (Woodby); Tr. 10,601 (Hood). One potential concern about the system, raised by NRC resident inspector Ron Cook, was that it might promote corrosion. Dr. Weeks opined that the polarity of the DC current in the system would have to reverse to cause a corrosion problem (Tr. 9325 (Weeks)). We are not aware of a mechanism (and none is reflected in the record) by which such a reversal in polarity might occur.

276. Leaching tests on sand samples from the backfill used at the Midland site have shown only trace amounts of chlorides, and a pH greater than neutral (8.6 to 8.9). This combination should minimize the extent of corrosion that might occur should the galvanic protection system or the pipe wrappings not perform their job. Furthermore, corrosion effects on all underground piping at the Midland site would be minimized by the operation of the site dewatering system. This system, discussed *supra* at Findings 98-116, should keep ground water levels below the elevation of the buried piping. Moreover, it is not anticipated that any low-level radioactive waste contamination would lead to an increase

in external corrosion to buried pipe at the site. *See* SSER # 2, § 3.12.1, at p. 3-42; Tr. 9153, 9158, 9161-62, 9303-05 (Weeks).

277. Should the galvanic protection system become inoperative, and assuming there were flaws in the coating on carbon steel pipes, corrosion at such locations would not be serious for periods up to at least 6 months. This is because other elements of the corrosion protection system would still be in effect — i.e., the nonaggressive chemical properties of the fill, and the materials from which the piping is constructed. Buried piping at the Midland site is designed with a 1/16-inch corrosion allowance, and pitting depths would not exceed one-half this allowance in 6 months. SSER, # 2, § 3.12.2, at p. 3-43; *see also* Tr. 7744-45 (Landers); Tr. 9167, 9217, 9305-06 (Weeks); Tr. 10,602-03 (Hood). We note that, during plant construction, the galvanic protection system has periodically been shut down for extended periods of time. For example, the system was inoperative from February through August 1982. Tr. 9228-29 (Woodby). In July 1982, near the end of that period, excavation of a stainless steel line revealed no visible corrosion on that piece of piping (Tr. 9301 (Weeks)).

278. The pipe-coating materials, such as fiberglass wrap or a coal tar paper wrap, are inherently flexible and should not fail as a result of the amount of strain that might occur in the Midland site buried piping. The protective wraps can "give" within the maximum acceptable ovalization and strain limits set for the piping. *See supra* Findings 270, 271. Further, should flaws develop in the protective wrap, the galvanic protection system should prevent corrosion at such flaws. Therefore, assuming the system remains operative, it is not anticipated that significant localized corrosion of coated carbon steel piping would occur as a result of soils settlement. SSER # 2, § 3.12.2, at p. 3-42; Tr. 8903 (Lewis); Tr. 9217 (Weeks).

279. Buried stainless steel piping at the Midland site is not coated on the outside, but is protected from corrosion by the galvanic protection system. Following the discovery, during construction, of pitting in the Nonseismic Category I stainless steel piping from the condensate storage tanks (*see supra* Finding 268), two studies were performed to determine the causes of the pitting. In the first, which was undertaken in 1979, the Applicant's consultant (Bechtel National, Inc.) examined this piping and concluded that the corrosion was a highly localized pitting, present on only one side of the piping. In view of the good soil chemistry at the Midland site, it is unlikely that this pitting would have been caused by interaction between the piping and the soil before the galvanic protection system was activated. However, the consultant could not determine the cause of the pitting but noted the lack of "known electrical sources" in

the vicinity of the corroded pipe sections. Stamiris Exh. 36. Subsequently, in a study dated January 26, 1981, the Applicant's consultant (Bechtel Group, Inc.) performed another study which suggested that these corrosion pits were caused by stray currents resulting from improper grounding during field welding of other components at the Midland site (Stamiris Exh. 37). The Staff believes this to be a likely explanation for the pitting. SSER # 2, § 3.11.3, at p. 3-43; Tr. 8878-79, 8886, 8904 (Lewis); Tr. 9385, 9434-35 (Weeks).

280. Although the recommendations of the two studies vary, it is significant that the experimental findings of the two studies were similar. *Cf.* Stamiris Exh. 37, at 2, with Stamiris Exh. 36; *see also* Tr. 9176 (Weeks). The different conclusions were attributed by Dr. Weeks to different investigators (including the contribution to the second report of a project engineer expert in corrosion matters) and to the discovery by the authors of the 1981 report of poor field welding procedures which could have given rise to the corrosion which was discovered. Tr. 9176, 9180 (Weeks); Stamiris Exh. 37, at 2, 7-10. Dr. Weeks also explained how electrical current could have caused the corrosion investigated in the first report (Tr. 9434-35). We find Dr. Weeks' reconciliation of the two reports to be credible. Further, Dr. Weeks utilized the two reports only for their discussion of the soil chemistry and the pitting corrosion. He also relied on other information in performing his review, and he formed his own independent conclusions. Tr. 9165-66, 9173-74, 9352-53, 9384-85 (Weeks). Moreover, the inspections of substantial portions of the remaining buried piping (which have been or were planned to be undertaken) provide the best assurance of the adequacy of protection against external corrosion of the buried piping. Tr. 9386 (Weeks); Tr. 9212-14, 9216 (R. Cook); Stamiris Exh. 38.

281. Construction personnel were advised to exercise greater care in assuring a firm grounding path exists when welding was taking place. Further, selected lengths of buried stainless steel piping in the BWST lines were being excavated and examined to determine the condition of the external surface of this piping. During the summer of 1982, all portions of the line that could be readily excavated were examined. The pipe came from the same area where at least one example of pitting had previously been found. During this inspection, no pitting was discovered. In addition, portions of the condensate storage lines have already been examined during the Applicant's investigation. The Applicant and the Staff have concluded that this proposed inspection followed by replacement of any defective piping will ensure the integrity of these systems. *See* SSER # 2, § 3.12.3, at p. 3-43; Tr. 8879-81 (Lewis); Tr. 9435, 9442 (Weeks). The Applicant and the Staff have also concluded

that the galvanic protection system now in place will help prevent any future external corrosion of stainless steel piping. *See* SSER # 2, § 3.12.3, at p. 3-43; Tr. 9160, 9168-69 (Weeks). Were the system to become inoperative and plant construction were later resumed, additional analysis of the corrosion of underground piping might be required.

C. Nonseismic Category I Piping

282. As set forth *supra* in Findings 90, 94, 97, if the Midland site permanent dewatering system lowers and maintains ground water levels below elevation 610 feet in the vicinity of the DGB and the railroad bay area of the auxiliary building, there will be no danger due to liquefaction at the site resulting from earthquakes equal to or smaller than the SSE.[104] At the Staff's request, the Applicant analyzed breaks in Nonseismic Category I underground piping to determine the effects of such breaks on the ability of the permanent dewatering system to maintain water levels below elevation 610 feet in these areas. SER, § 2.4.6.3, at 2-28 and 2-29.

283. Several Nonseismic Category I lines, called circulating water discharge lines (CWDL), are located to the east and west of the DGB, about 18 feet below the DGB's continuous reinforced concrete footings (SSER # 2, § 2.5.4.4.2, at p. 2-24; § 3.8.3.4, at p. 3-22; *see* FSAR Fig. 2.5-177 for the location of this piping). In this area, the dewatering system would normally control the ground water level to elevation 595 feet. The Applicant performed an analysis of a postulated failure of the Unit 2 CWDL (the largest Nonseismic Category I underground pipe near a critical structure). *See* Paris, ff. Tr. 9900, at 34; Tr. 9938-43 (Paris); SER, § 2.4.6.3, at p. 2-28. This analysis established that the ground water level would rise to elevation 607 feet over a period of approximately 3.3 days before the closest area dewatering well would automatically activate. Thereafter, operation of only one well would be sufficient to prevent ground water from rising significantly above elevation 610 feet. However, should all the area dewatering wells be inoperable at the time of the pipe break, the rising ground water would trigger the permanent dewatering monitoring system, resulting in appropriate actions under the proposed technical specifications. Moreover, since the top of the Unit 2 CWDL is at elevation 610 feet, ground water levels are not expected to rise significantly above this elevation as a result of a

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[104] The potential for liquefaction in areas to the north and west of the SWPS is being dealt with by replacement of loose sands in those areas. *See supra* Findings 90, 95.

CWDL break. *See* SER, § 2.4.6.3, at 2-28 to 2-29; Tr. 9938-43 (Paris). *See also* discussion, *supra* Findings 110 and 112-113.

284. The Applicant also analyzed the Nonseismic Category I condensate storage lines (CSLs) for a postulated failure. These lines consist of the two 20-inch-diameter supply lines and two 6-inch-diameter return lines that run from the condensate storage tanks (CSTs) located near the southeast corner of the DGB, underneath the DGB to the condensers located in the turbine building. SER, § 2.4.6.3, at p. 2-29.[105]

285. Prior to the placement of the DGB surcharge, the Applicant committed to monitoring the CSLs so as to evaluate pressures imposed on the line by the surcharge (Tr. 4404-06 (Kane); Tr. 2455-56 (Gallagher)). In addition, both CSLs were severed on the north side of the DGB at a point between the DGB and the turbine building so as to relieve stresses on the line and to the DGB due to settlement. (Some consideration was given to severing *both* ends of the CSLs, but apparently that course of action was not carried out.) *See, e.g.*, Tr. 4199-4200 (Hood). As a result of its analysis, the Applicant has concluded, and the Staff concurs, that, if any of the CSLs were to break so that the entire liquid inventory of the affected CST were to drain out through the break and remain in the area directly beneath the DGB, the ground water would not exceed elevation 610 feet even if the area dewatering wells were not operational. *See* SER, § 2.4.6.3, at p. 2-29; SSER # 2, § 2.4.6.3, at p. 2-5; *see also* discussion, *supra* Finding 110.

286. The Applicant has also evaluated a postulated break in a dewatering system header line. In this event, inflow of water could exceed the capacity of the affected dewatering pumps, producing a rise in ground water in the immediate vicinity of the affected wells. The installation of flexible header diversion hoses and backup interceptor wells provides reasonable assurance that ground water levels will not rise above elevation 610 feet. *See* SSER # 2, § 2.4.6.3, at 2-5 to 2-6; *see also supra* Finding 110.

287. A break in the 66-inch concrete cooling pond blowdown line would have minimal impact on ground water levels because of the low-pressure delivery of this line. The dewatering system has sufficient capacity to remove the volume that would be introduced into the ground water due to a rupture in this line. SSER # 2, § 2.4.6.3, at p. 2-7; Paris, ff. Tr. 9900, at 33.

288. CPC advises that its letter to the Staff of March 16, 1982 (File 0485.16, Serial 16269, not introduced into evidence) identified a 10-foot

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[105] *See* SSER # 2, Fig. 2.11 for the location of the CSL, designated 20"-1HDC-169, and the two CSTs. Figure 2.11, however, is inaccurate in that it indicates only one out of the four CSLs. Tr. 9123.

length of 48-inch-diameter line extending from the SWPS which, at the time, was classified by the Appliant as Seismic Category I (*see* App. FOF, ¶ 324, at 223 n.574). The Applicant later reclassified this portion of the 48-inch-diameter line as Nonseismic Category I. The NRC Staff concurred with the reclassification and agreed that failure of this 48-inch-diameter line would not cause a loss of essential SWS cooling. SSER # 2, § 9.2.1, at 9-1; *see also id.* § 3.9.3.1.1, at 3-32 to 3-33.

D. Conclusions with Respect to Underground Piping

289. The Licensing Board concludes that, although adequate analyses had not been completed at the time of the submission of Stamiris Contention No. 4.A(4) and Warren Contention 3, the Applicant has now adequately taken into account the effects of the preloading of the DGB on underlying piping. All pipes in the vicinity of the DGB have been analyzed for adverse effects due to the preload, and (assuming resumption of the project) conservative rattlespace monitoring requirements are to be required. Some piping, such as the diesel fuel oil lines, was not installed until *after* the preload, and thus was not subjected to preload stresses. Other piping, such as the condensate storage lines, had been installed prior to the preload but were severed so as to relieve stresses to the pipes and to the DGB.

290. The Licensing Board similarly concludes that, although Stamiris Contention No. 4.C(f) was to some extent meritorious at the time of its submission, the Applicant has now adequately evaluated the effects of differential settlement, dewatering-induced settlement and seismic settlement on buried piping. The Applicant and the NRC Staff have presented extensive testimony and numerous exhibits outlining the remedial actions and analyses which have been performed on the buried piping with respect to soils settlement. Moreover, the comprehensive monitoring program, which has been described *supra* in Findings 253-264, would provide additional assurance that Seismic Category I piping would continue to be safe throughout the operating life of the plant. In the event of plant operation, should settlement of Seismic Category I underground piping greater than predicted occur, the Applicant would be required to report such settlement and take corrective action prior to the point where settlement might affect the ability of that pipe to perform its intended function.

291. The Licensing Board further concludes that, under the programs described by the Applicant and Staff, there is reasonable assurance that the underground piping at the Midland site would be adequately protected from external corrosion. This conclusion is specifically subject to the

continued operation of the galvanic protection system; if the system were to become inoperative for extended periods, further analyses might be required.

292. Accordingly, the Licensing Board concludes that, so long as the proposed corrective actions (including replacement, rebedding, reinstallation, and monitoring, as appropriate) would be carried out satisfactorily (a question not considered in this Partial Initial Decision), there is reasonable assurance that Seismic Category I underground piping at the Midland site would be able to perform its intended functions and would not place undue risk on the public health and safety. Furthermore, there is reasonable assurance that postulated failures in Nonseismic Category I underground piping, were they to occur, would not adversely affect nearby Seismic Category I structures or piping.

VIII. ELECTRICAL DUCT BANKS AND CONDUITS

293. Stamiris Contention 4.C(f), as amended, states:

> 4. Consumers Power Company performed and proposed remedial actions regarding soils settlement that are inadequate as presented because:
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> * * *
>
> C. Remedial soil settlement actions are not based on adequate evaluation of dynamic responses regarding dewatering effects, differential soil settlement, and seismic effects for these structures:
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> * * *
>
> f. Related Underground Piping and Conduit.

Prehearing Conference Order, dated October 24, 1980, Appendix at 5-6, as supplemented by Ms. Stamiris' Answer to Applicant's Interrogatories, dated April 20, 1981. Similar safety-related concerns were expressed by former Intervenor Sharon Warren in her Contention 3 (quoted *supra* at note 93). Insofar as they relate to the electrical duct banks and conduits, they will be addressed here.

