STATE OF ILLINOIS :
SS
COUNTY OF Cook :

UNITED STATES OF AMERICA
NUCLEAR REGULATORY COMMISSION

BEFORE THE ATOMIC SAFETY AND LICENSING BOARD

In the Matter of	)	
	)	Docket Nos. 50-329-OM
CONSUMERS POWER COMPANY	)	50-330-OM
	)	50-329-OL
(Midland Plant, Units 1 and 2)	)	50-330-OL

TESTIMONY OF DR. W. GENE CORLEY
CONCERNING THE MIDLAND
CONCRETE WALL CRACK REPAIR PROGRAM

My name is W. Gene Corley. I am a Divisional Director, Engineering Development Division, Construction Technology Laboratories, a Division of the Portland Cement Association. The Portland Cement Association (PCA) is a nonprofit Illinois corporation devoted to the improvement in uses of Portland Cement Concrete. PCA has been retained as a consultant to Consumers Power Company, and I am the representative of the PCA who is most familiar with the issues described in this testimony.

I am joint author, with A. E. Fiorato and D. C. Stark, of Attachment 1, which is a PCA report entitled "Effects of Cracks on Serviceability of Structures at Midland Plant."

This report was originally submitted to the NRC Staff in April, 1982.

Subsequently, on August 2, 1982, Consumers Power Company agreed to a repair program for concrete cracks in category I safety grade buildings affected by soil fill which is more extensive than that recommended by PCA. (Attachment 2). I have no objection to these additional commitments and I believe that the repair program described in Attachment 2 is more than adequate to ensure that long term serviceability of category I safety related structures affected by soil fill conditions at the Midland Plant is not impaired by the observed cracks.

I have an M.S. and a Ph.D. in structural engineering from the University of Illinois and over twenty years of experience as a structural engineer, as described in more detail in my attached resume. My experience has included design, construction and testing of concrete structures. In addition, Ihave acted as a specialized consultant on many jobs where construction problems or structural damage have occurred. This specialized consulting work has included field inspections to evaluate earthquake damage, blast damage, damage caused by settlement, and other conditions relevant to questions raised by the NRC Staff in their review of the Midland plant. My previous work has also included development of information on fatigue properties of reinforcing bars and nonferrous metals.

I am a registered structural engineer in the state of Illinois, and a registered professional engineer in three other states.

I am currently a member of the American Concrete INstitute (ACI) Committee 318 on standard building code. In addition, I am a member of the ACI Technical Activities Committee, which has the responsibility for reviewing and approving all technical changes in all ACI codes and specifications, including ACI 318 and ACI 349.

I have personally visited the site and inspected the structures there which have displayed concrete cracking. I believe that based on my education and work experience and this inspection of the Midland structures, I am qualified to testify as an expert concerning the matters described in this testimony.

I swear that the statements made in this testimony and Attachment 1 are true and correct, to the best of my knowledge and belief.

Dr. W. Gene Corley

Subscribed and sworn to
before me this 8th day
of November, 1982.

Notary Public

Attachment 1

Report to

CONSUMERS POWER COMPANY
JACKSON, MICHIGAN

EFFECTS OF CRACKS ON SERVICEABILITY OF STRUCTURES AT MIDLAND PLANT

by

W. G. Corley, A. E. Fiorato, and D. C. Stark

Submitted by
CONSTRUCTION TECHNOLOGY LABORATORIES
A Division of the Portland Cement Association
5420 Old Orchard Road
Skokie, Illinois 60077

April 19, 1982

TABLE OF CONTENTS

EFFECTS OF CRACKS ON SERVICEABILITY OF STRUCTURES AT MIDLAND PLANT

by

W. G. Corley, A. E. Fiorato, and D. C. Stark*

INTRODUCTION

A series of previous reports have presented an evaluation of the structural significance of cracks observed in the Feedwater Isolation Valve Pits, Auxiliary Building Control Tower and Electrical Penetration Areas, Diesel Generator Building, and Service Water Pump Structure at Midland Nuclear Power Plant Units 1 and 2.[(1-4)**] Observed cracks in these structures were described and the significance of the cracks with regard to future load carrying capacity was discussed. A site plan for the Midland Plant, which indicates buildings evaluated, is shown in Fig. 1.

This report contains a discussion of effects of observed cracks on serviceability of the structures evaluated. Primary emphasis is given to durability of the concrete structures over their service life. Recommendations for repair of selected areas are also made.

*Respectively, Divisional Director, Engineering Development Division; Director, Construction Methods Department; and Principal Research Petrographer, Concrete Materials Research Department, Construction Technology Laboratories, a Division of the Portland Cement Association, 5420 Old Orchard Road, Skokie, Illinois 60077.

