J.O. No.

Consumer Power Company
Midland Plant Unit 1 & 2
Independent Assessment

Project Procedure 1.0

Issue Date: September 20, 1982

PROJECT QUALITY ASSURANCE PLAN

Approvals:

Neeri T. Georgan for ASL
Project Manager

Chief Engineer
Engineering Assurance

Richard B Kelly
Manager
Quality Assurance

SCOPE

This procedure describes the quality assurance plan for activities performed by Stone & Webster Engineering Corporation (SWEC) and its subcontractors in the soils remedial construction independent assessment of auxiliary building underpinning for the Consumers Power Company's Midland Plant - Unit One and Two. The work involved in this independent assessment shall be accomplished in the following manner:

a. Overview of the design and construction documents to gain familiarity with the work.

b. Assessment of construction and related quality activities for compliance with plans and specifications for the work. This will be accomplished through surveillance of construction and quality control activities.

c. Daily reviews presenting CP Co with any discovered noncompliances.

d. Submittal of nonconformance reports to the NRC (without prior CP Co review) with a copy to CP Co.

e. Submittal of a weekly progress report and a final report to the NRC (without prior CP Co review) with a copy to CP Co.

f. The final report shall be overviewed by senior level Stone & Webster management.

PROGRAM REQUIREMENTS AND ACTIVITIES

I. ORGANIZATION

The overall SWEC organization is depicted in SWSQPA 1-74A (Section I). The project organization is described in Attachment 1 to this plan.

II. QUALITY ASSURANCE PROGRAM

The overall SWEC quality assurance program is designed to provide assurance that all SWEC activities are accomplished in a controlled manner. The SWEC corporate QA program complies with 10CFR50, Appendix B, and NRC Regulatory Guides, and is described in an NRC approved topical report, SWSQAP 1-74A, "Standard Nuclear Quality Assurance Program."

This quality assurance plan shall be maintained up-to-date to reflect any changes in the scope of SWEC work.

This quality assurance plan identifies the procedures which implement the overall QA program as it applies to the SWEC scope. Insofar as possible, applicable standard SWEC procedures are used to govern the work. When standard procedures do not fit project circumstances, project procedures are issued to govern the work. Variances from standard SWEC procedures are approved according to Quality Standards (QS) 5.1 and Engineering Assurance Procedure (EAP) 5.7.

Personnel performing activities in accordance with this plan requiring qualification and certification are qualified and certified in accordance with Quality Standard 2.12 and Quality Assurance Directive 2.5.

III. DESIGN CONTROL

(Not within the SWEC scope)

IV. PROCUREMENT DOCUMENT CONTROL

Consulting Services are procured in accordance with Engineering Assurance Procedures 4.1 and 4.15.

V. INSTRUCTIONS, PROCEDURES, AND DRAWINGS

Project procedures, including variances, are prepared and controlled in accordance with Section II of this QA plan.

(Instructions, drawings and specifications are not within the SWEC scope).

VI. DOCUMENT CONTROL

(Not within SWEC scope)

VII. CONTROL OF PURCHASED MATERIAL, PARTS, EQUIPMENT, AND SERVICES

(Control of Purchased Material, Parts and Equipment - not within the SWEC scope).

Control of Services is in accordance with Engineering Assurance Procedure 7.1.

VIII. IDENTIFICATION AND CONTROL OF MATERIAL, PARTS, AND COMPONENTS

(Not within SWEC scope)

IX. CONTROL OF SPECIAL PROCESS

(Not within SWEC scope)

X. INSPECTION

Quality Assurance monitoring of the construction and quality activities is performed by surveillance of on-going work.

IX. TEST CONTROL

(Not within the SWEC scope)

XII. CONTROL OF MEASURING AND TEST EQUIPMENT

(Not within the SWEC scope)

XIII. HANDLING, STORAGE, AND SHIPPING

(Not within the SWEC scope)

XIV. INSPECTION, TEST, AND OPERATING STATUS

(Not within the SWEC scope)

XV. NONCONFORMING MATERIALS, PARTS, OR COMPONENTS

Nonconformances discovered by SWEC during the monitoring process are reported in writing to NRC with copy to CP Co. Nonconformances identified by subcontractors are reported in accordance with the procurement document.