294. Seismic Category I buried electrical duct banks at the Midland Plant run under the turbine building from the auxiliary building to the DGB and to the SWPS. Others run north from the auxiliary building to the borated water storage tanks and to the control room pressurization tanks. A third group runs from the emergency diesel fuel oil storage tanks to the DGB. The duct banks are buried at depths from 3 to 40 feet below grade level. Their dimensions vary from 18 x 19 inches to 74 x 20 inches. Each duct bank is rectangular in cross-section, constructed of concrete with a minimum thickness of 6 inches, possessing a minimum compressive strength of 3000 psi, with a nominal amount of grade 60

steel as reinforcement to avoid surface cracking. The steel is asserted to serve no structural purpose (*but see infra* Finding 304). Plastic or steel conduits, 2 to 4 inches in diameter, are placed inside the electrical duct banks. Electrical cables are then pulled through this conduit. The electrical cables are placed loosely in the conduits which are only partially filled by the cable volume. The electrical cables, which are ductile and capable of considerable stretching before breaking, are suitable for direct burial in wet and dry earth, and have a 40-year service life without considering the presence of the duct banks. Rinaldi/Matra, ff. Tr. 7537, at 11; Shunmugavel, ff. Tr. 12,016, at 1-4, Appendix A; Tr. 12,023-31 (Shunmugavel).

295. The function of the electrical duct banks is only to provide a space in the ground through which Seismic Category I electrical cable may be pulled. They are not required to provide a water-tight pressure boundary around the electrical cables, and cracking of the duct banks due to differential settlement or the leakage of water does not affect their design function. Therefore, although the duct banks are usually referred to as Seismic Category I, they serve no structural function; it really is the cables within the duct banks which are Category I. The Applicant has analyzed these duct banks for normal conditions, construction conditions, settlements, and seismic effects. In addition, the Applicant has given special consideration to the duct banks which temporarily restrained DGB settlement to ensure that they had not been damaged by this loading history (*see infra* Finding 303). Rinaldi/Matra, ff. Tr. 7537, at 11; Shunmugavel, ff. Tr. 12,016, at 1-2, Appendix A; Tr. 12,020-22 (Shunmugavel).

296. Based on the function of duct banks, Dr. Palanichamy Shunmugavel, the Applicant's witness, developed conservative acceptance criteria to overcome various problems — e.g., to avoid concentrated shear deformation large enough to cut or damage the electrical cables. These criteria specify allowable values of shear deformation for 2-, 3- and 4-inch conduits filled to maxima of 20, 56 and 51%, respectively. Maximum allowable longitudinal cable-pulling tension and maximum bend radii were also specified. Shunmugavel, ff. Tr. 12,016, at 3; Tr. 12,021-22, 12,033-35 (Shunmugavel).

297. Dr. Shunmugavel testified that, during normal operating conditions where the duct banks are buried in the earth, soil overburden, surcharge and live loads from surface traffic would be absorbed by duct bank concrete and distributed to the soil around and below the duct bank. He concluded that, as a result, the cables inside the duct banks and conduits would never see the effects of these loads. Dr. Shunmugavel further testified that the duct banks have the capacity to span dis-

tances up to 10 feet without any soil support. A cracked duct spanning a 10-foot gap might require some support; however voids are not expected beneath the duct banks during the life of the plant. Shunmugavel, ff. Tr. 12,016, at 2, 4; Tr. 12,027-35 (Shunmugavel).

298. Under construction conditions, the concrete duct banks are protected from nearby construction activity by the placement of sufficient earth cover over them. Notwithstanding such planned protection, however, on two separate occasions duct banks have been injured during construction because of drilling errors. These incidents have been extensively reviewed on this record, as part of the consideration of QA/management attitude issues (with which we are not dealing in this Decision).

The duct bank concrete and conduits protect the cable pathways from being obstructed by laitance (drippings of cement mixture or aggregate that can harden and form obstructions) and other trash from construction activity. To ensure that the electrical cable is protected when it is pulled through a duct, the duct is first cleaned and checked for continuity and obstructions by pulling a segmented hard-fiber-composition mandrel through it. Shunmugavel, ff. Tr. 12,016, at 2, 4 and Appendix A, Fig. 7-3; Tr. 12,023, 12,034 (Shunmugavel).

299. Where duct banks cross the freezewall constructed in conjunction with the installation of a dewatering system for the auxiliary building, the soil around and below the ducts has been removed in order to isolate the duct banks from the effects of freezing. Monitoring pits have also been installed. The portions of the ducts in the excavated pits were to be encircled with 6-inch-thick polyethylene planks and backfilled with fly-ash cement and compacted soils. The Staff has identified on page 2-36 of SSER # 2 the information required to be provided by the Applicant in regard to a modification of the originally proposed freezewall crossing design. The issue of duct banks crossing the freezewall was extensively covered during hearings in November and December 1983, in connection with an alleged violation of the Board's April 30, 1982 Order (LBP-82-35). That issue also is one of the QA/management attitude issues which are not being dealt with in this Decision. *See supra* p. 32.

Dr. Shunmugavel testified that during construction, when the present backfill was to be excavated and replaced in the area north of the SWPS, some of the duct banks in that area would be temporarily unsupported. These duct banks would then have been evaluated and temporary supports placed under them, if necessary, during the excavation process. Shunmugavel, ff. Tr. 12,016, at 5; Tr. 12,034 (Shunmugavel); SSER # 2, § 2.5.4.4.5, at p. 2-36 (second paragraph).

300. Dr. Shunmugavel also evaluated the integrity of the electrical duct banks and conduits under differential soil settlement conditions. He estimated that the maximum duct bank settlement from October 1978 through the year 2025 would be 3 inches, and also that this 3-inch maximum duct bank settlement would occur over a minimum distance of 25 feet. The 3-inch maximum duct bank settlement prediction takes into account secondary consolidation to the year 2025, settlement effects due to the temporary and permanent site dewatering systems, a 0.5-inch allowance for possible laydown loading and a 0.25-inch allowance for possible seismic shakedown settlement due to an earthquake with peak ground acceleration of 0.19*g*. The NRC Staff was in agreement with the estimates of differential soil settlement used in Dr. Shunmugavel's analysis.

A conservative evaluation performed by Dr. Shunmugavel based on the maximum allowable longitudinal cable strain of 0.333×10^{-3} indicated that the duct banks could actually tolerate up to 3 inches of differential settlement over as little as a 12-foot length. Based on this evaluation, the estimated maximum duct bank settlement of 3 inches over a 25-foot length during the plant's operating life could easily be accommodated.

Dr. Shunmugavel also testified that, except in one area, discussed *infra* in Finding 301, the electrical cables can accommodate the concentrated shear deformations which could result from the predicted differential soil settlement at various interfaces between the Midland Plant buildings and the duct banks. Shunmugavel, ff. Tr. 12,016, at 5-7; Tr. 12,028-29 (Shunmugavel); Tr. 12,075, 12,100 (Kane).

301. Results of Dr. Shunmugavel's evaluation indicate that there is a potential problem with concentrated shear deformations caused by differential interface settlements where seven duct banks enter the north wall of the SWPS. In addition, cables contained in one of these seven duct banks also exceed allowable concentrated shear deformations at the interface between the existing backfill material and the fly-ash cement mixture which will be used to replace certain liquefiable sands northwest of the SWPS.

To remedy this problem, a polyethylene called "Ethafoam" was to be wrapped around the duct banks in these areas to isolate them from the predicted concentrated shear deformations. The Ethafoam isolation would have occurred, subject to the NRC's work authorization procedure, at the same time the present backfill north of the SWPS was to be excavated and replaced with the fly-ash cement mixture. Dr. Shunmugavel testified that the 6-inch design thickness of the Ethafoam would be adequate to isolate the duct banks from the effects of shearing or any other load resulting from laydown equipment or traffic. Staff witness Frank Rinaldi expressed general agreement with the testimony of Dr. Shunmugavel. Responding to a question from the Board, Mr. Rinaldi agreed that Ethafoam would retain enough insulating capacity, even after dead and live loads are considered, because of its limited compressibility and the spreading out of surface loads with depth below the surface. In response to Board questions concerning the characteristics and use of Ethafoam, Dr. Shunmugavel did not have the requested data at hand; the Applicant accordingly agreed to provide an affidavit supplementing the response elicited in the record. That affidavit, which was distributed to the Board and the parties on August 8, 1983, constitutes a full answer to the Board's questions and a useful addition to Dr. Shunmugavel's testimony. Since neither the Staff nor Ms. Stamiris has objected to this affidavit, through proposed findings or otherwise, we are treating it as an integral part of the record on this topic. Shunmugavel, ff. Tr. 12,016, at 7-8; Tr. 12,017-19, 12,025-31 (Shunmugavel); Tr. 12,040-41, 12,046-47 (Rinaldi); Affidavit of Dr. Shunmugavel Concerning the Use of Ethafoam at Midland, dated August 2, 1983 (transmitted to Board and parties on August 8, 1983).

302. Finally, Dr. Shunmugavel conducted a seismic evaluation of the Category I electrical duct banks and conduits at the Midland site. Seismic compression, shear and surface wave effects were included in the evaluation. Using 1.5 times the ground response spectra for the FSAR SSE, Dr. Shunmugavel concluded that the maximum values determined for these duct bank sections are well within the allowable acceptance criteria for strain and concentrated shear deformations.

Seismic interactions between the buildings and duct banks could occur if clearances along the axial direction between the duct banks and the buildings were not sufficient to accommodate maximum relative seismic motion. Dr. Shunmugavel evaluated these clearances using 1.5 x FSAR SSE and determined that there was no problem from such seismic interaction at Midland. As noted previously, the acceptability of designs made on this basis for Seismic Category I structures is contingent on the satisfactory completion of a seismic margin review. Shunmugavel, ff. Tr. 12,016, at 8-9; Tr. 12,017-18 (Shunmugavel); Tr. 7540, 7558, 12,130-31 (Rinaldi).

303. Four Seismic Category I duct banks enter the DGB from below. For a period of time in 1978 due to the greater-than-anticipated settlement of the DGB and inadequate clearances between the duct banks and the building footings, these duct banks supported part of the weight of the DGB. In November 1978, Applicant eliminated this load transfer by increasing the clearances at the vertical joints between the duct banks

and the footings. In May 1980, after the DGB surcharge program, all of the conduits in the duct banks were checked and no obstruction or discontinuity was encountered. The cables were pulled through and placed in those conduits in 1981.

The Applicant analyzed the DGB duct banks and concluded that this one-time loading condition has not affected their ultimate strength. Since the duct banks are not required to provide a watertight boundary around the cables, any cracking caused by this episode would not affect their design function. The Category I cables have not been affected because they were not in place until after the DGB was isolated from the duct banks and after the surcharge of the DGB. Shunmugavei, ff. Tr. 12,016, at 8 and Appendix A; Tr. 12,021 (Shunmugavel); Tr. 12,109-10 (Rinaldi).

304. The NRC Staff expressed agreement with the Applicant's analysis of duct banks and conduits. Mr. Rinaldi testified that the Staff believes that the Applicant has adequately taken into account all dead, live and seismic loads in its evaluation of Category I buried electrical duct banks, conduits and cable at the Midland site. In response to a Board question, he cited a number of conservative aspects of this duct bank design in support of the above Staff belief, including not relying on the steel reinforcement in fact used, providing for unsupported spans far greater than reasonably expectable, and the use of fly-ash lean concrete as a support mixture instead of soil.

In responding to a relevant portion of Stamiris Contention 4.C(f) and Warren Contention 3, Mr. Rinaldi summarized testimony given in February 1982 expressing satisfaction with plans for meeting initial Staff concerns about electrical duct banks, subject to adequate documentation. This documentation has since been thoroughly audited by one of the Staff's consultants at the office of the CPC architect-engineer to verify the previous conclusions, and was found to be acceptable. Rinaldi, ff. Tr. 12,080, at 2, 8-10; Tr. 7554, 12,042, 12,045-46, 12,117-18 (Rinaldi).

305. The Licensing Board concludes, based on the foregoing findings, that the Applicant has adequately resolved the concerns raised in Stamiris Contention 4.C(f) relating to the remedial soils measures taken or planned for Seismic Category I duct banks and conduit at the Midland site. The Board finds reasonable assurance that they are capable of performing their intended safety function over the projected lifetime of the plant. This conclusion is subject to satisfactory completion of the remedial work north of the Service Water Pump Structure described *supra* in Finding 301, as well as to the satisfactory outcome of a seismic margin review (see *supra* Finding 302).

IX. SLOPE STABILITY OF BAFFLE AND PERIMETER DIKES[106]

306. The cooling pond enclosed within the perimeter dikes is a polygonal body of water approximately 880 acres in area, located south and east of the Midland Plant, which would have provided cooling water to the condensers during normal plant operation. The pond is bordered on the northeast by the Tittabawasee River. The pond design includes intake and outlet areas which are separated by a baffle dike to assure proper water circulation. The water level of the cooling pond during normal plant operation would be maintained at elevation 627 feet. The bottom of most of the cooling pond lies between elevations 605 and 610 feet.

The Emergency Cooling Water Reservoir (ECWR), located in the northeast corner of the cooling pond, is an area of the larger cooling pond which has been excavated in the natural soils to elevations ranging from 593 to 596 feet, below the original ground surface elevation of approximately 605 feet. The ECWR is classified Seismic Category I. In the event of the failure of the cooling pond perimeter dikes and the loss of the larger cooling pond reservoir, water for safe shutdown of the reactors and for mitigation of accident conditions is retained in the ECWR. The ECWR is designed to contain a sufficient volume of circulating water to cool the plant for a 30-day period without makeup. If the ECWR were used, return cooling water would be discharged to the ECWR through two 30-inch Seismic Category I reinforced concrete water pipes ("return pipes"). Hendron, ff. Tr. 3940, at 6-7, Figs. 1 and 2; Kane, ff. Tr. 3484, at 3; Tr. 3577-79 (Kane, Singh).