**Numbers in parentheses refer to references listed at the end of this report.

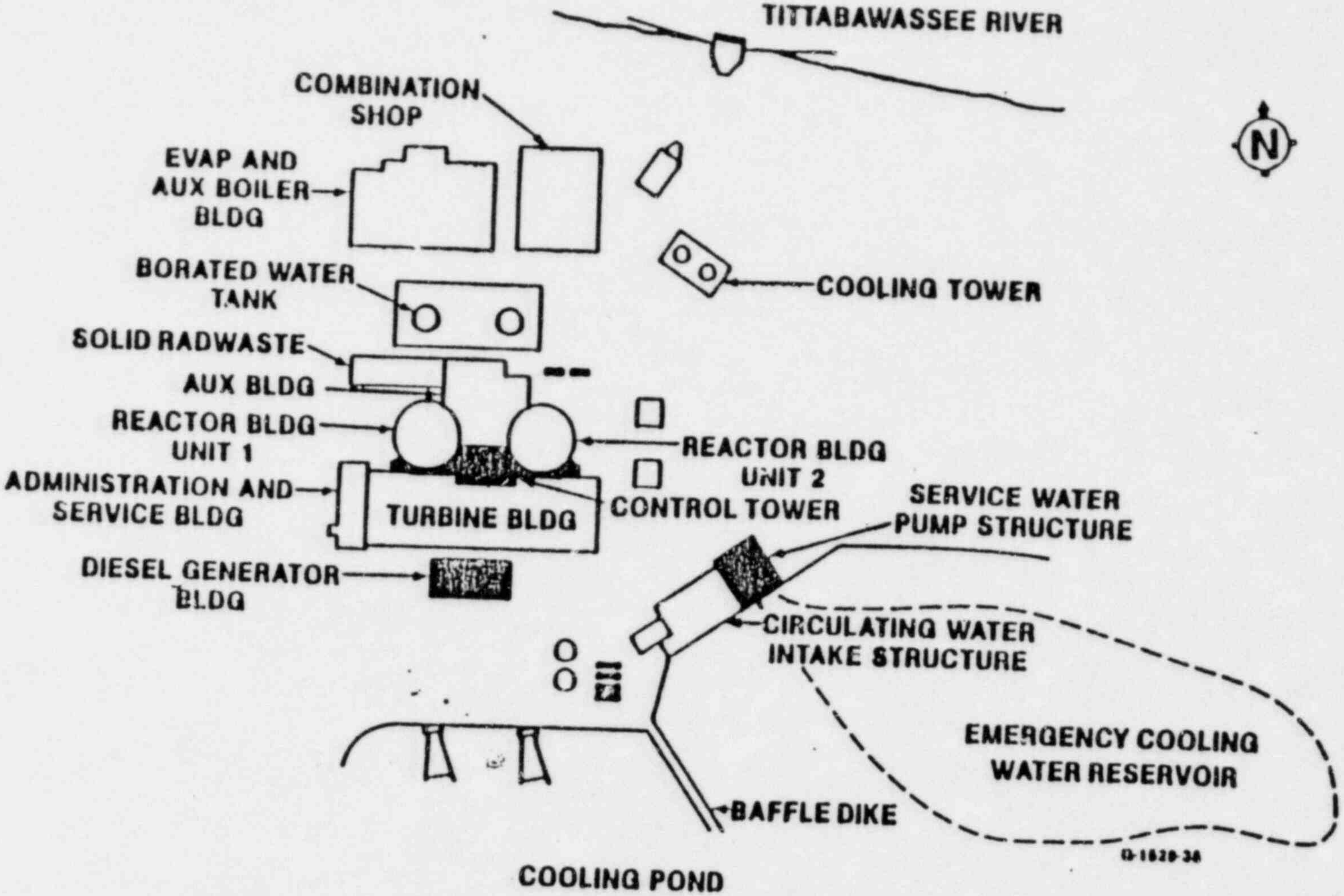

FIG. 1 SITE PLAN

OBSERVED CRACKS IN MIDLAND PLANT STRUCTURES

Cracks observed in the Feedwater Isolation Valve Pits and the Auxiliary Building Control Tower and Electrical Penetration Areas of Midland Plant Units 1 and 2 were primarily attributed to restrained volume changes that occurred during curing and drying of concrete. Cracks observed in the Diesel Generator Building were attributed to restrained volume changes, and reported differential settlement between duct banks under the building and the north and south portions of the building. Cracks observed in the Service Water Pump Structure were attributed primarily to restrained volume changes although the occurrence of settlement related cracking could not be entirely dismissed.

In terms of future serviceability of these structures, and potential problems with durability, cracks located in exterior exposed surfaces would be expected to have the most significant influence. This is because exposure conditions for exterior surfaces are more severe than those for interior surfaces. Maximum reported crack width in exterior surfaces of structures investigated at Midland was approximately 0.025 in. However, most observed cracks were significantly smaller than this maximum value. The fact that observed crack widths were spread over a wide range is consistent with most observations of cracking in concrete members. Crack widths are inherently subject to wide scatter.[5,6]

American Concrete Institute Committee 224 lists "tolerable crack widths" for reinforced concrete members as a function of different exposure conditions.[6] For interior members, a "tolerable crack width" of 0.016 in. is listed. For exterior members subject to humidity, moist air, or in contact with soil, the "tolerable crack width" is listed as 0.012 in. ACI Committee 224 emphasizes that "it should be expected that a portion of the cracks in the structure will exceed these values by a significant amount."[6] Committee 224 also notes that their tabulation of width limits "is a general guide for tolerable crack widths at the tensile face of reinforced concrete structures for typical conditions and is presented as an aid to be used during the design process."[6] The crack widths are related to service conditions.

The presence of crack widths in excess of selected tolerable values occurs because crack limits can only be related to equations that predict "probable" maximum widths.[6] Although this probable value usually means that approximately 90 percent of crack widths in the member are below the calculated value, isolated cracks in excess of twice the width of the computed maximum can occur.[6] Research data also indicate that the range in randomness of crack widths increases with size of member.[6]

It should also be noted that equations for evaluating crack widths of flexural members are related to instantaneous or short term loading. Volume changes related to shrinkage, creep, or temperature and humidity variations, are not taken into account. For beams under nominally constant loading, research data have shown that crack widths can increase significantly with time.[7]

Thus, the maximum width would not be expected to remain constant after a crack initially forms. Therefore, in evaluating cracks in an existing structure, tolerances developed for design can not be arbitrarily applied.

For structures evaluated at the Midland Plant, most of the cracking, and crack growth, related to restrained volume changes should have taken place since construction was completed. Future movement of cracks related to normal volume and temperature changes should not affect conclusions developed in this report. However, cracks that may develop as a result of unanticipated settlement or from underpinning operations should be evaluated to determine their effects. The need for repair of such cracks can only be determined after their significance has been evaluated. Evaluation of such cracks has been included as part of the "Recommended Program for Monitoring Structural Integrity" of Midland Plant structures.[1-4]

Based on the above discussion, crack widths observed in structures investigated at the Midland Plant are judged to be within the range implied by published tolerable crack width limits.

DURABILITY OF CONCRETE STRUCTURES AT MIDLAND

This discussion covers durability of concrete as related to structures investigated at the Midland Plant. Emphasis is given to durability questions relevant to observed cracks in the Feedwater Isolation Valve Pits, Auxiliary Building Control Tower and Electrical Penetration Areas, Diesel Generator Building, and Service Water Pump Structure. Prior to discussing specific measures for each structure, a basic discussion of durability of concrete structures is presented.

Durability of concrete is defined as "its ability to resist weathering action, chemical attack, abrasion, or any other process of deterioration."[8,9] With regard to questions of potential durability problems in Midland Plant structures, three types of concrete deterioration were considered: freezing and thawing, chemical attack, and corrosion of reinforcement.

Freezing and Thawing

Although the actual mechanism is quite complicated, freeze-thaw damage is basically caused by expansion and diffusion of freezing water in the pore system of cement paste and aggregates.[8,9,10] Freeze-thaw cycles cause progressive deterioration as a result of continued expansive pressures from excess water that freezes in concrete. Since freeze-thaw deterioration requires the presence of absorbed water that can be frozen, the occurrence of freeze-thaw deterioration on vertical surfaces is rare.

Resistance to freeze-thaw damage is obtained by designing structural members to minimize exposure to moisture, by using concrete having low in-place permeability, by using a low water-cement ratio, by using air-entrainment, and by using sound aggregates.[8,9,10] Concrete with low permeability does not absorb as much water which can later freeze.

According to information provided by Bechtel, concrete mixes used in walls of the buildings investigated at the Midland Plant had water-to-cementitious material ratios ranging from 0.41 to

0.47. These ratios are within the limit of 0.50 recommended by American Concrete Institute Committee 201 for concrete resistance to freeze-thaw damage.[8] In addition, since exterior exposed surfaces in walls of the structures are unlikely to collect or transmit water, occurrence of freeze-thaw damage is judged to be unlikely. It is not expected that cracks of the type observed in the inspected structures would have potential to collect and retain water.

Chemical Attack

Dry concrete does not react with dry chemicals.[8,9] For deterioration to take place, chemicals must be in solution and in sufficient concentration to provide an aggressive environment.[8,9] Although buildings are exposed to a number of potentially corrosive chemicals under normal environmental and atmospheric conditions, concretes generally resist chemical attack from normal conditions of exposure.

American Concrete Institute Committee 515 has prepared detailed tables on effects of chemicals on concrete.[11] General types of chemical attack include acid or alkali attack, or sulfate attack. Concrete's resistance to chemical attack is dependent upon the type and concentration of the chemical solution in contact with the concrete, the temperature and pressure of the solution, and the quality of the concrete.[9]

Deterioration of concrete by acids is primarily the result of the reaction of acids with calcium hydroxide in the hydrated portland cement paste.[8,11] This results in the formation of water-soluble reaction products and subsequent disintegration of the concrete. Strong alkaline solutions (over 20%) attack other constituents in the hardened paste to cause disintegration.(8,11) Sulfate attack results from complex chemical reactions between sulfate solutions and constituents of hydrated portland cement paste that result in expansive compounds which cause progressive disintegration of concrete.(8,11) In all cases the rate of chemical attack is more rapid in warmer climates.(8-11)

Conditions at the Midland Plant suggest the following hypothetical situations as being conducive to chemical attack:

1. Highly concentrated acid solutions in the cooling pond that could attack concrete in walls of the Service Water Pump Structure.
2. High sulfate contents in the soil, in the cooling pond, or in groundwater adjacent to the concrete structures.
3. Atmospheric pollution that could, in combination with moisture, form "acid rain."

According to Michigan MPDES Permit Application, Amendment 3, dated September 30, 1981, the pH* level of the cooling pond water can range from 7.0 to 9.0. This pH level can be compared to that of potable groundwater which has a pH of approximately 7.0. Seawater has a pH range from 8.0 to 9.0 Thus, pH levels of the cooling pond water are not unusual.

*The pH value of a solution is a measure of its acidity on basicity. A neutral solution, or pure water, has a pH of 7. Stronger acids have lower pH values.(9)

With regard to sulfate attack, no unusual levels of sulfates in soils or groundwater at the Midland Plant have been reported to Construction Technology Laboratories staff. Sulfate levels in the cooling pond are listed in the Michigan MPDES Permit Application, Amendment 3, dated September 30, 1981. According to the permit, sulfate levels can reach maximum values of 908 mg/l (908 ppm of SO_4). This compares to values of 2500 to 3000 mg/l of sulfate present in seawater. Potable ground water has a sulfate level of approximately 30 mg/l.

American Concrete Institute Committee 201 considers sulfate levels in water of 150 to 1500 mg/l as a "moderate exposure" condition, and recommends a maximum water-cement ratio of 0.50 for this exposure condition. As mentioned previously, structures at the Midland Plant have water-to-cementitious material ratios of 0.41 to 0.47. These ratios are below the limit recommended by ACI Committee 201. Committee 201 also recommends that Type II cement be used for "moderate exposure" conditions. According to Bechtel, Type II cements were used in concretes for the structures evaluated. Therefore, the structures should have adequate resistance to sulfate attack.

Generally, air pollution severe enough to cause damage to concrete structures would not be tolerated on the basis of environmental concerns. Therefore, it is not anticipated that external walls which are exposed to the atmosphere at the Midland Plant would be susceptible to any more damage than would occur in any concrete structure located in a similar environment.

With regard to concrete's resistance to chemical attack, the presence of cracks would expose more surface area to chemical solution. However, considering the exposure conditions and concrete quality for structures at the Midland Plant, it is concluded that chemical effects would not be any more severe than for other concrete structures in the area.

Corrosion of Reinforcement

Concrete normally provides a high degree of corrosion protection for embedded reinforcement.[8,9] This protection occurs because high alkalinity of the concrete provides a passive environment for the steel. In addition, air dry concrete provides a relatively high electrical resistivity which helps to resist corrosion.[8]

Corrosion of reinforcing steel is considered to be an electrochemical process.[8,9] Electrochemical corrosion results from flow of electric current and accompanying chemical reactions within the concrete. Flow of electric current can be induced by stray electrical currents, by contact between different metals in concrete, or by differential concentration cells that may develop within the concrete. The principal type of electrochemical corrosion in concrete structures occurs as a result of corrosion cells that develop within the concrete and steel.[8]

Normally corrosion is prevented because a passive iron oxide film forms on the surface of the steel. This film occurs in the presence of moisture, oxygen, and water-soluble alkaline products formed during hydration of cement. However, the passive film can be destroyed if the alkaline environment of the concrete is lost. Reduction in alkalinity can occur by carbonation of the hydrated portland cement or by ingress of chloride ions in the presence of oxygen.[8,9] Penetration of oxygen and chloride ions through concrete can result in corrosion cells being formed. The cells form when anodic and cathodic areas develop along steel reinforcement because of differences in moisture content, oxygen concentration, and chloride ion concentration.[8] Corrosion is initiated at anodic areas on reinforcement.

Since products of corrosion ("rust") take up a larger volume than the original steel, expansive forces are eventually generated as corrosion becomes severe. These forces can cause cracking and spalling. Primary elements essential for electrochemical corrosion in reinforced concrete are:

1. Presence of an electrolyte
2. Presence of oxygen

An electrolyte is a solution capable of conducting electric current by ionic flow.[8] For example, moisture and chloride ions will form an electrolyte capable of conducting a "corrosion current."