307. The ECWR is bounded on the southwest by the baffle dike, which separates the intake and outlet areas of the cooling pond. The ECWR area is bounded on the northeast by the upstream slope of the perimeter dike. The perimeter dike runs from the power block area down along the Tittabawasee River and extends into the cooling pond area. The Category I return pipes which drain into the ECWR exit from the SWPS and run along the southwest and northeast sides of the ECWR. On the southwest side, the return pipe runs along the base of and parallel to the slope of the baffle dike. On the northeast side, the

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106 The Staff would have us rely in part on SER §§ 2.5.6.1 through 2.5.6.7 (at 2-47 to 2-51) in our evaluation of the slope stability of the dikes. CPC objects, because of the lack of formal sponsorship of those sections and the consequent lack of a proper opportunity for cross-examination on their contents. We are noting these sections here since they are relevant and we find no area of conflict between them and our record. However, we do not depend on them to any significant extent in making our findings. *See Southern California Edison Co.* (San Onofre Nuclear Generating Station, Units 2 and 3), ALAB-717, 17 NRC 346, 365-68 (1983).

return pipe runs along a berm at the base of and parallel to the slope of the perimeter dike. The critical portions of the cooling pond dikes are those slopes adjacent to the ECWR which, if they moved, might deform these pipes. Where the perimeter dike separates the ECWR section of the cooling pond from the river it has been zoned and covered with an outer layer of riprap to reduce the action of river flow and river erosion and to ensure slope stability. To reduce water seepage into the perimeter dike from the river or the ECWR, a slurry trench tied into the impervious natural layer below the dike has been installed to prevent seepage into dike sands. Hendron, ff. Tr. 3940, at 7-8, Figs. 2, 5 and 6; Tr. 3526-27, 3529 (Kane).

308. The subsoils underneath the portions of the perimeter and baffle dikes adjacent to the ECWR consist of, from lower to higher elevations, dense water-bearing sands, a thick mantle of dense impervious glacial till, preconsolidated lacustrine clay, uniform silty sand, topsoil and surficial silt. The elevation of the surface of the glacial till is not uniform and pockets or layers of gravel, sand, silt and clays may lie between the glacial till and the preconsolidated lacustrine clay and topsoil. The presence of a layer of silty sand where glacial till had previously been assumed was confirmed by borings taken by Woodward Clyde Consultants (*see infra* Finding 312).

The topsoil and surficial silt were removed from beneath the entire dike embankment during the construction of the baffle and perimeter dikes. The soils composition of the baffle dike consists of both cohesive fill and some granular fill which has been designated in some reports as "random" fill, covered by layers of gravel and riprap. The composition of the perimeter dike consists of compacted cohesive fill, covered by layers of gravel, riprap, topsoil and seeding. Hendron, ff. Tr. 3940, at 13-15, 18, 22, Figs. 5 and 6; Singh, ff. Tr. 3488, at 5; Tr. 3496-97 (Singh).

309. Ms. Stamiris' Contention 4.B, as amended, raises several safety-related concerns, including one on cooling pond dikes. It states:

> 4. Consumers Power Company performed and proposed remedial actions regarding soils settlement that are inadequate as presented because:
>
> * * *
>
> B. Slope stability of cooling pond dikes is not assured because they were built with the same soils and procedures [as the soils foundation for the DGB].[107]

[107] The phrase "same soils and procedures" refers to Item A of Stamiris Contention 4 that alleges several soils and procedures problems at the diesel generator building.

Prehearing Conference Order dated October 24, 1980, Appendix at 5-6, as supplemented by Ms. Stamiris' Answer to Applicant's Interrogatories, dated April 20, 1981.

310. Safety-related concerns regarding the slope stability of the portions of the baffle and perimeter dikes adjacent to the ECWR originally arose due to the excess settlement of the DGB. When the NRC Staff realized that the settlement difficulties were attributable to inadequately compacted soils, the Staff reevaluated the construction of those portions of the baffle and perimeter dikes which could impinge upon the operation of the return pipes and the ECWR. The Staff's primary concern was whether the soils materials in those portions of the cooling pond dikes which could affect Seismic Category I equipment had sufficient shear strength properties to withstand the various loading conditions likely to be imposed on the dikes during plant operation.

Initial questions concerning the stability of these slopes were posed to CPC by the Staff through the Army Corps of Engineers. These included requests for a determination of the static factor of safety for the dike slopes which contain the two return pipes, a seismic analysis for these slopes, profiles across the dikes, and a discussion of the available shear strength data and the choice of shear strengths used in the stability analyses. In its November 1980, response, CPC identified the critical sections of the dike slopes, analyzed them for a static factor of safety and performed a pseudo-static analysis which indicated that the yield acceleration for the critical dike slopes exceeded the ground acceleration associated with the SSE. Kane, ff. Tr. 3484, at 2-3; Tr. 10,095, 10,105-07 (Hood); Hendron, ff. Tr. 3940, at 8-9, 17-18; Singh, ff. Tr. 3488, at 3-4.

311. The Army Corps of Engineers found CPC's answer to be not satisfactory in one respect: the Corps believed that the shear strength parameters used in Applicant's stability analyses might not be representative of actual dike soils conditions. In response, the Applicant contracted Woodward Clyde Consultants to perform a boring and sampling program of the portions of the baffle and perimeter dikes near the ECWR. The boring locations were selected by Army Corps of Engineer personnel, and were conducted by Woodward Clyde Consultants under the Army Corps' observation. The final results of the boring and sampling program were submitted to the Army Corps of Engineers and the NRC Staff in July 1981. On the basis of the boring samples and the previous CPC responses, the Army Corps of Engineers concluded that the fill material placed in the baffle and perimeter dikes exceeds its design parameters. Hendron, ff. Tr. 3940, at 9-10; Singh, ff. Tr. 3488, at 3-4.

312. The boring samples conducted by Woodward Clyde Consultants established the existence of a layer of silty sand below the dike where the presence of glacial till had been assumed. As a result, the Army Corps of Engineers could not reach a conclusion as to whether the stability of the slopes of the dikes adjacent to the ECWR would adversely affect the safe operation of the ECWR until the Applicant had demonstrated that the shear strength of the layer of silty sand equals or exceeds the parameters specified in the FSAR stability analysis. Dr. Alfred J. Hendron, a Professor at the University of Illinois, conducted an independent assessment on behalf of the Applicant to evaluate, among other things, the shear strength of the layer of silty sand. He concluded that the undrained shear strengths of this material are much stronger than the undrained shear strengths of the foundation clay. This estimate was confirmed by three triaxial tests conducted by Woodward Clyde Consultants on boring samples of this material.

Mr. Hari Singh, Staff witness from the Army Corps of Engineers, stated that Dr. Hendron's testimony establishes that the shear strength of the fine sand equals or exceeds previously specified soils strength parameters, and that he could therefore conclude that the slopes of the dike would remain stable under static loading conditions. Mr. Kane, another NRC Staff witness, concurred, testifying that the baffle and perimeter dikes' soils materials are no less resistant than the materials described in the PSAR. Singh, ff. Tr. 3488, at 5; Tr. 3489-94 (Singh); Staff Exh. 3; Hendron, ff. Tr. 3940, at 3-4, 22-23; Tr. 3960-61 (Hendron); Tr. 4140 (Kane).

313. Dr. Hendron's assessment evaluated the static factor of safety for the baffle and perimeter dikes adjacent to the ECWR. Further, Dr. Hendron evaluated the critical yield acceleration for these critical dike slopes under seismic loadings. Dr. Hendron also evaluated the stability of these critical dike slopes under the conditions of a rapid drawdown of the cooling pond water level from an elevation of 627 to 604 feet, in the extreme event that the perimeter dike would fail at some other location away from the ECWR. Mr. Singh testified that Dr. Hendron's analytical approach was in accordance with the accepted Army Corps of Engineers' manual and procedures.

Dr. Hendron's analyses evaluated the critical sections of the baffle and perimeter dikes and assumed the steepest slope. The critical portions of these dikes are the upstream slope of the northeast perimeter dike which inclines towards the ECWR and the northeast slope of the baffle dike which inclines northeast towards the ECWR. Movement in either of these slopes would tend to deform the return pipes and impair the operation of the ECWR.

The results of Dr. Hendron's analyses indicate that the soils materials in the critical portions of the baffle and perimeter dikes have sufficient shear strength and resistance to preclude lateral deformation of the dike slopes towards the ECWR. Hendron, ff. Tr. 3940, at 10-11, 17-21, 29-38 and Appendix A; Tr. 3942-51, 3987-95 (Hendron); Tr. 3655-58, 4114-19 (Singh).

314. The static factors of safety determined by Dr. Hendron for long-term stability in terms of "effective" stresses for the critical portions of the baffle and perimeter dikes are 2.18 and 2.66, respectively. These factors of safety greatly exceed the 1.5 factor of safety normally used in the design of dikes for nuclear power plants. They are conservatively calculated in that the effective cohesion for all materials is taken as zero, an effective angle of shearing resistance of 28.5° is used although measured values ranged from 28.5 to 35.0, and the shear strength parameters of the glacial till were taken equal to those of the foundation clays. Hendron, ff. Tr. 3940, at 17-25, 31-32; Tr. 3953-55, 3992-95 (Hendron); Tr. 3655-56 (Singh).

315. In the unlikely event that the perimeter dike were to fail at some location away from the ECWR, the rapid draining of cooling pond water into the Tittabawasee River could potentially cause the critical slopes of the baffle and perimeter dikes adjacent to the ECWR to slide. This phenomenon has been referred to as the "rapid drawdown."

Dr. Hendron performed two evaluations of dike stability for rapid drawdown of pond level from 627 to 604 feet. The first evaluation used values of undrained shear strength appropriate to cooling pond levels of 627 feet and yielded factors of safety for the critical portions of the baffle and perimeter dikes of 2.73 and 3.55, respectively. These values are significantly higher than the static long-term values noted, *supra*, because of negative pore pressure developing during shear.

Dr. Hendron then utilized a method accepted by the Army Corps of Engineers which is more conservative because it assumes that negative pore pressure will dissipate rapidly and cannot be counted on to increase the undrained shear strength. That this is a very severe assumption is reflected in the use of 1.0 as the minimum factor of safety for this case by the Army Corps of Engineers. This approach yielded factors of safety for the critical portions of the baffle and perimeter dikes of 1.34 and 1.50, respectively. Hendron, ff. Tr. 3940, at 33-35; Tr. 3946-51 (Hendron); Tr. 3517, 4114-17 (Singh).

Dr. Hendron concluded that the factors of safety obtained for this extreme condition are sufficient to assure the integrity of the return pipes during the improbable event of a rapid drawdown. Mr. Singh testified that he had reviewed the drawdown analyses performed by Dr. Hendron, and that the more conservative analysis was performed in accordance with the Army Corps of Engineers manual and procedures. Messrs. Singh and Kane concurred with Dr. Hendron's conclusion that a factor of safety of 1.34 would be adequate to assure the stability of the critical portions of the baffle and perimeter dikes during a rapid drawdown of the cooling pond from the level of 627 feet. Hendron, ff. Tr. 3940, at 34-35; Tr. 3952-53 (Hendron); Tr. 3517, 3656-58, 4114-17 (Singh); Tr. 3649 (Kane).

316. The analyses performed by Dr. Hendron and the Army Corps of Engineers also assessed the stability of the baffle and perimeter dikes under the flooding conditions specified in the FSAR, i.e., with the Tittabawasee River raised to the level of 620 feet. However, these analyses did not address the flooding levels associated with the Probable Maximum Flood ("PMF") with the river level at 631 feet. This is an extreme condition dependent on a coincidence of events in upstream retention areas.

Although PMF questions are not related directly to the shear strength and properties of dike materials, and hence are peripheral to the OM contention under consideration, they have been extensively addressed on our record. In August 1981, Dr. Hendron testified that he felt no concern about dike stability during a PMF but that there might be concern about erosion and the need for rip-rap. Based on preliminary hydrological information, the Staff consultant, Mr. Singh, expressed concern that a PMF might breach the perimeter dike and thereby induce damage because of erosion. Staff witness Joseph Kane also noted that the outstanding design questions concerned the dike's capability to prevent and withstand wave runup. Messrs. Singh, Hendron and Kane further indicated that in their opinion the PMF should not cause dike stability problems in the vicinity of the ECWR and that erosion to the outside slope of the perimeter dike should not affect the operation of the ECWR and the return pipes. They indicated, however, that the acceptability of the dikes in respect to a PMF was still under study.

In November 1982, Staff witness Raymond Gonzales testified that, based on studies submitted by the Applicant, NRC was satisfied that any PMF overtopping would be minor and would not impact on the cooling pond dikes. To preclude possible dike damage by erosion, NRC would require a suitable dike inspection and maintenance program. Tr. 3962-63, 3966-69 (Hendron); Tr. 3575, 3639-40, 4117-21 (Singh); Tr. 3641-44, 3650-52, 4123-36 (Kane); Tr. 10,113-15, 10,121-28 (Gonzales).

317. Dr. Hendron also evaluated the stability of the critical portions of the baffle and perimeter dikes under seismic loadings. He did not evaluate the capability of the Category I water return pipes to withstand seismic action. However, CPC performed a dynamic seismic analysis which confirmed the capability of these pipes to withstand current seismic criteria.[108] This affidavit indicated that, although initially based on the FSAR SSE (0.12*g*), the actual seismic input used was conservatively chosen so as to encompass the requirements of the SSRS.

Dr. Hendron did assess the dynamic resistance of the dike slopes in terms of critical yield acceleration using an approach that has been accepted by the NRC Staff for demonstration of stability under dynamic loads. Using very conservative assumptions Dr. Hendron determined that the yield accelerations for the critical portions of the baffle and perimeter dikes are 0.54*g* and 0.61*g*, respectively, i.e., three times larger than the values required for a critical yield acceleration of 0.19*g*. Dr. Hendron also testified that liquifaction of the foundation materials under the baffle and perimeter dikes is not a problem. Thus the critical slopes of these dikes would not experience significant inelastic movement under the seismic loadings associated with the SSE. Hendron, ff. Tr. 3940, at 16-17, 22-23, 35-36 and Appendix A; Tr. 3955-61, 3984-92 (Hendron); Tr. 3658-59 (Kane).

318. The Licensing Board finds that the soils materials placed in the baffle and perimeter dikes exceed design parameters and have sufficient shear strength to withstand the loadings likely to be imposed on the dikes should the Midland facility be operated. The Board further finds that the slopes of the portions of the baffle and perimeter dikes adjacent to the ECWR would be stable under all anticipated static loadings, conditions of rapid drawdown of cooling pond water, and the seismic loadings associated with earthquakes far greater than the FSAR SSE or the SSRS earthquake. Accordingly, the Board concludes, contrary to Stamiris Contention 4.B, that there is reasonable assurance that the critical slopes of the baffle and perimeter dikes are stable and will not adversely affect the safe operation of the ECWR or impinge upon the integrity of the two Category I water return pipes. This conclusion assumes the applicability of a suitable dike inspection and maintenance program, as proposed by the Staff (Finding 316, *supra*).

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[108] *See* Affidavit of Dr. Thiruvengadam, Enclosure E to Letter of Februa 3, 1983 from P.P. Steptoe (Applicant's counsel) to this Board.

Conclusions of Law

Based upon the foregoing Findings of Fact and upon consideration of the entire evidentiary record in these proceedings, including earlier rulings (such as LBP-82-35), the Board makes the following conclusions of law:

1. Although we have found many of the existing or proposed structures and soils remedial actions to be satisfactory (subject in some cases to certain technical specifications or conditions), any reasonable assurance conclusions bearing on the OL proceeding would also be subject to satisfactory execution of the remedial measures and satisfactory construction of the various facilities. Since each of these subjects must be subject to a further decision (and, in some cases, to further evidentiary hearings), and taking into account the present suspension of construction and questions concerning whether the project will ever be completed, we are declining at this time to express any conclusions of law, with respect to the OL proceeding.

2. With respect to the OM issues, we reiterate our conclusion (set forth in LBP-82-35) that the soils-related quality assurance deficiencies set forth in Part II and in Appendix A of the Modification Order were an adequate basis for the issuance of that Order.

3. For the reasons set forth in ¶ 1 of these conclusions, we are declining at this time to render a decision as to the extent to which the Modification Order should be sustained; except that the Modification Order shall continue in effect to the extent directed by LBP-82-35, pending further Order of this Board.

ORDER

On the basis of the foregoing Findings of Fact, Conclusions of Law and Opinion, and the entire record, it is, this 23rd day of January 1985, ORDERED:

1. The issues and contentions dealt with in this Decision are *resolved* to the extent set forth in this Decision and subject to the terms and conditions set forth herein.

2. CPC's motion for reconsideration of our Prehearing Conference Order dated May 5, 1981 (concerning use of backfitting procedures in the OL seismic review) is *denied*.

3. Requirements imposed by LBP-82-35 are *continued in effect*, pending further Order of this Board.

4. Jurisdiction is *retained*, pending issuance of a final Initial Decision in the OM proceeding, to entertain new information arising from the Dow-CPC litigation and significantly affecting issues covered by this Partial Initial Decision.

5. CPC's September 10, 1984 proposal, to the extent that it asserts that no further hearings be held at this time and that CPC file an additional report on the status of the project in 6 months, is *granted;* with the understanding that we be *informed* promptly of any significant developments (including but not limited to plans or proposals for the restart of construction). The foregoing project status report should be *filed* on or before April 1, 1985. Parties may respond within 10 days of service (15 days for the Staff). The Board's ruling on CPC's proposal that its current obligation to forward audit and nonconformance reports to the Board and parties be discontinued is *deferred*, pending our receipt and evaluation of a further report in early 1985 on this question. (In the interim, CPC need furnish the Board only one copy of such audit and nonconformance reports.)

6. In accordance with 10 C.F.R. §§ 2.760, 2.762, 2.764, 2.785, and 2.786, this Partial Initial Decision shall become effective immediately and will constitute, with respect to the matters resolved herein (and subject to the limitations set forth herein), the final decision of the Commission thirty (30) days after issuance hereof, subject to any review pursuant to the above-cited Rules of Practice. Any party may take an appeal from this decision by filing a Notice of Appeal within ten (10) days after service of this Partial Initial Decision. Each appellant must file a brief supporting its position on appeal within thirty (30) days after filing its Notice of Appeal (forty (40) days if the Staff is the appellant). Within thirty (30) days after the period has expired for the filing and service of the briefs of all appellants (forty (40) days in the case of the Staff), a party who is not an appellant may file a brief in support of, or in opposition to, any such appeal(s). A responding party shall file a single, respon-

sive brief *only*, regardless of the number of appellants' briefs filed. (*See* 10 C.F.R. § 2.762 (1984).)

THE ATOMIC SAFETY AND
LICENSING BOARD

Frederick P. Cowan
ADMINISTRATIVE JUDGE

Jerry Harbour
ADMINISTRATIVE JUDGE

Charles Bechhoefer, Chairman
ADMINISTRATIVE JUDGE

[Appendices B and C have been omitted from this publication, but may be found in the NRC Public Document Room, 1717 H Street, NW, Washington, DC 20555.]

APPENDIX A

Soils-Related Contentions

Following is the text of the Intervenors' contentions which have been at issue in the soils-related hearings. These contentions include both those raising technical design issues (some of which are resolved in this Decision) and those involving QA/managerial attitude issues (not resolved by this Decision).

I. *OM Contentions of Barbara Stamiris* (from Appendix to Prehearing Conference Order dated October 24, 1980, as modified by Stamiris Answers to Applicant's interrogatories, dated April 20, 1981; Contentions 6 and 7 from LBP-84-20, 19 NRC 1285, 1287 (1984)):

1. Consumers Power Company statements and responses to NRC regarding soil settlement issues reflect a less than complete and candid dedication to providing information relevant to health and safety standards with respect to resolving the soil settlement problems, as seen in:

a) the material false statement in the FSAR (Order of Modification, Appendix B);

b) the failure to provide information resolving geologic classification of the site which is pertinent to the seismic design input on soil settlement issues (Responses to FSAR Questions 361.4, 361.5, 361.7 and 362.9);

d) the failure to provide adequate acceptance criteria for remedial actions in response to 10 C.F.R. § 50.54(f) requests (as set forth in part II of the Order of Modification);

and this managerial attitude necessitates stricter than usual regulatory supervision (ALAB-106) to assure appropriate implementation of the remedial steps required by the Order Modifying Construction Permits, dated December 6, 1979.

2. Consumers Power Company's financial and time schedule pressures have directly and adversely affected resolution of soil settlement issues, which constitutes a compromise of applicable health and safety regulations as demonstrated by:

   a) the admission (in response to § 50.54(f) question #1 requesting identification of deficiencies which contributed to soil settlement problems) that the FSAR was submitted early due to forecasted OL intervention, before some of the material required to be included was available;

   b) the choice of remedial actions being based in part on expediency, as noted in Consumers Power Company consultant R.B. Peck's statement of 8-10-79;

   c) the practice of substituting materials for those originally specified for "commercial reasons" (NCR QF203) or expediency, as in the use of concrete in electrical duct banks (p. 23 Keppler Report) [March 22, 1979 Keppler Investigation Report conducted by Region III, Dec. 78-Jan. 79];

   d) continued work on the diesel generator building while unresolved safety issues existed, which precluded thorough consideration of Option 2 Removal and Replacement Plan; and

   e) [withdrawn by letter dated June 1, 1981]

3. Consumers Power Company has not implemented its Quality Assurance Program regarding soil settlement issues according to 10 C.F.R. Part 50, Appendix B regulations, and this represents a repeated pattern of quality assurance deficiency reflecting a managerial attitude inconsistent with implementation of Quality Assurance Regulations with respect to soil settlement problems, since reasonable assurance was given in past cases (ALAB-147, ALAB-106 and LBP-74-71) that proper quality assurance would ensue and it has not.

The Quality Assurance deficiencies regarding soil settlement include:

a) 10 C.F.R. Part 50, Appendix B, Criteria III, V, X and XVI as set forth in the Order of Modification;

b) 10 C.F.R. Part 50, Appendix B, additional criteria denoted by roman numerals below:

I. The Applicant has failed to assume responsibility for execution of the QA program through its failure to verify and review FSAR statements (pp. 6-8 and p. 21, Keppler Report) and through its reliance on final test results not in accordance with specified requirements (p. 16, Keppler Report);

II. The QA program was not carried out according to written policies, procedures and instructions, in that oral directions were relied upon and repeated deviations from policies occurred regarding compaction procedures (p. 9-14, Keppler Report);

VII. Control of purchased material has not been maintained, in that examination and testing of backfill materials did not occur in accordance with regulations (NCR QF29, NCR QF147);

IX. Control of non-destructive testing was not accomplished by qualified personnel using qualified procedures regarding

a) moisture control (Keppler Report p. 14-16; QA Request SD40, NCR QFS52, 172, 174 and 199);

b) compaction procedures (Keppler Report, p. 9; NCR QFS 68, 120 and 130); and

c) plant fill work (pp. 24 and 25, Keppler Report);

XI. Test programs did not incorporate requirements and acceptance limits adequately in the areas referenced in a, b and c above, and do not meet these requirements regarding soil settlement remedial actions;

XIII. Measures were not adequately established to prevent damage or deterioration of material regarding frost effects on compacted fill (pp. 16 and 17, Keppler Report);

XV. Measures were not taken to control non-conforming material in order to prevent the inadvertent use (NCR QF29 and QF127);

c) the settlement of the Administration Building in 1977 should have served as a quality indicator, preventing the same inadequate procedures from occurring in the 1978 construction of the diesel generator building causing its eventual settlement.

4. Consumers Power Company performed . .d proposed remedial actions regarding soils settlement that are inadequate as presented because:

   A. Preloading of the diesel generator building

      1) does not change the composition of the improper soils to meet the original PSAR specifications;

      2) does not preclude an unacceptable degree of further differential settlement of diesel generator building;

      3) does not allow proper evaluation of compaction procedures because of unknown locations of cohesionless soil pockets;

      4) may adversely affect underlying piping, conduits or nearby structures; and

      5) yields effects not scientifically isolated from the effects of a rise in cooling water and therefore not measured properly;

   B. Slope stability of cooling pond dikes is not assured because they were built with the same improper soils and procedures [as the soils foundation for the DGB] (NCR QF172);

   C. Remedial soil settlement actions are not based on adequate evaluation of dynamic responses regarding dewatering effects, differential soil settlement, and seismic effects for these structures:

      a. Auxiliary Building Electrical Penetration Areas and Feedwater Isolation Valve Pits

      b. Service Water Intake Building [sic] and its Retaining Walls

      c. Borated Water Storage Tanks

      d. Diesel Fuel Oil Storage Tanks

      e. Diesel Generator Building

      f. Related Underground Piping and Conduit.

   D. Permanent dewatering

      1) would change the water table, soil and seismic characteristics of the dewatered site from their originally approved PSAR characteristics — characteristics on which the safety and integrity of the plant were based, thereby necessitating a reevaluation of these characteristics for affected Category I structures;

      2) may cause an unacceptable degree of further settlement in safety-related structures due to the anticipated drawdown effect;

      3) to the extent subject to failure or degradation, would allow inadequate time in which to initiate shutdown, thereby necessitating reassessment of these times.

Therefore, unless all the issues set forth in this contention are adequately resolved, the licensee actions in question should not be considered an acceptable remediation of soil settlement problems.

5. [withdrawn by letter dated June 1, 1981]

6. Consumers misrepresented its time schedule for completion of the Midland plants to the NRC, including the NRC Staff and this Licensing Board. *See* paragraphs 20, 37, 39-48 [of initial Dow complaint against CPC, dated July 14, 1983].

7. Consumers used and relied on U.S. Testing test results to fulfill NRC regulatory requirements while knowing that these test results were invalid. *See* par. 24, 35 [of initial Dow complaint].

II. *OL Contention 24 of Mary Sinclair* (from statement of contentions dated October 31, 1978, as modified in accordance with Special Prehearing Conference Order dated February 26, 1979, at 8):

> Serious questions have been raised concerning the ground stability of portions of the site [of the Midland facility]. At least one of the essential buildings of the reactor complex [the DGB] is reported sinking, and construction has been halted on that building. As a result of the serious and unresolved questions concerning ground stability, the findings required by 10 C.F.R. §§ 50.57(a)(3)m and 50.57(a)(6) cannot be made.

III. *OL Contention 2 of Wendell H. Marshall* (from supplemental statement of contentions dated October 31, 1978, as clarified by Special Prehearing Conference Order dated February 26, 1979, at 21):

> Present geological conditions, according to newspaper accounts, is causing the settling of the [diesel] generator building at the Nuclear Power Plant site.

IV. *OM Contentions of Sharon K. Warren* (from Appendix to Prehearing Conference Order dated October 24, 1980):

1. The composition of the fill soil used to prepare the site of the Midland Plant — Units 1 and 2 is not of sufficient quality to assure that pre-loading techniques have permanently corrected soil settlement problems. The NRC has indicated that *random* fill dirt was used for backfill. The components of random fill can include loose rock, broken concrete, sand, silt, ashes, etc. all of which cannot be compacted through pre-loading procedures.

2. A. Because of the known seepage of water from the cooling pond into the fill soils in the power block area, permanent dewatering procedures being proposed by Consumers Power Company are inadequate, particularly in the event of increased water seepage, flooding, failure of

pumping systems and power outages. Under these conditions, Consumers cannot provide reasonable assurance that stated maximum levels can be maintained.

B. Given the facts alleged in Contention 2.A, and considering also that the Saginaw Valley is built upon centuries of silt deposits, these highly permeable soils which underlie, in part, the diesel generator building and other class I structures may be adversely affected by increased water levels producing liquefaction of these soils. The following will also be affected:

1) borated water tanks

2) diesel fuel oil tanks.

3. Pre-loading procedures undertaken by Consumers Power have induced stresses on the diesel generating building structure and have reduced the ability of this structure to perform its essential functions under that stress. Those remedial actions that have been taken have produced uneven settlement and caused inordinate stress on the structure and circulating water lines, fuel oil lines, and electrical conduit.

UNITED STATES OF AMERICA
NUCLEAR REGULATORY COMMISSION

ATOMIC SAFETY AND LICENSING BOARD

Before Administrative Judges:

John H Frye, III, Chairman
Dr. James H. Carpenter
Dr. Peter A. Morris

In the Matter of **Docket No. 40-2061-ML**
(ASLBP No. 83-495-01-ML)

KERR-McGEE CHEMICAL CORPORATION
(West Chicago Rare Earths Facility) **January 23, 1985**

Licensing Board rules on petitions for reconsideration and clarification of its Memorandum and Order ruling on the admissibility of contentions (LBP-84-42, 20 NRC 1296). In response to Staff's motion, Licensing Board rules that Kerr-McGee's contention (which seeks a determination that its plan for permanently disposing of mill tailings at its West Chicago is acceptable) is an acceptable contention, that Staff's obligation to supplement the record on NEPA issues springs from the People's contention rather than Kerr-McGee's, that Staff must circulate a supplemental impact statement to accomplish this supplementation, and that the Board will not refer its ruling admitting Kerr-McGee's contention to an appeal board for interlocutory review. The Board denies the People's motion for reconsideration of its ruling removing references to Part 61 from one of their subcontentions on the ground that Part 61 is inapplicable and grants their motion for r--onsideration of the denial of another subcontention which seeks to req -- Staff to respond to certain comments on the DES.