Generally, steps taken to prevent corrosion are related to providing a low permeability concrete with adequate cover over reinforcing steel. While it would appear that presence of cracks in concrete structures would increase risk of corrosion, no conclusive evidence has been found to indicate that any relationship exists between crack widths and corrosion.[12] It has been found that cracks with widths less than 0.06 in., which run approximately transverse to the direction of reinforcing steel, have little influence on corrosion.[8,12] A greater risk of corrosion occurs from cracks that run along the line of the reinforcing bar.[8,12]

For structures investigated at the Midland Plant, it is not anticipated that corrosion would be a problem with regard to future durability. The presence of cracks in exterior wall surfaces above grade will have little effect on corrosion because these areas are not subject to moisture conditions conducive to corrosion damage. The same is true for walls that are below grade level but above the water table.

For walls below the water table and for the south wall of the Service Water Pump Structure adjacent to the cooling pond, the potential does exist for build up of chloride ions as a result of alternate wetting and drying of concrete.

It should be noted that the chloride level in the cooling pond adjacent to the Service Water Pump Structure is relatively low. According to the Michigan MPDES Permit Application, Amendment 3, dated September 30, 1981, chloride (Cl) concentration in the cooling pond can reach a maximum of 425 mg/l. This concentration can be compared to the level of chloride in seawater which can be 19,000 mg/l. Potable ground water would have chloride levels of approximately 20 mg/l. Thus, the cooling pond environment is not severe. However, as a precaution against possible build up of chloride ions in the splash zone of the cooling pond, it is recommended that this area of the wall be coated to prevent possible ingress of chloride.

The Michigan MPDES Permit Application also indicates that the pH level of the cooling pond water can range from 7.0 to 9.0. This pH level can be compared to that of seawater which ranges from 8.0 to 9.0 and that of potable groundwater, which is approximately 7.0. The pH level in the cooling pond water is not considered to be low enough to severely reduce the alkaline environment that the concrete provides for reinforcement.

RECOMMENDATIONS FOR REPAIR

Epoxy injection of existing cracks above the water table in the Feedwater Isolation Valve Pits, the Auxiliary Building Control Tower and Electrical Penetration Areas, the Diesel Generator Building, or the Service Water Pump Structure is not required to ensure future structural integrity. Epoxy injection would have no influence on capacity of these structures since the existing cracks are not detrimental to capacity.

Although epoxy injection would increase overall stiffness of the cracked structures, it is unlikely that original stiffness would be recovered,[13] nor is it necessary to recover the original stiffness.

Epoxy injection of existing cracks in exterior and interior walls above the water table is not considered essential to ensure durability of the structure. Freeze-thaw damage is not considered likely in the walls because the vertical surfaces provide adequate drainage to prevent water from being trapped.

Freeze-thaw deterioration does not occur in unsaturated concrete. In addition, atmospheric exposure conditions at the Plant are not reported to be unusually severe. Therefore, deterioration from chemical attack is not anticipated. Finally, in the absence of chloride ions, the alkaline atmosphere at the level of the reinforcing bars will prevent damage from corrosion in walls above the water table.

For cracks in walls below the water table, epoxy injection or other means of stopping leakage is recommended. This recommendation represents a precautionary measure against possible durability problems that could result from a gradual build up of chlorides or sulfates as concrete is subjected to repeated wetting and drying. Epoxy injection can be applied from the interior surface. Only cracks with visible signs of leakage need to be injected. A water insensitive epoxy system should be used. General guidelines on epoxy injection have been reported by American Concrete Institute Committee 546.[14]

It is recommended that a surface coating be applied to the exterior of the south wall of the Service Water Pump Structure. This coating should cover the splash zone area of the wall adjacent to the cooling pond.* This recommendation is a precautionary measure against possible corrosion problems that

*It is reported that the water level in the south cells of the Service Water Pump Structure is maintained at the same elevation as the cooling pond. Since conditions in these cells are not conducive to repeated wetting and drying, as in the exterior splash zone, coating of interior walls is not considered necessary.

could result if a gradual build up of sufficient chloride ion occurs as the concrete adjacent to the cooling pond is subjected to repeated wetting and drying. The coating will restrict ingress of chloride ions carried by the cooling pond water.

The splash zone can be generally defined as the portion of wall subject to repeated wetting and drying. According to the Midland Plant Final Safety Analysis Report, Revision 33, dated April 1981, the maximum operating water level in the cooling pond is at elevation 627 ft. The minimum level is at elevation 618 ft. The minimum level is based on a 100-day drought with no stream withdrawals made from the Tittabawassee River. Thus, the minimum level would not be reached under normal conditions. The normal operating level of the cooling pond ranges from elevation 626 ft to elevation 627 ft.

It is recommended that the exterior surface of the entire width of the south wall be coated between elevation 626 ft and elevation 637.5 ft. This will provide protection from chloride build up caused by repeated wetting and drying under normal operating conditions.

Peformance criteria for the coating material include:

1. The coating material should cover cracks
2. The coating material should have a low enough modulus to permit natural movement of cracks
3. The coating should be able to withstand the range of environmental conditions that can be encountered at the site
4. The coating should be water resistant

5. The coating should bond to damp concrete
6. The coating material should resist debonding from moisture movement or vapor pressure within the wall
7. The coating should exhibit long-term stability
8. The coating should not react with chemicals in cooling pond water

According to manufacturers' data, the following coatings are considered suitable for the intended application:

1. Rubberstone Hi-Fill Fibrated.

   United Coatings, Inc.
   1130 E. Sprague Avenue
   Spokane, Wash. 99202

2. Aquaflex

   Dural International Corp.
   95 Brook Avenue
   Deer Park, N.Y. 11729

3. Sika-Top 144

   Sika Chemical Corp.
   Box 297
   Lyndhurst, N.J. 07071

Other suitable coatings may be available. American Concrete Institute Committee 515 provides recommendations for use of waterproofing barrier systems on concrete.[11]

It is recommended that repairs be made after completion of underpinning operations.

SUMMARY AND CONCLUSIONS

This report presents a discussion of observed cracks in the Feedwater Isolation Valve Pits, Auxiliary Building Control Tower and Electrical Penetration Areas, Diesel Generator Building, and Service Water Pump Structure located at Midland Nuclear Power

Plant Units 1 and 2. Effects of observed cracks on future durability of the structures are discussed.

Observed cracks in walls above the water table are not expected to have a significant influence on future durability of the structures. Therefore, epoxy injection of these cracks is not considered necessary.