ATOMIC ENERGY ACT: DENIAL OF APPLICATION
RULES OF PRACTICE: DENIAL OF APPLICATION
ADMINISTRATIVE PROCEDURE ACT: DENIAL OF APPLICATION

Under the Administrative Procedure Act, the Atomic Energy Act, and the Commission's Rules of Practice, an application cannot be denied without stating reasons for the denial. These reasons must indicate why the application does not comply with the statute and regulations under which it is filed. *SEC v. Chenery Corp.*, 318 U.S. 80, 94; 87 L. Ed. 626, 636 (1943); *Commonwealth Edison Co.* (Byron Nuclear Power Station, Units 1 and 2), ALAB-770, 19 NRC 1163 (1984); 5 U.S.C. § 555(e); 10 C.F.R. § 2.103(b).

NEPA: RECIRCULATION OF AN ENVIRONMENTAL IMPACT STATEMENT

Where an FES disregards broad areas of environmental impact or fails to apprise the public of the nature of the proposed action and its expected consequences, recirculation of the statement is necessary.

RULES OF PRACTICE: REFERRAL OF RULING

Admission of a contention which will require further Staff review does not result in unusual delay which justifies referral for interlocutory review. *Duke Power Co.* (Catawba Nuclear Station, Units 1 and 2), ALAB-687, 16 NRC 460, 464 (1982), *rev'd on other grounds*, CLI-83-19, 17 NRC 1041 (1983).

MEMORANDUM AND ORDER
(Ruling on Motions for Reconsideration)

In LBP-84-42, 20 NRC 1296 (1984) we ruled on the admissibility of contentions and defined the scope of this proceeding. The NRC Staff and the People of the State of Illinois[1] seek relief with respect to those rulings. Staff moved for reconsideration, clarification or referral to the

[1] The People and the Illinois Department of Nuclear Safety have intervened in this proceeding. Hereinafter they are collectively referred to as "the People."

Appeal Board on November 16, and the People moved for reconsideration on November 2. Kerr-McGee opposes both motions but suggests that some clarification may be appropriate. Staff opposes a portion of the People's motion and supports a portion. The People oppose the Staff's motion. We deal with Staff's motion first.

STAFF'S MOTION

Before discussing Staff's motion in detail, some background is necessary. Kerr-McGee has filed an application for a license amendment which would permit it to permanently dispose of thorium mill tailings generated at its West Chicago Rare Earths Facility in a disposal cell located on site. Staff reviewed this application and published a Final Environmental Statement (FES) in which it concluded that, rather than permanent disposal, onsite storage in a similar disposal cell should be approved and a decision on permanent disposal deferred until several years of monitoring data had been accumulated. Petitions to intervene were invited. The People responded to this invitation and filed a contention (AG-1) which attacked the FES on the ground, *inter alia*, that it constituted an illegal segmentation of an overall plan to permanently dispose of these tailings on site. Although it did not request a hearing, Kerr-McGee filed a contention (KM-1) in which it asserts that its plan for onsite disposal should be approved now.

It should be noted that the FES is an unusual document. The various alternatives were not evaluated to the point of a final Staff conclusion for the reason stated on page 1-6 that "the Staff has no basis on which to evaluate the applicant's proposal for use of the site as a disposal site." Staff was in this unusual position because of two factors. The U.S. Environmental Protection Agency (EPA) standards governing disposal required by the Uranium Mill Tailings and Radiation Control Act (UMTRCA) had not been published in final form and the NRC criteria for mill tailings disposal (10 C.F.R. Part 40, Appendix A) had been in part temporarily suspended by congressional action. Because Staff had no basis for evaluating other alternatives, in its view the only available option at that time was Alternative III, onsite storage in a safe manner.

The situation dramatically changed after Staff issued its FES. EPA's standards were promulgated[2] and the temporary suspension of Appendix A expired. The Commission initiated rulemaking to bring Appendix A

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[2] 40 C.F.R. Part 192, promulgated on October 7, 1983, 48 Fed. Reg. 45,926.

into conformity with the EPA standards.[3] Staff now has a basis for evaluating alternatives.

As noted above, the People's Contention AG-1 challenged the FES on the ground that it constituted an illegal segmentation of an overall plan for permanent onsite disposal. All parties agreed that this and other issues raised by Contention AG-1 should be decided on briefs. Additionally, the admissibility of Contention KM-1 was extensively briefed.

In LBP-84-42 we ruled on the legal issues presented in these briefs. We agreed with the People that the FES illegally segments the overall plan to dispose of these tailings and admitted Kerr-McGee's contention over Staff's objection. Staff seeks reconsideration of our ruling admitting Contention KM-1, and clarification with respect to the obligation to supplement its environmental and safety reviews in connection with both Contentions KM-1 and AG-1. In the event we order Staff to supplement the record on the safety and environmental aspects of Kerr-McGee's proposal for permanent disposal, Staff requests that we refer our ruling on Contention KM-1 to the Appeal Board for prompt decision.

Admissibility of Contention KM-1

In LBP-84-42, we agreed with Staff that Kerr-McGee had indeed waived its right to cause a hearing to be held on its application. However, we disagreed with Staff that that waiver precluded Kerr-McGee from filing a contention urging that its application be approved and that such a contention was outside the scope of this proceeding as defined by the Commission. In its motion, Staff takes the position that we erred in these two respects.

Staff does not contest our ruling insofar as it stands for the proposition that an applicant, as a party to an adjudicatory hearing held at the request of another, may file contentions that are within the scope of the proceeding. As we understand Staff's position, it is founded on Staff's view that it had denied Kerr-McGee's application. Thus, Staff argues that Kerr-McGee, having waived its right to challenge Staff's denial of its application by requesting a hearing, may not now pursue such a contention unless it is within the scope of the hearing as defined by the People's contentions. In Staff's view, Contention KM-1 is not within that scope. Staff's position is best summed up in the following paragraphs from pages 4-5 of its motion.

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[3] See 49 Fed. Reg. 46,418, 46,425 (Nov. 26, 1984).

It is true that an applicant will be admitted as a party without first requesting a hearing, instituted at the request of another party, that may affect its interests. The Staff does not dispute that an applicant may raise issues, in the form of contentions or otherwise, that are within the scope of a proceeding requested by another party. It does not follow, however, that an applicant may also avoid the consequences of waiving its hearing rights by submitting contentions that expand the scope of the proceeding to include the very issue on which a hearing was waived.

By admitting Kerr-McGee's Contention 1, the Board has, in effect, overruled the Staff's denial of the Kerr-McGee proposal without that denial having been properly placed in issue before the Board. Indeed, in holding that Kerr-McGee waived its right to initiate a hearing on the denial of its proposal, the Board should have recognized that Kerr-McGee had forfeited its right to file contentions that would initiate a review of that denial.

There are sound public policy reasons for reaching this result. Kerr-McGee participated in numerous iterations with the Staff over its proposal. It knew in detail the Staff's position, and, but for the Illinois intervention, was willing to accept it. That Kerr-McGee did [illegible] request a hearing is conclusive evidence of that fact. Applicants should be [illegible] accountable for the consequences of their considered acts. Kerr-McGee did not ask for a hearing. Consequently, in view of Board's correct ruling that waiver does obtain in NRC adjudicatory proceedings, Kerr-McGee should be held to have accepted the Staff's conclusion and should be bound by it.

Kerr-McGee opposes Staff's request for reconsideration of this ruling. Kerr-McGee points out that, because the Environmental Protection Agency's standards applicable to its proposal were not published until after its opportunity to request a hearing had expired, it could not have filed such a request along with Contention KM-1 within that period. But, even if Contention KM-1 could have been filed along with a timely request for hearing, Kerr-McGee goes on to point out that, in the course of briefing this issue originally, Staff took the position that Contention KM-1 could have been litigated within the scope of the matters raised by the People, and attacks Staff's reading of the Commission's delegation as too narrow.[4]

The People also oppose this aspect of Staff's motions. They take issue with Staff's assertion that it denied Kerr-McGee's application. Their position is best summed up in the following excerpts from pages 6-8 of their December 13 response.

In short, the subject matter of this proceeding is Kerr-McGee's application and the petitions concerning it, together with Staff recommendations and the FES. Under Sec. 189 and the NRC's rules this is true regardless whether Kerr-McGee requested a hearing or filed contentions and regardless what the Staff's FES may recommend.

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[4] Kerr-McGee's December 13 Response at 2-5.

> Finally, it should be pointed out that even if Kerr-McGee had itself applied for an amendment authorizing onsite storage, the scope of the proceeding would have to include the impacts of, and legal requirements associated with, permanent onsite disposal. This is because the FES acknowledges that once the wastes are buried on site they will likely remain there forever (FES at 1-8). In light of this acknowledgment, the hearing requirement of Sec. 189 would be violated if the hearing were limited to short-term issues.
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> * * *
>
> For all these reasons, whatever theoretical questions the Staff's motion may raise, in *practical* terms they have no bearing on this proceeding. No matter how you cut it, whether under the AEA or NEPA, the proceeding must address permanent disposal of the wastes.

Staff's position is premised in part on the proposition that it denied Kerr-McGee's application and that, in order to preserve its right to challenge that denial, Kerr-McGee was obligated to request a hearing. We did not in LBP-84-42 view Staff's action on Kerr-McGee's application as a denial. If that action does constitute a denial, Staff's position that Kerr-McGee waived its right to challenge it has considerable force. For this reason, we asked Staff to provide specific citations to its denial, whether its denial required a determination that the West Chicago site is not suitable for permanent disposal of the tailings, and whether Staff had made such a determination.

Staff responded to these questions on January 3. Staff takes the position that it denied the application at pages 1-3 to 1-9 of the FES and that that denial was memorialized in the Commission's June 7, 1983, notice of opportunity for hearing.[5] Staff notes that it stated in the FES that it had selected Alternative III as the preferred alternative and that that alternative was the only currently acceptable alternative.

Staff goes on to argue that its denial of Kerr-McGee's application does not require a Staff determination that the West Chicago site is unsuitable on the ground that NEPA does not require such a finding, but rather a finding whether there are environmentally preferable alternatives to that put forth in the application. In that event, Staff argues that it must take whatever steps it can to see to it that that alternative is implemented. Staff states that it has made no determination as to the suitability of this site for permanent disposal.

We must reject this position. It is clear to us that Staff did not deny Kerr-McGee's application. Rather, Staff sought to defer it. At page 1-2 of the FES the Staff states that "[u]nder Alternative III, the decision on ultimate disposal of the radioactive wastes would be deferred." In Part

[5] 48 Fed. Reg. 26,381.

VI of the same chapter of the FES (p. 1-8) Staff outlines the conditions under which licensed storage on site would be terminated. This statement clearly implies to us that the Kerr-McGee's application has not been denied, but rather has been deferred because of the lack of regulatory standards under which it could be judged (*see* FES at p. 1-6).

Whatever the merits of Staff's argument that NEPA does not require it to reject the West Chicago site in order to deny Kerr-McGee's proposal may be, it misses the mark. Under the Atomic Energy Act and the most basic principals of jurisprudence, Staff may not deny this application without stating reasons. Yet Staff has made no determination with regard to the suitability of this site and is prepared to approve storage in a disposal cell proposed by Kerr-McGee for permanent disposal in full recognition of the fact that once storage is implemented, removal to another site is unlikely. This does not smack of application denial, and we are unable to find any language in the FES which does.

The simple fact is that, in order to deny the application, Staff must state some reasons. Not to do so is the epitome of arbitrary and capricious action. Yet, because of the lack of regulatory standards, no reasons could be given. For this reason, Staff had no choice but to defer, as it has done in the FES, its determination with respect to this application. *SEC v. Chenery Corp.*, 318 U.S. 80, 94; 87 L. Ed. 626, 636 (1943); *Commonwealth Edison Co.* (Byron Nuclear Power Station, Units 1 and 2), ALAB-770, 19 NRC 1163, 1168-69 (1984); 5 U.S.C. § 555(e).

Indeed, this result is required by the Commission's regulations. Section 2.103(b) of 10 C.F.R. provides that, before the Director of Nuclear Material Safety and Safeguards may deny an application, he must find that the application does not comply with the requirements of the Atomic Energy Act and the Commission's regulations. Here, because no such finding was or could have been made, no denial could issue.[6]

Before leaving this topic, we are compelled to add the observation that, had Staff reached a determination that the West Chicago site was unsuitable for permanent disposal, Kerr-McGee might well have requested a hearing. Instead, in these circumstances where it appeared from the FES that there was at least a substantial probability that that site would ultimately be approved, there is little apparent reason why Kerr-McGee should follow that course. Those circumstances dramatically changed when the People, perceiving that Staff would ultimately an-

[6] Moreover, we are compelled to note that a denial of Kerr-McGee's application might have had some unintended results. Kerr-McGee's license, STA-583, has an expiration date of August 31, 1979. This license remains in effect by virtue of Kerr-McGee's July 25, 1979, application for renewal. If this is the application Staff claims to have denied, then that denial would have the result of terminating Kerr-McGee's license. We doubt that Staff would wish such a result.

prove onsite disposal, requested a hearing in order to seek a determination that this site was not suitable as a repository for the tailings. At that point, the issue of site suitability was clearly presented for the first time and Kerr-McGee moved to protect its interest in obtaining approval. These are not circumstances which give rise to a waiver of Kerr-McGee's right to contest a Staff denial of its application.

Moreover, the People's contentions brought Kerr-McGee's contention clearly within the scope of the proceeding. Even were we to accept Staff's position on application denial, which we do not, the existence of Contentions AG-1 and AG-2 would require that Contention KM-1 be accepted.[7]

For the foregoing reasons, we deny Staff's motion for reconsideration, and move to its motion for clarification.

Clarification of LBP-84-42

In its request for clarification of our rulings in LBP-84-42, Staff notes that the admission of Contention KM-1 requires that permanent disposal of these tailings be considered now. Staff then states that we did not find the FES inadequate to support Alternative III, although it notes that our "decision could be read as finding the FES inadequate to support Alternative III with regard to serial segmentation and the need for a cost-benefit analysis."[8] Staff goes on to assert its right to continue to support Alternative III and defend its "denial" of Kerr-McGee's application, and states:

> Accordingly, the Staff does not bear the primary burden of demonstrating in this proceeding, the suitability of the West Chicago site for permanent disposal. Similarly, the Staff has no burden of demonstrating the superiority of any alternative disposal site. The primary evidentiary burden on the acceptability of an alternative for licensing is on the advocate of the alternative.[9]

Staff then concludes this portion of its request for clarification with the assertion that, if Kerr-McGee wishes to trigger a new Staff review of its proposal, it should file a new application, which would permit Staff to recover the cost of that review in license fees.

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[7] Because of this result, we need not address Staff's argument that an applicant which has waived its right to request a hearing may not file contentions which are outside the scope of those filed by an intervening party.

[8] Staff's Motion at 8 n.2.

[9] *Id.* at 8-9.

In the second part of its request for clarification, Staff repeats its position with respect to its obligations set forth in the first part. It then assumes for the sake of argument that some deficiencies in the FES are not attributable to the admission of Contention KM-1, but rather to Contention AG-1. Under this assumption, Staff requests clarification in three areas.

First, Staff states that under LBP-84-42, it is for it to assess the extent to which long-term environmental impacts require further treatment and whether this may be accomplished in testimony.

Second, Staff states that it need not undertake any additional review of alternative sites unless it is determined after hearing that Kerr-McGee's alternative site investigation generated insufficient information to permit the required "hard look." Staff notes that the People are free to develop in this proceeding any alternative sites they wish considered.

Third, Staff may assess for itself what needs to be considered in the cost-benefit balance and whether that may be accomplished in testimony.

In its response to the Staff's motion for clarification, Kerr-McGee notes that Staff believes that its additional obligations stem from the admission of Contention KM-1. Kerr-McGee points out that Staff's obligation, if any, to supplement the record on NEPA issues stems from the admission of Contention AG-1. Kerr-McGee, while noting its inability to determine what portions of LBP-84-42 Staff considers ambiguous, states its belief that it is appropriate for us to provide guidance to the extent we find Staff's interpretation of its obligations to be incorrect.

The People take strong issue with the Staff's position that any deficiencies in the FES may be cured through testimony rather than by requiring the issuance of a supplemental environmental statement for comment prior to hearing. The People maintain that the defects in the FES are not of a minor nature and therefore may not be corrected at hearing, relying on *Public Service Co. of Oklahoma* (Black Fox Station, Units 1 and 2), ALAB-573, 10 NRC 775, 785-87 (1979); *Florida Power & Light Co.* (Turkey Point Nuclear Generating Station, Units 3 and 4), ALAB-660, 14 NRC 987, 1014 (1981); and *Boston Edison Co.* (Pilgrim Nuclear Generating Station, Unit 2), ALAB-479, 7 NRC 774 (1978).

As to what the supplemental EIS should cover, the People stand by their letter to Staff counsel of October 30, 1984.[10] In that letter they take the following positions with respect to Contention AG-1:

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[10] That letter is attached to Staff's motion.

1. AG-1(b) — long-term impacts of onsite disposal — requires the Staff to consider:
   a) specific measures for excluding humans over the long-term;
   b) long-term maintenance and monitoring, its cost, and its funding;
   c) long-term reliability of the disposal cell;
   d) long-term radiological impacts on children; and
   e) the likelihood and effects of settlement on the disposal cell.
2. AG-1(d) — cost-benefit balance — requires Staff to compare the above factors along with economic and socioeconomic factors for West Chicago and alternatives. With respect to alternate sites (AG-1(c)), the People expect that Staff will not limit its consideration to those sites identified by Kerr-McGee, but will consider other sites as well and will require sufficiently detailed information on these sites to make the analysis credible. The People read the FES as a decision to defer detailed analysis of alternatives, and suggest that, if this is so, the alternative site question must be freshly addressed. To this end, the People offer the assistance of various state agencies.
3. AG-1(g) — consistency of Kerr-McGee's proposal with applicable Federal and State policies — the Staff should address the Illinois ground water protection standards, Illinois' policy on the siting of long-term radioactive waste disposal facilities, and the re uirements of the Uranium Mill Tailings Radiation Control Act and Append A to 10 C.F.R. Part 40.

In their response to Staff's motion, the People reiterate their position that under NEPA, the Staff must perform the alternate site analysis and may not rely on information generated by Kerr-McGee for that purpose. The People rely on *Pilgrim,* ALAB-479, *supra,* 7 NRC at 794, for this point.

The first point to be addressed is Staff's assumption that the deficiencies in the FES (segmentation and the need for a cost-benefit analysis) which were identified in LBP-84-42 are solely related to Contention KM-1. That is not the case. First, Contention KM-1 asserts that Kerr-McGee's proposal meets the standards of UMTRCA and EPA's regulations promulgated under it. It does not assert that the Staff's NEPA analysis has been inadequate and, in fact, Kerr-McGee defended that analysis in briefing Contention AG-1.

Second, Contention AG-1 clearly raises the issue of segmentation and lack of a cost-benefit balance. It was in connection with Contention AG-1 that we held that NEPA requires Staff to consider permanent onsite disposal now. Thus Contention AG-1 serves to force a NEPA consideration of permanent onsite disposal, while Contention KM-1 forces an evaluation of Kerr-McGee's proposal under UMTRCA. Staff's assumption that our NEPA ruling was tied to Contention KM-1 is incorrect; that ruling would have been made in response to Contention AG-1 had Kerr-McGee refrained from advancing Contention KM-1.

Staff is thus similarly incorrect in its assumption that we did not find the FES inadequate to support Alternative III. We specifically found that Alternative III was but one step toward the goal of permanent disposal and that the FES strongly indicated that once Alternative III were implemented, Staff would approve permanent onsite disposal.[11] Consequently, we held that NEPA and certain of the CEQ regulations which have been adopted by the Commission required that permanent onsite disposal be considered in connection with Alternative III.

Thus Staff is incorrect in its assumption that we did not require it to support or oppose Kerr-McGee's proposal. We did require Staff to take a position on that proposal. By admitting Contention KM-1, we required Staff to take a position under UMTRCA and by our ruling on Contention AG-1, we required Staff to perform an environmental review of Kerr-McGee's proposal.

Nevertheless, we are compelled to observe that Staff is legally free to pursue Alternative III if it wishes, although we can perceive no practical reason for doing so.[12] We have held that Kerr-McGee's application is still pending before the Staff and that, even if that application were no longer pending, Staff's preferred alternative requires a NEPA consideration of permanent disposal. Consequently Staff will have to consider and conclude whether it will approve permanent onsite disposal at West Chicago. If Staff concludes that Kerr-McGee's proposal is acceptable, there appears to us to be no practical reason for it to continue to support Alternative III. Should Staff conclude that Kerr-McGee's proposal is unacceptable, perhaps Alternative III might assume slightly more practicality. However, even in that situation, we do not believe it to be a sensible alternative because:

> First, the cost of storage at West Chicago and subsequent removal to another site is regarded as prohibitive by the Staff;[13]
>
> Second, the FES reveals that there is currently no compelling reason why storage in an engineered cell is necessary for the short term. The FES indicates that radioactive materials are not leaking into the aquifer[14] and that airborne emissions are not excessive;[15] and

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[11] LBP-84-42, *supra*, 20 NRC at 1316.

[12] In answer to our query as to which reasons exist to continue to advocate Alternative III if permanent disposal must be considered now under NEPA, Staff responded on January 3 by indicating that, in light of all the circumstances, Staff considers Alternative III a prudent course.

[13] *See* LBP-84-42, *supra*, 20 NRC at 1309.

[14] FES, § 5.6.2.1, at p. 5-11.

[15] FES, Table 5.5, at p. 5-28.

Third, as we indicated in LBP-84-42 (20 NRC at 1304-05), we do not perceive any regulatory restraint under Alternative I which would be avoided by Alternative III.

Nonetheless, while we see nothing to be gained by continuing to pursue Alternative III, Staff is of course free to show us that our conclusion in this regard is in error.

We agree with Staff that we have no authority to direct it in the performance of its independent regulatory and review functions. But we do have authority to pass on the adequacy of Staff's review when it is properly challenged. That is the course which we have followed here. By admitting Contentions KM-1 and AG-1, we have concluded that Staff must determine the acceptability of Kerr-McGee's application under UMTRCA and review it under NEPA. While we agree with Staff that we cannot dictate the timing of its review or the conclusion it should reach, we must reaffirm our right to pass on the adequacy and legality of its actions when they are, as here, properly challenged under procedures established by the Commission. The fact that our rulings require additional Staff effort does not impinge on Staff's independence.

Staff's request for clarification with respect to long-term environmental impacts, alternate sites, and cost-benefit balancing under NEPA essentially asserts that Staff may determine for itself first, how much supplementation of the FES is necessary, and second, whether that supplementation may be done in testimony or whether a supplemental impact statement is necessary.[16] We agree with the first proposition but not the second. It is for Staff to determine in the first instance how much supplementation is necessary and to defend its position with evidence. We may not decide this matter in advance of receiving that evidence along with Kerr-McGee's and the People's evidence. However, we do note that Staff has made no determination under UMTRCA and has not reviewed permanent onsite disposal under NEPA.

The People's letter advises Staff of the People's position on the NEPA issues. Staff should take this position into account in making its determination and be aware that it will have to defend its decisions

[16] We wish to note that, while we cannot direct Staff in precisely how it goes about its business, it would appear useful, as a practical matter, for Staff to undertake its supplementation of this FES only after Staff has determined whether it will approve Kerr-McGee's proposal under UMTRCA. Should Staff make an adverse determination, its NEPA review, to the extent it is not duplicative of the UMTRCA review, would be meaningless. Should Staff elect to support Kerr-McGee under UMTRCA, and should that view not prevail at hearing, Staff's NEPA review would similarly be meaningless. We recognize that there is a great deal of overlap to Staff's NEPA and UMTRCA reviews, so that any additional work to accomplish the NEPA review may be small once the UMTRCA review is complete. Nonetheless, a NEPA review is meaningless if Kerr-McGee's proposal does not meet UMTRCA standards.

accordingly. Staff must remember that its NEPA evaluation is of permanent onsite disposal, and its assessment of the supplementation necessary must be made in that context.

The People have made it clear that they do not consider Kerr-McGee's alternative site inquiry adequate and have offered technical assistance to the Staff in making its evaluation. While without the benefit of an evidentiary presentation we cannot conclude, as the People do, that Kerr-McGee's inquiry is inadequate and that, under NEPA, Staff may not rely on data generated by Kerr-McGee,[17] we must note that the People have clearly raised these issues and have offered technical assistance. It thus may turn out to be inappropriate for Staff to limit its alternative site analysis solely to data generated by Kerr-McGee. *Ibid.; Pilgrim,* ALAB-479, *supra,* 7 NRC at 780-81.

Staff's decision whether to proceed by way of testimony or a supplemental impact statement is on a different footing. The People take the position that at least a supplemental impact statement must be prepared and circulated for comment prior to hearing. In LBP-84-42, we did not directly address this point. On further consideration, we agree with the People. The fact that we have held that NEPA demands that permanent onsite disposal be considered now dictates this result. This omission alone is of sufficient magnitude to require circulation of a supplemental impact statement prior to hearing. Appeal Boards have noted that if an FES disregards broad areas of environmental impact or fails to apprise the public of the nature of the proposed action and its expected consequences, recirculation of the FES may be necessary. *Turkey Point,* ALAB-660, *supra,* 14 NRC at 1014; *Black Fox,* ALAB-573, *supra,* 10 NRC at 786. Staff's failure to consider permanent onsite disposal constitutes disregard of a broad environmental impact which will require circulation of a supplemental statement or recirculation of an amended FES in order to advise the public of the nature of the proposal and its expected consequences.

Referral to an Appeal Board

Staff requests that, in the event we order it to supplement the record on the UMTRCA and NEPA considerations incident to Kerr-McGee's application, we refer our ruling on Contention KM-1 to the Appeal

[17] We reaffirm our conclusion in LBP-84-42 that Staff may, in the absence of some demonstrated reason not to, rely on information generated by Applicants. *Public Service Co. of New Hampshire* (Seabrook Station, Units 1 and 2), CLI-77-8, 5 NRC 503, 524-25 (1977); 10 C.F.R. §§ 51.45 (b) and (c) and 51.60.

Board. Staff asserts that if it must engage in such supplementation, the delay and expense entailed would be detrimental to the public interest. Staff argues that if the People's view of the NEPA questions prevails, the proceeding could be delayed for as long as 2 years and considerable expense incurred. Staff points out that this expense could not be recovered from Kerr-McGee[18] and would result in immediate and irreparable harm to Staff as a result. Staff argues that the public interest would be served by a prompt resolution of this proceeding which would settle the question of the disposition of Kerr-McGee's tailings. Staff concludes by arguing that our ruling will infringe on its exercise of its independent responsibilities.

Both Kerr-McGee and the People oppose a referral. Kerr-McGee correctly points out that, to the extent Staff must supplement the FES, referral of our ruling on Contention KM-1 would in no way alleviate Staff's problems. Further, Kerr-McGee anticipates that such supplementation need not be extensive and anticipates being called on to assist in the effort. Moreover, Kerr-McGee argues that Staff's obligations to supplement the record are in no way an intrusion on its independent responsibilities.

The People maintain that interlocutory review, as a practical matter, can have no effect on the scope of the proceeding, that Staff does not contest our finding on illegal segmentation and that it is that finding which results in the necessity for supplementation on NEPA issues, and that there has been no persuasive showing that unusual delay or expense will result from our rulings.

We deny the request for referral. We agree with Staff that the public interest demands a resolution of the problem presented by Kerr-McGee's application. We part company with Staff in its thinking that a referral of our ruling on Contention KM-1 will somehow further that public interest. To the contrary, we think such a referral would needlessly delay such a resolution by diverting the parties' efforts from the issue — permanent disposal of these tailings. Both Kerr-McGee and the People are anxious to resolve that issue. Any delay occasioned by the necessity for Staff to engage in further reviews, while unfortunate, is not unusual.[19] Rather, it is essential to the resolution of the issue and therefore in the public interest.

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[18] In response to our inquiry on this point, Staff indicated that Kerr-McGee paid the full fee applicable at the time its application was submitted and that Commission policy forbids the retroactive assessment of the currently applicable fees. The present fees are based on the cost of processing an application and were adopted after submission of Kerr-McGee's application.

[19] *Duke Power Co.* (Catawba Nuclear Station, Units 1 and 2), ALAB-687, 16 NRC 460, 464 (1982), *rev'd in part on other grounds*, CLI-83-19, 17 NRC 1041 (1983).

It is also unfortunate that Staff may not bill Kerr-McGee for the costs incurred in its reviews, but that fact cannot control the disposition of the issues here presented. Staff must nonetheless discharge its responsibilities under UMTRCA and NEPA.

Finally, we note that Staff's obligations under NEPA would in no way be affected by interlocutory review of the admission of Contention KM-1. To that extent, Staff's request for referral could not alleviate its problems. Nor do our rulings infringe on Staff's independent responsibilities. Both Kerr-McGee and the People have challenged Staff's actions in accord with the procedures established by the Commission. If we were to agree with Staff on this point, we would effectively preclude those challenges and deny to Kerr-McGee and the People that which the Commission has granted them. We have no authority to take that action.