For cracks in walls below the water table, it is recommended that epoxy injection or other means be used to stop leakage. This precautionary measure is intended to prevent possible corrosion problems that could result from gradual build up of chloride ions.

It is also recommended that the south wall of the Service Water Pump Structure be coated within the splash zone area adjacent to the cooling pond. The coating represents a precautionary measure against possible corrosion problems that could result from gradual build up of chloride ions.

It is recommended that repairs be made after completion of underpinning operations.

Epoxy injection of existing cracks is not required to ensure future structural integrity.

REFERENCES

1. Corley, W. G. and Fiorato, A. E., "Evaluation of Feedwater Isolation Valve Pits at Midland Plant," Report to Consumers Power Company, Construction Technology Laboratories, a Division of the Portland Cement Association, Skokie, Illinois, January 1982, 24 p. + Appendix.

2. Corley, W. G. and Fiorato, A. E., "Evaluation of Auxiliary Building Control Tower and Electrical Penetration Areas at Midland Plant," Report to Consumers Power Company, Construction Technology Laboratories, a Division of the Portland Cement Association, Skokie, Illinois, January 1982, 44 p. + Appendix.

3. Corley, W. G. and Fiorato, A. E., "Attachment 4 - Evaluation of Cracking in Diesel Generator Building at Midland Plant," Report to Consumers Power Company, Construction Technology Laboratories, a Division of the Portland Cement Association, Skokie, Illinois, February 1982, 34 pp.

4. Corley, W. G. and Fiorato, A. E., "Evaluation of Cracking in Service Water Pump Structure at Midland Plant," Report to Consumers Power Company, Construction Technology Laboratories, a Division of the Portland Cement Association, Skokie, Illinois, February 1982, 40 p. + Appendix.

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6. ACI Committee 224, "Control of Cracking in Concrete Structures," Concrete International: Design and Construction, Vol. 2, No. 10, October 1980, pp. 35-76.

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13. Iliya, R. and Bertero, V. V., "Effects of Amount and Arrangement of Wall Panel Reinforcement on Hystereic Behavior of Reinforced Concrete Walls," Report No. UCB/EERC-80/04, Earthquake Engineering Research Center, University of California, Berkeley, February 1980, 156 pages.

14. ACI Committee 546, "Guide for Repair of Concrete Bridge Superstructures," Concrete International: Design and Construction, Vol. 2, No. 8, September 1980, pp. 69-88.

Attachment 2

James W Cook
Vice President - Projects, Engineering and Construction

General Offices: 1945 West Parnall Road, Jackson, MI 49201 • (517) 788-0453

August 2, 1982

Mr Harold R Denton, Director
Office of Nuclear Reactor Regulation
Att: Division of Licensing
US Nuclear Regulatory Commission
Washington, DC 20555

MIDLAND PROJECT
MIDLAND DOCKET NO 50-329, 50-330
MIDLAND CONCRETE WALL CRACK REPAIR PROGRAM
FILE: 0485.16 SERIAL: 18371

A meeting was held on June 25, 1982 between Consumers Power Company and the NRC to resolve outstanding technical issues regarding the soils remedial actions. During the meeting, Consumers Power Company committed to repair concrete cracks in the category I safety grade buildings affected by the soil fill. The purpose of this letter is to formally document our commitments.

The crack repair program Consumers Power Company has committed to perform is based on the recommendations of our consultants and the concerns expressed by the NRC Staff. It applies to the Diesel Generator Building, Service Water Pump Structure, Control Tower and Electrical Penetrations Areas of the Auxiliary Building and Feedwater Isolation Pits. The Program, which will be completed prior to the first refueling of the plant, consists of the following three points:

1) Repair by epoxy injection any cracks in the structures which are below the permanent ground water table and which exhibit weeping characteristic. This repair will be performed from the inside of the structures.

2) Coat the splash zone of the exterior surface of the south wall of the Service Water Pump Structure which is in contact with cooling pond water with waterproofing compounds. The waterproofing compound will be one of the three compounds recommend by consultants in their report "Effects of Cracks on Serviceability of Structures in the Plant" submitted to the Staff as an enclosure to letter from J W Cook to H R Denton, Serial 16884, Dated April 23, 1982 or equivalent.

3) Repair by epoxy injection existing cracks which are 20 mils and larger and apply a sealant to the surfaces of the concrete walls in the following accessible areas (i.e. areas where removal of soil or installed equipment or installed components is not necessary to perform the repair). The extent (length) of the crack that will be injected with epoxy will be limited to crack widths of 10 mils or larger.

Diesel Generator Building

(a) All accessible interior reinforced concrete walls.

(b) All accessible exterior concrete walls.

Control Tower & Electrical Penetration Areas

(a) All accessible exterior concrete walls.

SWPS

(a) All accessible exterior concrete walls.

Prior to the initiation of repairs, all cracks 20 mils and larger and weeping cracks in the applicable areas will be identified. A verification of this identification to a tolerance of +5 mils will be performed. This verification and subsequent repair will be in accordance with the quality program. The material for structural epoxy adhesive will be "Concresive 1380" manufactured by Adhesive Engineering Company, or equivalent.

James W. Cook

JWC/WJC/mkh

CC Atomic Safety and Licensing Appeal Board, w/o
CBechhoefer, ASLB, w/o
MMCherry, Esq, w/o
FPCowan, ASLB, w/o
RJCook, Midland Resident Inspector, w/o
RSDecker, ASLB, w/o
SGadler, w/o
JHarbour, ASLB, w/o
GHarstead, Harstead Engineering, w/o
DSHood, NRC, w/o (2)
DFJudd, B&W, w/o
JDKane, NRC, w/o
FJKelley, Esq, w/o
RBLandsman, NRC Region III, w/o
WHMarshall, w/o
JPMatra, Naval Surface Weapons Center, w/o
WOtto, Army Corps of Engineers, w/o
WDPaton, Esq, w/o
SJPoulos, Geotechnical Engineers, w/o
FRinaldi, NRC, w/o
HSingh, Army Corps of Engineers, w/o/
BStamiris, w/o

CONSUMERS POWER COMPANY
Midland Units 1 and 2
Docket No 50-329, 50-330

Letter Serial 18371 Dated August 2, 1982

At the request of the Commission and pursuant to the Atomic Energy Act of 1954, and the Energy Reorganization Act of 1974, as amended and the Commission's Rules and Regulations thereunder, Consumers Power Company submits additional information on Midland Concrete Wall Crack Repair Program.

CONSUMERS POWER COMPANY

By JW Cook
J W Cook, Vice President
Projects, Engineering and Construction

Sworn and subscribed before me this 2nd day of August 1982

Beverly A. Avery
Notary Public Beverly A. Avery
Jackson County, Michigan

My Commission Expires Jan 16, 1985

mi0782-0193b100

Soils

3 4/30/82

BCC RCBauman, P-14-312B, w/o
AJBoos, Bechtel, w/a
JEBrunner, M-1079, w/a
WJCloutier, P-24-505, w/a
WGCorley, PCA, w/a
BDhar, Bechtel, w/a
PJGriffin, P-24-513, w/a
EMHughes, Bechtel, w/a
RWHuston, Washington, w/a
JAMooney, P-14-115A, w/a
DBMiller, Midland, w/a
MIMiller, IL&B, w/a
KBRazdan, P-14-419, w/a
JARutgers, Bechtel, w/a
JRSchaub, P-13-309A, w/a
MASozen, BAPC Consultant, w/a
PPSteptoe, IL&B, w/a
TJSullivan/DMBudzik, P-24-624A, w/o
RLTeuteberg, P-24-505, w/a
TRThiruvengadam, P-14-400, w/a
FVillalta, P-14-419, w/a
FCWilliams, IL&B, w/a
NRC Correspondence File

oc0482-0085a100

APPENDIX B

LIMIT ANALYSIS OF SERVICE WATER PUMP STRUCTURE

A.E. Fiorato
W.G. Corley

APPENDIX B

TABLE OF CONTENTS

APPENDIX B

LIMIT ANALYSIS OF SERVICE WATER PUMP STRUCTURE

by

W. G. Corley and A. E. Fiorato*

INTRODUCTION

In the main body of the report entitled "Evaluation of Cracking in Service Water Pump Structure at Midland Plant" (February 1982), cracks observed in the Service Water Pump Structure were described and their significance was evaluated. Observed cracks were primarily attributed to restrained volume changes that occur in concrete during curing and subsequent drying. No evidence of stuctural distress was observed. Although the possibility of settlement related cracking at the intersection of the north overhang with the south portion of the structure could not be completely eliminated, crack patterns did not support the conclusion that settlement was a primary cause of cracking.

As a measure of significance of observed cracks relative to future integrity of the structure, the tensile stress that uncracked concrete may be assumed to carry was compared to available tensile capacity provided by structural reinforcement crossing the cracks. This calculation was made for sections in the vicinity of cracks that had a measured width of 0.010 in. or greater. In the calculation, concrete is assumed to carry a

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principal tensile stress of $4\sqrt{f'_c}$ where f'_c is specified concrete compressive strength.

Based on calculations of tensile capacity, it was determined that available horizontal reinforcement in the east and west walls of the Service Water Pump Structure provided a resistance of approximately 97% of the tensile stress that could be assumed to be attributed to concrete. Resistance provided by vertical reinforcement exceeded by a significant margin the tensile stress assumed to be carried by concrete. It was reasoned that if cracks in these walls had an inclination of at least 15° from vertical, both vertical and horizontal reinforcement would be sufficiently mobilized so that the resultant of forces would exceed the stress attributed to concrete tensile strength. It was therefore concluded that resistance provided by the reinforcement was sufficient.

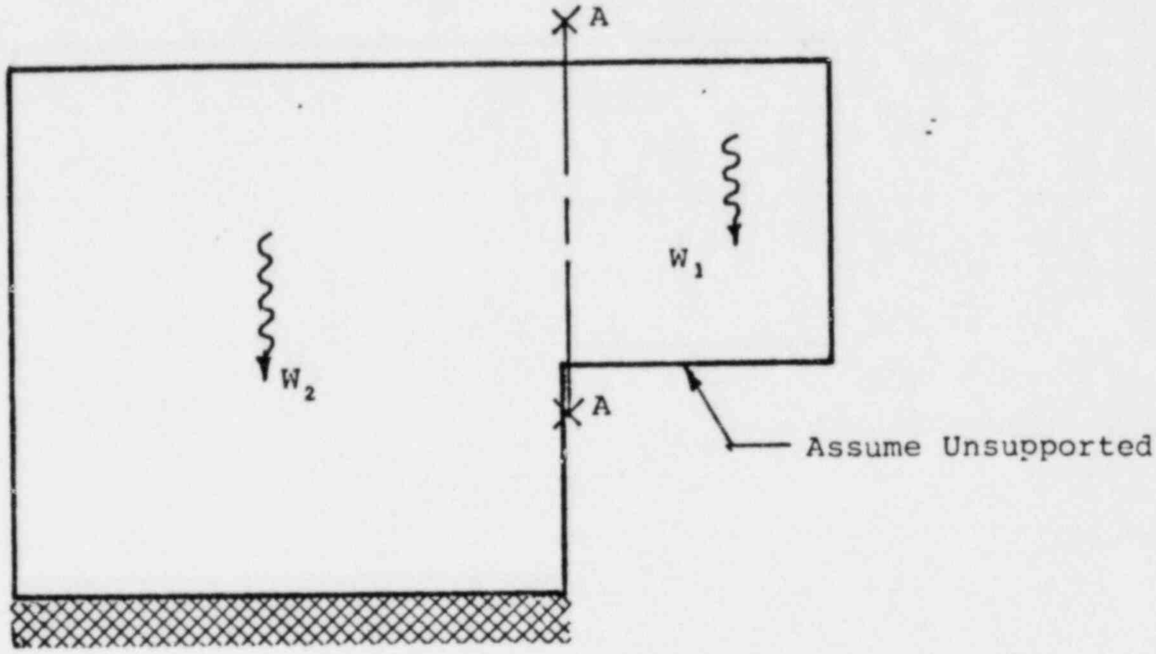
Nuclear Regulatory Commission staff members reviewed the report entitled "Evaluation of Cracking in Service Water Pump Structure at Midland Plant." After review, staff members requested that a more detailed analysis be made to evaluate the east and west walls of the Service Water Pump Structure. Therefore a limit analysis of the Service Water Pump Structure was undertaken. This analysis includes consideration of interior walls as well as east and west exterior walls. The objective of this Appendix is to describe the approach used for the limit analysis and the results obtained. Although it is not feasible to predict every limit case, limit analysis cases described in the following sections serve as examples of the inherent strength of the structure.