THE PEOPLE'S MOTION

The People have moved for reconsideration or clarification of our rulings on Contentions AG-1(g) and (h). They raise two points in connection with Contention AG-1(g). On the first of these, Illinois ground water standards, the People state:

> This contention alleges that the FES fails to consider applicable federal, state, and local policies, including Illinois' groundwater protection standards. In its ruling on this contention, the Board states:
>
>> We admitted this contention in our February 24 Prehearing Conference Order (pp. 7-8) on condition that the People demonstrate that Kerr-McGee is subject to these requirements and on our finding that we are competent to enforce them. The applicability of these requirements is the subject of litigation in the courts of Illinois. Thus, the first condition has not yet been satisfied.
>
> Board decision, pp. 48-9, n.84. To the extent that the Board has held that applicability of State laws governing nonradiological hazards remains an open question, the People respectively disagree.

Unfortunately, the People have quoted our ruling on Contention AG-2(g). We admitted Contention AG-1(g) as filed after eliminating its references to Part 61. Consequently this portion of the motion is denied.

Kerr-McGee and the People are engaged in a continuing debate with regard to the applicability of the Illinois ground water standards.[20] Much

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[20] *See* Kerr-McGee's Response of December 13, 1984, at 10-14; People's Reply to Kerr-McGee's Response of January 4, 1985, at 1-3.

of this debate centers on the question of Federal preemption of the Illinois standards. Both Kerr-McGee and the People cite various Federal and Illinois court decisions in support of their preemption arguments.

Our ruling on Contention AG-2(g) quoted by the People[21] was based on the assumption that the People are seeking to enforce the Illinois ground water standards in *People of the State of Illinois v. Kerr-McGee Chemical Corp.*, No. 80 CH 298 (18th Judicial Circuit of Illinois), and that Kerr-McGee was resisting. Thus unless that litigation is resolved favorably to the People, the ground water standards will not be applicable to Kerr-McGee. We will discuss this matter with the parties in more detail at the next prehearing conference. In the interim, we would appreciate Kerr-McGee and the People providing us with copies of the complaint, answer, and Memorandum of Opinion dated March 21, 1984, in the above case.

Kerr-McGee notes that we have not been consistent in our rulings on Contentions AG-1(g) and -2(g). That is correct. AG-1(g) asserts that the Staff must consider the Illinois ground water standards in its environmental review. That contention was admitted because, regardless of their applicability to Kerr-McGee, Staff must indeed touch on these standards even if it simply pauses to note that they are not applicable (should that turn out to be the case). AG-2(g), on the other hand, states Kerr-McGee must demonstrate that its disposal cell will not violate these ground water standards. In our view, such a demonstration should not be required if the standards are not applicable. Hence, this contention was treated differently.

We noted in LBP-84-42[22] that Kerr-McGee and the People had agreed that this contention should be interpreted to require Kerr-McGee to show that it had complied with these requirements (assuming they are applicable) prior to license authorization. Staff points out that this may not be necessary.[23] We will also explore this matter at the next prehearing conference.

The People's second point with respect to Contention AG-1(g) concerns 10 C.F.R. Part 61. As filed, that contention asserted that Staff has ignored the guidance provided by Part 61. Because Part 61 is not applicable to this proceeding, we struck this assertion in the contention.

The People argue that the underlying policies of Part 61, which pertains to land disposal of radioactive waste, are relevant to this proceeding

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[21] LBP-84-42, 20 NRC at 1325 n.84

[22] *Ibid.*

[23] Staff's November 20, 1984 Response to the People's Motion at 2.

and that Appendix A to Part 40 expresses the same policies. Consequen y, the People believe that the Staff should consider Part 61 in the FES. Kerr-McGee and Staff oppose.

This part of the People's motion is denied. Part 61 is not applicable to this proceeding, and consequently there is no obligation compelling Staff to consider it in the FES. This is not to say, however, that Part 61 might no furnish some indication of the Commission's intent should that inter not be explicitly set out in Part 40. All parties remain free to look to Part 61 (and any other relevant material) in attempting to reconcile ambiguities in Part 40.

Kerr-McGee correctly points out that we did not strike references to Part 61 in Contentions AG-2(u) and (w). This was an oversight. Those references are also stricken.

The People object to our denial of Contention AG-1(h). This contention asserted that Staff has not adequately responded to comments on the DES concerning alternate sites, the rationale for rejecting offsite disposal, and long-term environmental impacts. Because we had admitted contentions on all these points, we rejected Contention AG-1(h) as redundant.

The People argue that, because a supplemental impact statement must be circulated, Staff should respond to these comments and they point out that, by adopting Alternative III, Staff postponed a close analysis of these points. Kerr-McGee and Staff oppose, arguing that the hearing record offers the appropriate vehicle to correct any deficiencies in this regard.

The People's motion is granted. Staff has not considered permanent onsite disposal in the FES, and apparently as a consequence, felt it unnecessary to respond to these comments. We have held that Staff's failure to consider permanent onsite disposal requires that a supplemental impact statement be circulated. Staff should respond to the comments in question along with its response to the comments on the supplemental impact statement.

SUMMARY

For the convenience of the parties, we summarize our rulings below.

1. Staff's motion for reconsideration of the admission of Contention KM-1 is denied because:
   (a) Staff did not deny Kerr-McGee's application; and
   (b) Contention KM-1 is within the scope of the matters raised by Contentions AG-1 and -2.

2. The deficiencies which we found in the FES are related to Contention AG-1:

(a) The FES is inadequate to support Alternative III because of its failure to consider permanent onsite disposal; and

(b) Contention KM-1 requires Staff to review Kerr-McGee's disposal plan under UMTRCA.

3. Although we can see no practical reason for Staff to continue to pursue Alternative III, Staff is free to attempt to demonstrate that our conclusion in this regard is incorrect.

4. It is for Staff to determine in the first instance how much supplementation to the FES is necessary to comply with NEPA. In making this determination:

(a) Staff should take the People's position into account and realize that it will have to defend its conclusions at hearing; and

(b) Staff should realize that its NEPA review of permanent onsite disposal at West Chicago is meaningless if West Chicago does not meet UMTRCA standards.

5. Staff must circulate a supplement to the FES, evaluating permanent onsite disposal at West Chicago, for public comment. When Staff responds to those public comments, it must also respond to previous comments on the DES identified in Contention AG-1(h) (*see* item 8, below).

6. Staff's request for referral of our ruling admitting Contention KM-1 to an appeal board is denied.

7. The People's motion for reconsideration of our ruling on Contention AG-1(g) is denied. At the next prehearing conference in this proceeding, we wish the parties to address the question of the applicability of the Illinois ground water standards to Kerr-McGee's proposal and Staff's position that, even if these standards are applicable, we need not delay licensing action on Kerr-McGee's proposal pending compliance with them.

8. The People's motion for reconsideration of our ruling on Contention AG-1(h) is granted and that contention is admitted. In responding to public comments on the supplement to the FES, Staff must also respond to the comments identified in this contention. (*See* item 5, above.)

Order

In consideration of the foregoing, it is, this 23rd day of January 1985, ORDERED

1. Staff's motion for reconsideration of our ruling admitting Contention KM-1 or for referral to the Appeal Board is denied;

2. Staff's motion for clarification is granted consistent with the views expressed herein;

3. The People's motion for reconsideration of our rulings on Contention AG-1(g) is denied and the references to 10 C.F.R. Part 61 in Contentions AG-2(v) and (w) are stricken; and

4. The People's motion for reconsideration of our ruling denying the admission of Contention AG-1(h) is granted and that contention is admitted.

ATOMIC SAFETY AND
LICENSING BOARD

Dr. James H. Carpenter
ADMINISTRATIVE JUDGE

Dr. Peter A. Morris
ADMINISTRATIVE JUDGE

John H Frye, III, Chairman
ADMINISTRATIVE JUDGE

Bethesda, Maryland
January 23, 1985

Directors' Decisions Under 10 CFR 2.206

UNITED STATES OF AMERICA
NUCLEAR REGULATORY COMMISSION

OFFICE OF NUCLEAR REACTOR REGULATION

Harold R. Denton, Director

In the Matter of	**Docket Nos. 50-289**
	50-320
	50-219
	(10 C.F.R. § 2.206)
GENERAL PUBLIC UTILITIES NUCLEAR CORPORATION	
(Three Mile Island Nuclear Station, Units 1 and 2)	
(Oyster Creek Nuclear Generating Station)	**January 15, 1985**

The Director of the Office of Nuclear Reactor Regulation denies a petition submitted by Joanne Doroshow on behalf of the Three Mile Island Alert, Inc., and other named Petitioners requesting action with respect to the Three Mile Island Nuclear Station (TMI) Units 1 and 2 and the Oyster Creek Nuclear Generating Station.

RULES OF PRACTICE: SHOW CAUSE PROCEEDING

Where the Commission has before it the Petitioners' allegations in another proceeding, it is inappropriate to use 10 C.F.R. § 2.206 procedures to initiate a show cause proceeding.

DIRECTOR'S DECISION UNDER § 10 C.F.R. 2.206

INTRODUCTION

By Petition dated August 13, 1984, Joanne Doroshow, on behalf of Three Mile Island Alert, Inc. (TMIA), and others[1] requested that the Nuclear Regulatory Commission revoke the licenses of General Public Utilities Nuclear Corporation (GPUN) to operate Three Mile Island Nuclear Station (TMI) Units 1 and 2 and the Oyster Creek Nuclear Generating Station. As the basis for this request, Petitioners assert that GPUN lacks the requisite character to safely operate a nuclear reactor. Specifically, Petitioners allege that management's past record indicates defects in "foresight, judgment, perception, resolve, integrity and values" which reflect negatively upon its present ability to demonstrate the qualities of character required for an NRC license holder.

In accordance with usual NRC practice, the Petition was referred to the Staff for appropriate action in accordance with 10 C.F.R. § 2.206. A notice was published that the Petition was under consideration. 49 Fed. Reg. 35,447 (Sept. 7, 1984). On August 22, 1984, Petitioners filed supplemental pages to replace certain pages to the Petition, and on October 1, 1984, filed additional sections to supplement the Appendix to the Petition. On October 12, 1984, the Licensee filed its response to the Petition. The Staff has completed its evaluation of the Petition and, for the reasons stated in this Decision, the Petitioners' request is denied.

DISCUSSION

Petitioners' Allegations with Regard to TMI Unit 1

Petitioners assert a number of factual circumstances in support of their request that the license of GPUN to operate TMI Unit 1 be revoked, including that, essentially, Metropolitan Edison Company, GPUN and all GPU subsidiaries are one company and were run since before the accident by the same individuals. As such, Petitioners allege that public health and safety require that the Licensee show that its past

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[1] Additional Petitioners are Peter C. Wambach, State Representative, 103d Legislative District, Commonwealth of Pennsylvania; John Shumaker, State Senator, 15th District, Commonwealth of Pennsylvania; Pat Sordill, Essex County Women's International League for Peace and Freedom; Alan Swenson, SANE; A. Jane Perkins, Harrisburg City Council; Larry J. Hochendoner, County Commissioner, Dauphin County, Pennsylvania; Judith Marlow, Safe Energy Alternatives Alliance, Dr. D.K. Cinquemani, Essex SEA Alliance, and Louise Bradford, TMIA.

record of wrong-doing is unrelated to fundamental character flaws inherent within the company. The Staff has already considered the issues raised by the Petition. Virtually no new information or argument is presented by the Petitioners which has not been fully considered by the Staff in its analysis of the issues. *See* "TMI-1 Restart: An Evaluation of the Licensee's Management Integrity as It Affects Restart of Three Mile Island Nuclear Station, Unit 1, Docket 50-289," NUREG-0680, Supp. No. 5 (July 1984); NUREG-0680, Supp. No. 4 (October 1983), and NUREG-1020LD (September 1983). *See also* NRC Staff's Reply to Other Parties' Comments in Response to CLI-84-18, October 29, 1984; NRC Staff's Brief in Response to CLI-84-18, October 9, 1984, and NRC Staff's Comments on the Commission's January 20, 1984 List of Integrity Issues in Restart Proceeding, February 21, 1984.[2] Based upon its assessment, the Staff has concluded that GPUN can operate TMI Unit 1 without undue risk to the health and safety of the public and that these issues do not raise a bar to restart of TMI Unit 1.

Apart from the Staff's view of the substance of the Petitioners' allegations, another consideration leads me to deny Petitioners' request that the license of GPUN to operate TMI Unit 1 be revoked. The Commission itself has before it the question of whether further hearings are warranted on such matters as are covered in NUREG-0680, Supp. No. 5. *See* Commission Order, CLI-84-18, 20 NRC 808, 809 (1984). In fact, most of Petitioners' allegations have been incorporated in TMIA's response to the Commission's Order.[3] In view of the pending question before the Commission of the need for further hearings, it is inappropriate for me to initiate show cause proceedings in response to Petitioners' request. *See Pacific Gas and Electric Co.* (Diablo Canyon Nuclear Power Plant, Units 1 and 2), CLI-81-6, 13 NRC 443 (1981); *Consolidated*

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[2] As indicated in RC Staff's Reply to Other Parties' Comments in Response to CLI-84-18, at 3 n.2, some new information, not previously considered by the Staff prior to the issuance of NUREG-0680, Supp. No. 5, was cited in the Petition. The new information consisted of I&E Inspection Report 50-289/84-12, dated August 14, 1984; Attachment B of the Petition, concerning allegations of Licensee's use of unqualified welders; Attachment C of the Petition concernin[illegible]attal of Staff's conclusions on the Parks/King/Gischel issue, and the Special Report of GPU's Rec[illegible]tuted OARP Review Committee. In the Staff's Reply to Other Parties' Comments in Response to CLI-84-18, the Staff stated that it considered this material and found that it did not modify the Staff's position on any of the issues concerning restart and that the information, either separately or in conjunction with other available information, did not raise a significant safety issue.

[3] The Staff reviewed TMIA's response to CLI-84-18, including those portions of the Petition which were incorporated in TMIA's response (as well as those portions of the Petition incorporated in the Union of Concerned Scientists' response to CLI-84-18), and found no reason therein to change the Staff's previous position on the issues. *See* NRC Staff's Reply to Other Parties' Comments in Response to CLI-84-18, October 29, 1984; *see also* NRC Staff's Brief in Response to CLI-84-18, October 9, 1984.

Edison Co. of New York (Indian Point, Units 1, 2 and 3), CLI-75-8, 2 NRC 173 (1975). Petitioners' request as it relates to TMI Unit 1 is therefore denied.