METHODOLOGY

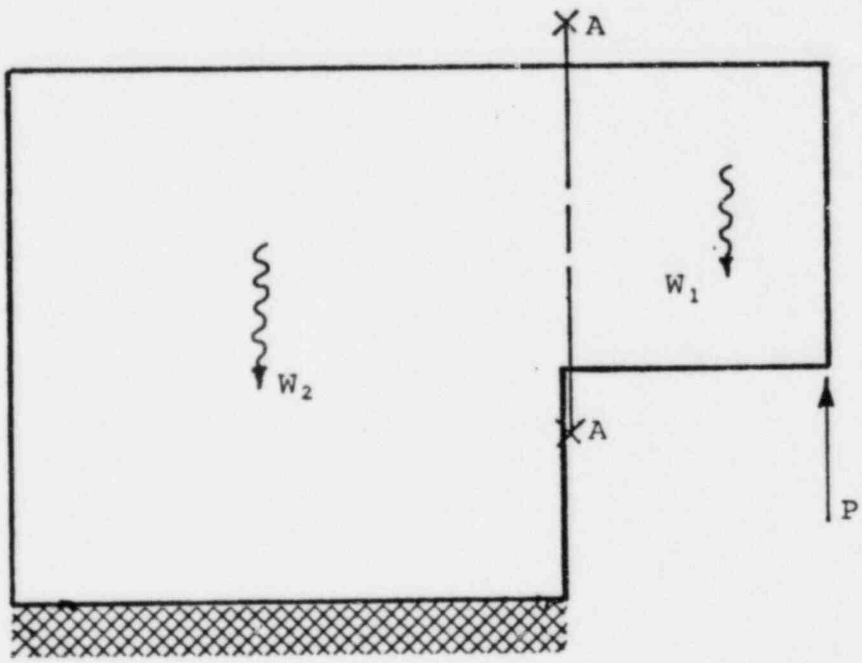
The basic approach used in the limit analysis of the Service Water Pump Structure was to determine if forces that can be induced in the structure are sufficient to exceed capacity of walls assuming the existence of cracks. In-plane shear capacity of cracked walls was of primary concern in the analyses. Capacities were calculated using representative material properties, and section geometries determined from drawings provided by Bechtel.* All walls crossing sections analyzed were considered to contribute to bending and shear resistance. Contributions of exterior and interior walls were calculated assuming the structure to act as a unit. Reinforcement details were checked to insure that available development lengths were adequate.

Figures B-1 and B-2 illustrate hypothesized loading conditions that would induce critical vertical and horizontal forces on wall sections of the Service Water Pump Structure. In Case 1, shown in Fig. B-1, the entire north overhang of the structure was assumed to be unsupported. Thus, the weight of the north overhang would induce shear and moment forces at Section A-A. As shown in Figs. 10 and 11 in the main body of this report, east and west walls of the Service Water Pump Structure contain cracks at locations corresponding to Section A-A in Fig. B-1. The assumption of complete lack of support for the north end of the structure is extremely conservative, but it provides an estimate of maximum shears and moments that could be induced at Section A-A.

*Drawings used for analysis of the Service Water Pump Structure are referenced in Table 2 of the main body of this report.



(a) Case 1



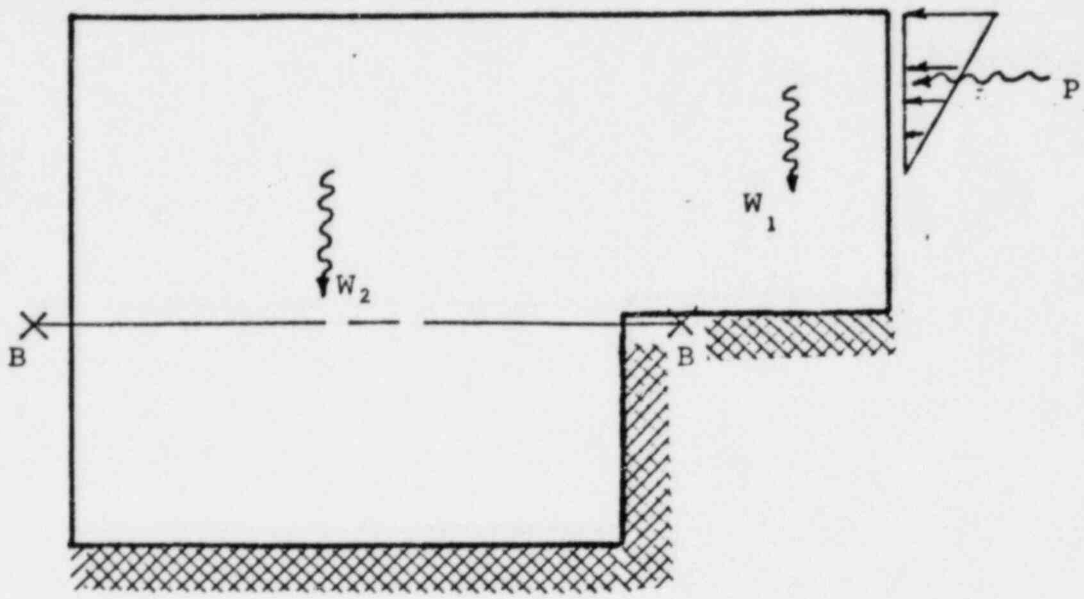
(b) Case 2

Fig. B-1 Hypothesized Loading Conditions for Critical Vertical Forces on Service Water Pump Structure