Petitioners' Allegations with Regard to TMI Unit 2

As a basis for action with regard to TMI Unit 2, Petitioners make essentially the same argument as they do with regard to TMI Unit 1 that the Licensee has demonstrated such defects of character as to mandate revocation of its license to operate this facility. Specifically, Petitioners assert as a basis for action with regard to TMI Unit 2 a number of contentions relating to the Licensee's management of cleanup operations at the TMI-2 facility.

The Staff has considered the deficiencies in the TMI-2 cleanup operations raised by the OI investigation (OI Report dated September 1, 1983, Allegations Regarding Safety-Related Modifications and QA Procedures, H-83-002, and OI Report dated May 18, 1984, Allegations Regarding Discrimination for Raising Safety-Related Concerns, H-83-002) and referred to in the Petition. The violations were found individually to be of minor safety significance. Enforcement action was taken on these matters.[4] The Staff's recent reconsideration of its position regarding the results of the previous OI investigation on the polar crane may result in the earlier enforcement action being modified. In addition, the Staff is currently reviewing the technical implications of newly discovered "unlike kind" brake modifications made by the Licensee on the polar crane.[5]

The Staff has also completed an extensive Performance Appraisal Inspection which reviewed the Licensee's Quality Assurance program, safety review functions, design changes, maintenance, facility operations, corrective action systems and training. The Staff had determined, based on the results of this inspection, that the Licensee is performing adequately in each of these areas. *See* Performance Appraisal Inspection 50-320/84-08 (May 15, 1984). For these reasons, Petitioners' request for revocation of the license for TMI Unit 2 is denied.

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[4] A Notice of Violation, Severity Level IV, was issued February 3, 1984. Letter from Richard DeYoung to P.R. Clark, President, GPUN (EA 83-89).

[5] Letters from Bernard J. Snyder, Program Director, TMI Program Office, Office of Nuclear Reactor Regulation (NRR) to F.R. Standerfer, Vice President/Director, TMI Unit 2 (October 9, 1984 and October 18, 1984).

Petitioners' Allegations with Regard to Oyster Creek Nuclear Generating Station

Petitioners assert as a basis for their request for action with regard to Oyster Creek that there is a nexus between management of Oyster Creek and TMI Units 1 and 2 such that Licensee's poor record regarding its management of TMI Units 1 and 2 can reasonably be viewed as arising from defects of character also affecting safe operation of the Oyster Creek facility. In addition, Petitioners express concern regarding Edward G. Wallace. Mr. Wallace is currently Manager of the Expanded Safety Systems Facility Project at Oyster Creek. While Manager of Licensing of TMI in 1979, Mr. Wallace drafted a "dishonest" response to a Notice of Violation (NOV) issued by NRC after the accident.

The Staff has evaluated Met-Ed's response to the NOV with respect to current GPUN management integrity and concluded that there is reasonable assurance that GPUN can operate TMI-1 with no undue risk to public health and safety. The Staff's review of circumstances surrounding the response to the NOV is contained in NUREG-0680, Supplement No. 5, at 8-15 through 8-22. As a separate matter, the Staff is considering what enforcement action, if any, is appropriate regarding the Licensee's response to the NOV. Petitioners are correct that Mr. Wallace was involved in preparing the response to the NOV and is currently in a "technical" management position with respect to Oyster Creek. However, Mr. Wallace is not involved in any way in the on-line operation of the Oyster Creek facility. His role is that of an offsite project manager and his current work (e.g., project management of the Expanded Safety Systems Facility) is subject to extensive review by GPUN management as required by 10 C.F.R. § 50.59. Moreover, Mr. Wallace was candid and cooperative during the OI investigation of this matter. For these reasons, the Staff finds that there is reasonable assurance that GPUN can and will meet its regulatory responsibilities with no undue risk to public health and safety with Mr. Wallace in his present management position.

The Licensee's response to the NOV is relevant to proceedings concerning the restart of TMI-1; however, the Staff has argued before the Commission that it is not material to a restart decision and that further hearings on this matter should not be required. The Petitioners have filed their views with the Commission as part of the TMI-1 Restart Proceeding (*see* TMIA Response to Commission Order of September 11, 1984, dated October 9, 1984 and TMIA Reply Comments to NRC Staff's Brief in Response to CLI-84-18, dated October 29, 1984). If the record is subject to reopening for further hearings and the TMI-1 Restart

Proceeding dictates a different result with respect to Mr. Wallace, appropriate action will be taken at that time.

Additionally, Petitioners argue that Oyster Creek has a notably poor record in its own right. As a basis for this allegation, the Petitioners allege that Oyster Creek was so poorly run that it had to be shut down for maintenance work since early 1983, that results of a study undertaken by Rohrer, Hibler and Replogle, Inc. (RHR) show that training is still inadequate, and that the results of a study by Basic Energy Technology Associates, Inc. (BETA) show that plant maintenance at Oyster Creek has not yet reached the point where required equipment reliability can be reasonably assumed.

With regard to the shutdown of Oyster Creek, the outage was not, as Petitioners allege, due to a poor operating record. Rather, the extended shutdown was a pre-planned outage to accomplish plant maintenance and modification activities. These activities involved preventive and corrective maintenance, surveillance testing and inspection, and engineering and installation of improved design features. The nature and extent of the activities had been planned and integrated in a systematic manner. Due to the extent of the outage, the Staff recognized the need to assess the condition of the plant and operators prior to the resumption of licensed operations. The results of such assessment are discussed in a memorandum dated September 28, 1984, from the Regional Administrator of Region I to the Directors of the Office of Inspection and Enforcement and the Office of Nuclear Reactor Regulation. A copy of the memorandum is being provided to Petitioners with this Decision. The Staff has determined that during the outage the Licensee has taken action to improve the physical plant and confirm through testing the adequacy of existing plant conditions, that these actions are indicative of a responsible Licensee, and that the results are not indicative of a poorly run facility.

Special inspections were performed (*see* Inspection Report No. 50-219/84-06) with respect to maintenance to review organizational structure, administrative controls, organizational interfaces, completed safety-related work packages, and preventive maintenance. Extensive corrective and preventive maintenance was satisfactorily performed during the outage. Prior to resumption of power operations, appropriate surveillance testing was performed to assure conformance with the technical specifications. The Staff is satisfied that the technical specifications are adequate for monitoring required equipment reliability and determining whether or not the plant can operate with degraded equipment.

With regard to Petitioners' allegations based on their review of the BETA and RHR reports, the Staff has specifically reviewed training, maintenance, and adherence to procedures at Oyster Creek in order to independently assess Licensee performance in these areas. The results of the Staff's efforts with respect to the BETA and RHR reports are documented in Inspection Report No. 50-219/84-06. The review and inspections focused on the current staff and plant systems and procedures. The Staff's efforts were geared towards obtaining a better understanding of current attitudes and conditions as exhibited by current performance.

With regard to procedures and adherence to procedures, the Staff has specifically examined, during 1983 and 1984, policies governing plant operations. As discussed in the Staff's assessment memorandum and its referenced reports, policies are widely distributed and generally well understood by plant operators and supervisors. Procedures have been found to be technically adequate and capable of being properly implemented. Licensee's management has demonstrated a strong commitment at Oyster Creek to adherence to procedures and requirements. Accordingly, the Petitioners' allegations do not provide an adequate basis for initiating show cause proceedings to revoke the Oyster Creek license.

CONCLUSION

Based upon the foregoing discussion and the information contained in the referenced documentation, I have concluded that no adequate basis exists for revocation of GPUN's license to operate TMI Unit 1 or Oyster Creek or to maintain TMI Unit 2. Accordingly, the Petitioners' request has been denied. A copy of this decision will be filed with the Secretary for the Commission's review in accordance with 10 C.F.R. § 2.206(c) of the Commission's regulations.

Harold R. Denton, Director
Office of Nuclear Reactor
Regulation

Dated at Bethesda, Maryland,
this 15th day of January 1985.

UNITED STATES OF AMERICA
NUCLEAR REGULATORY COMMISSION

OFFICE OF NUCLEAR REACTOR REGULATION

Harold R. Denton, Director

In the Matter of | **Docket No. 50-295 (10 C.F.R. § 2.206)**

COMMONWEALTH EDISON COMPANY
(Zion Station, Unit 1) | **January 23, 1985**

The Director of the Office of Nuclear Reactor Regulation grants in part and denies in part a Petition by Edward Gogol alleging inadequacies in the containment integrated leak rate test performed in 1981 at Zion Nuclear Power Station, Unit 1. The Petition sought a variety of relief including immediate NRC action to deal with the threat raised by the alleged inadequate leak rate test of the Zion Unit 1 facility and the completion of an adequate and properly supervised retesting of the facility. Petitioner also requested copies of all documents collected by either the licensee or the NRC in the course of the retest.

TECHNICAL ISSUE DISCUSSED: CONTAINMENT LEAK RATE TESTING

Discrepancies in the Containment Integrated Leak Rate Test (CILRT) for the Zion Nuclear Power Station, Unit 1 required retesting of the facility to demonstrate compliance with 10 C.F.R. Part 50, Appendix J.

RULES OF PRACTICE: ORDERS

It is not necessary for the NRC to issue orders in response to a petition pursuant to 10 C.F.R. 2.206 when the licensee agrees to take remedial measures similar to those requested by the petition.

DIRECTOR'S DECISION UNDER 10 C.F.R. § 2.206

On June 5, 1984, Mr. Edward Gogol filed on behalf of Citizens Against Nuclear Power a Petition for Emergency Relief (Petition). The Petition contended that the Commonwealth Edison Company's (CECo) document "Zion Unit 1 Reactor Containment Building Integrated Leak Rate Test Report," dated April 24, 1981, revealed that repeated efforts were made to obtain a satisfactory verification test to validate the performance and reliability of the basic test performed on March 12, 1981, at Zion Nuclear Power Station, Unit 1. The Affidavit of Dr. Zinovy V. Reytblatt, attached to the Petition, contended that these repeated efforts to obtain a satisfactory verification test demonstrated that the basic test had been deficient. Consequently, it was alleged that the American National Standards Institute ANSI N45.4-1972 specified in Appendix J to 10 C.F.R. Part 50 was not met and, accordingly, Zion Nuclear Power Station, Unit 1 was in noncompliance with the Commission's regulations regarding containment leak rate testing. Based on the above allegation, the Petition requested the following relief: (1) that the NRC act immediately to remove the threat posed by this situation; (2) that the NRC immediately order CECo to perform a scientifically valid Containment Integrated Leak Rate Test on Zion Nuclear Power Station, Unit 1; (3) that the NRC supervise and review this test, and certify both that this test is scientifically valid and performed in accordance with ANSI N45.4-1972; (4) that a copy of all documents containing actual test data, test logs, calculations, graphs, etc., collected by CECo or the NRC in the course of this test or its review, be provided on a timely basis to the Petitioner; and (5) that if (1) through (4) are not or cannot be accomplished, that Zion Nuclear Power Station, Unit 1 operating license be suspended. The request for documents was reiterated in Mr. Gogol's letter of August 6, 1984.

As a result of the Petition, the NRC Region III Office investigated the various allegations contained in the Petition. The regional inspectors performed a special inspection of the 1981 and 1983 Containment Integrated Leak Rate Tests (CILRT) performed for the Zion Nuclear Power Station, Unit 1.

The inspection identified discrepancies in the above-mentioned CILRTs and, on July 19, 1984, the Region III Office notified CECo that Zion Nuclear Power Station, Unit 1 was not in compliance with Appendix J to 10 C.F.R. Part 50 and the Zion Nuclear Power Station, Unit 1 Technical Specifications. A copy of the Region's notification was sent to Mr. Gogol as an enclosure to the Director's letter to him, dated July 30, 1984, acknowledging receipt of the Petition. The Inspection Reports documenting the Region III Office's inspection findings (50-295/84-11 and 50-305/84-11) were also sent to Mr. Gogol, along with twenty-seven other documents in NRC s possession relevant to the CILRTs performed at Zion Nuclear Station, Unit 1, by letter dated September 27, 1984.

Upon notification by the Region III Office, CECo voluntarily shut down Zion Nuclear Power Station, Unit 1 and performed a valid CILRT, portions of which were witnessed by Region III inspectors. The results of that inspection are also contained in Inspection Reports 50-295/84-11 and 50-304/84-11.

The CILRT showed Zion Nuclear Power Station, Unit 1 containment integrity. Consequently, Zion Nuclear Power Station, Unit 1 containment has been demonstrated to be in compliance with Commission regulations in 10 C.F.R. Part 50, Appendix J.

To the extent that the Petition sought immediate NRC action to remove any threat posed by unacceptable CILRTs at Zion Nuclear Power Station, Unit 1, such actions were taken and the relief requested by the Petition was granted. To the extent that the Petition sought NRC review of the CILRT conducted at Zion Unit 1 and copies of all documents in the possession of the NRC regarding that CILRT, those portions of the Petition have also been granted.

The remainder of the Petition is denied. It was not necessary for the NRC to issue an order in this matter, because CECo agreed to take remedial measures similar to those requested upon notification that the plant did not comply with Appendix J. *See Rochester Gas and Electric Corp.* (R.E. Ginna Nuclear Power Plant), DD-82-3, 15 NRC 1348, 1357-58 (1982). Nor is it appropriate for the NRC to supervise a CILRT or certify its validity. Compliance with NRC regulations is the responsibility of the Licensee. The NRC did review the test, and this review provides reasonable assurance that the Commission's regulations are met. Consequently, there is no cause to suspend the operating license for Zion Nuclear Power Station, Unit No. 1.

To the extent the Petition sought documents collected by CECo but not in the possession of the NRC, the request is denied for the same reasons I stated in an earlier Director's Decision on a Petition filed by Mr. Gogol which requested similar relief.[1] As noted there, to honor such a request would impose substantial burdens and costs on the NRC without a clear corresponding benefit. Section 2.206 does not provide a means for general discovery of documentation in the possession of Commission

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[1] *Commonwealth Edison Co.* (LaSalle County Station, Units 1 and 2), DD-84-6, 19 NRC 891, 895-96 (1984).

licensees. *See Texas Utilities Generating Co.* (Comanche Peak Steam Electric Station, Units 1 and 2), DD-83-11, 18 NRC 293, 295 (1983).

For the reasons stated in this Decision, the Petitioner's request for action pursuant to 10 C.F.R. § 2.206 has been granted in part and denied in part. As provided by 10 C.F.R. § 2.206(c), a copy of this Decision will be filed with the Secretary for the Commission's review.

Harold R. Denton, Director
Office of Nuclear Reactor
Regulation

Dated at Bethesda, Maryland,
this 23rd day of January 1985.