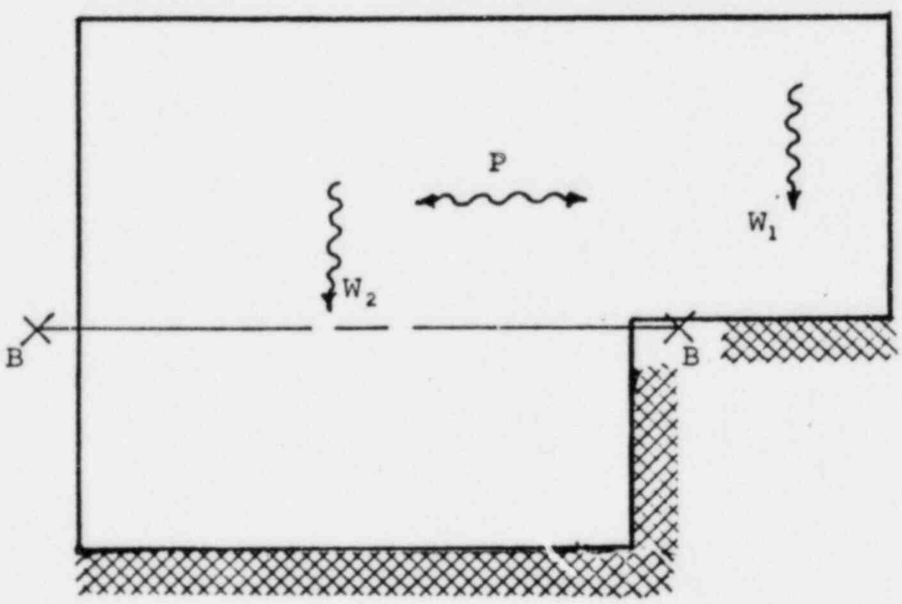
The second case considered for determination of maximum vertical forces that could be induced in the Service Water Pump Structure walls is shown in Fig. B-1, Case 2. This case assumes that a vertical force P is applied at the extreme north end of the structure in the upward direction. Such a force could be assumed to occur if control of jacking operations during underpinning is lost. Case 2 was evaluated by calculating the force P that would be required to induce yielding at Section A-A or that would overturn the structure.

In addition to evaluating critical sections for vertical shear forces, two cases were considered for transfer of horizontal shear forces in walls of the Service Water Pump Structure. These are illustrated in Fig. B-2. Case 3 assumes a horizontal load applied to the north wall of the Service Water Pump Structure. This force induces moments and shears at Section B-B. The limit on magnitude of this horizontal force was determined by evaluating shear and moment capacities at Section B-B, and also by considering rigid body movement of the structure.

The final hypothesized loading condition that was analyzed is shown in Fig. B-2, Case 4. This condition considers horizontal forces induced in walls of the Service Water Pump Structure as a result of seismic motion. If such a force is developed, it would be necessary to transfer horizontal shear through Section B-B. This section was analyzed by considering shear capacity at Section B-B, and also by evaluating the magnitude of forces that can reasonably be expected as a result of ground accelerations.



(c) Case 3



(d) Case 4

Fig. B-2 Hypothesized Loading Conditions for Critical Horizontal Forces on Service Water Pump Structure

RESULTS OF ANALYSES

This section presents results of limit analyses made for hypothesized loading conditions shown in Figs. B-1 and B-2.*

Case 1 - North End of Structure Unsupported

If it is assumed that the entire north overhang of the structure is unsupported, the deadweight W_1 shown in Fig. B-1 will induce moment and shear at Section A-A. Calculated dead weight of the north end of the structure, excluding equipment weight, would induce a nominal vertical shear stress of approximately 130 psi on walls of the structure at Section A-A. Shear resistance at Section A-A was calculated by shear friction theory in accordance with Section 11.7 of American Concrete Institute Building Code Requirements for Reinforced Concrete (ACI 318-77). Based on shear friction analysis, the nominal shear stress that can be resisted at Section A-A is approximately 275 psi. Shear friction analysis assumes the presence of a crack at the section being evaluated.

The moment at Section A-A induced by dead weight W_1 was calculated to be approximately 50% of the yield moment at that section. Thus, if the north overhang of the Service Water Pump Structure were completely unsupported, reinforcement would not yield in flexure nor would nominal shear capacity be exceeded.

*Analyses described in this report did not include evaluation of potential foundation failures. However, if foundation failures were to limit capacity at lower forces than calculated for cases considered, nominal stresses in the walls would be even lower.

Case 2 - Upward Force on North End of Structure

For Case 2 it was hypothesized that a concentrated force (line load) was induced along the north wall of the Service Water Pump Structure. Calculations were made to determine potential limits on the magnitude of this force.

The upward force shown in Fig. B-1 for Case 2 can increase until either shear or moment capacity at Section A-A is exceeded or until the structure uplifts. Calculations indicate that uplift of the structure would be the limiting criterion. The load P that would lift the structure would result in a nominal shear stress of approximately 150 psi in walls at Section A-A. Immediately adjacent to the north wall of the structure, the nominal shear stress would be approximately 275 psi. Calculated shear friction capacity of a vertical section through the walls in the north end of the Service Water Pump Structure corresponds to a nominal shear stress of approximately 275 psi. Therefore, shear capacity is adequate. The moment at Section A-A corresponding to uplift of the structure was calculated to be approximately 50% of the yield moment.

Case 3 - Horizontal Load on North Wall

Case 3, illustrated in Fig. B-2, assumes the presence of a horizontal force on the north wall of the Service Water Pump Structure. Limits on the magnitude of this force were estimated by considering moment and shear resistance at Section B-B as well as potential rigid body movement of the structure. The horizontal force assumed for Case 3 was applied at 2/3 the height of the north wall above top of grade. This corresponds to a triangular force distribution.

For the assumed loading condition, the force that would overturn the structure would induce a nominal shear stress of approximately 225 psi at Section B-B. Analysis of shear resistance at Section B-B using shear friction theory indicates a shear capacity of approximately 310 psi. Thus the structure would tend to overturn before shear friction resistance was exceeded.

Assumptions made for Case 3 are unrealistically conservative. It is not possible to develop a force of the magnitude required to either overturn the structure or exceed shear friction capacity. The force required to overturn the structure corresponds to a uniform pressure of approximately 60 psi on the north wall. Design tornado wind load for the structure corresponds to approximately 2.3 psi.

If the horizontal force is assumed to act on the south wall, an even larger pressure is required to reach shear friction capacity of the vertical walls.

Case 4 - Horizontal Inertial Forces

The final hypothesis that was considered is Case 4 illustrated in Fig. B-2. Case 4 considers development of inertial forces that could potentially induce critical shear stresses in walls of the Service Water Pump Structure. Calculations for this condition indicate that horizontal forces large enough to exceed the shear friction resistance along Section B-B are equivalent to the deadweight of the structure excluding equipment. To generate such forces, a seismic event well in excess of what can reasonably be expected would be required.

SUMMARY AND CONCLUSIONS

This report presents a summary of limit analyses made to evaluate capacity of walls of the Service Water Pump Structure at Midland Nuclear Power Plant Units 1 and 2. Analyses were made for various hypothesized loading conditions to demonstrate inherent strength of the structure. Wall capacities were estimated assuming cracked sections. Results indicate that the walls have sufficient in-plane shear capacity to resist hypothesized limiting forces. For all cases evaluated, it was determined that shear capacity of walls in the Service Water Pump Structure would not limit capability of the structure to resist in-plane forces.