XVI. CORRECTIVE ACTION

Reporting under 10CFR50.55(e) is accomplished in accordance with QS-16.2 and EAP-16.2.

Reporting under 10CFR21 is accomplished in accordance with QS-16.3 and EAP-16.3.

XVII. QUALITY ASSURANCE RECORDS

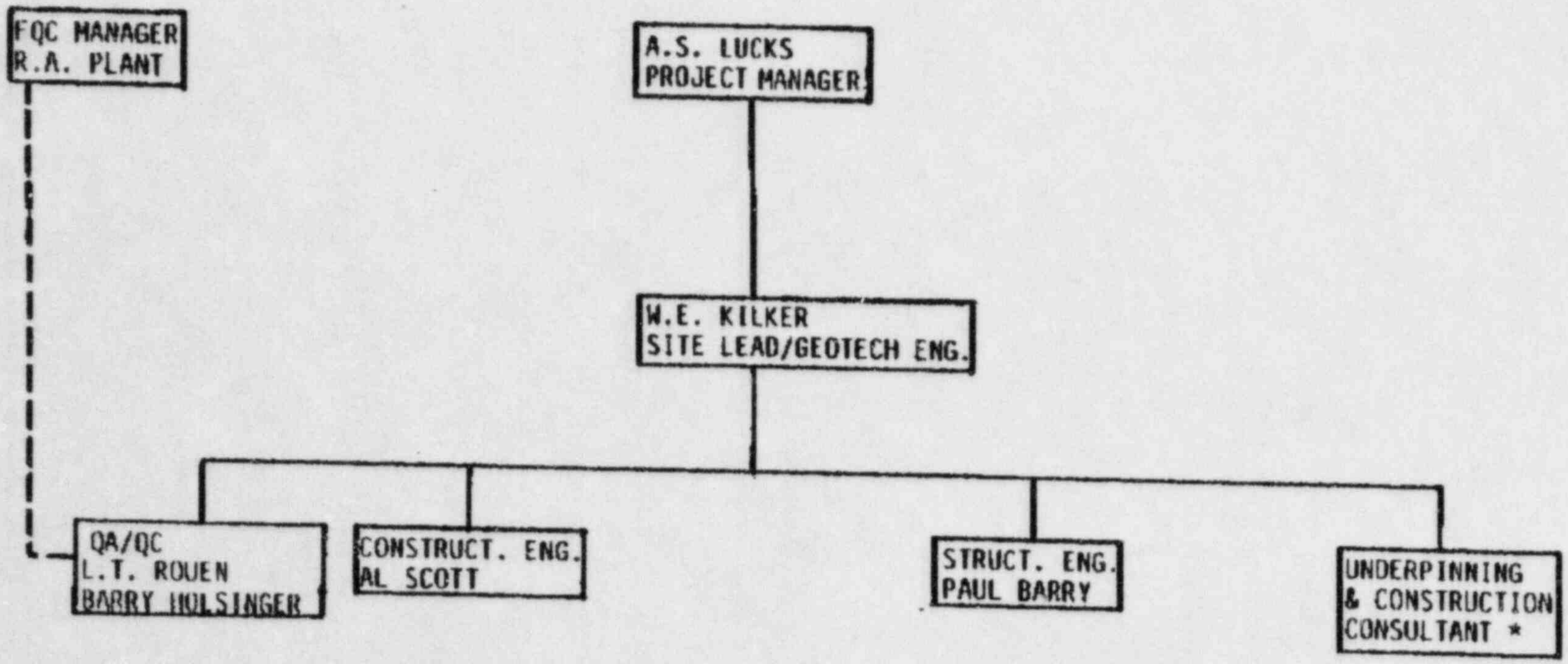
SWEC General Policy and Procedure for records collection, retention, and turnover to Consumers Power Company are described in QS-17.1 and EAP-17.2 and as detailed in the scope.

XVIII. AUDITS

(Not within SWEC scope)

Attachment 1
Page 1 of 1

MIDLAND UNIT 1 & 2 PROJECT ORGANIZATION



*PARSONS, BRINCKERHOFF, QUADE & DOUGLAS

DETAILED EXPERIENCE RECORD
ROUEN, LAWRENCE T. 78274

STONE & WEBSTER ENGINEERING CORPORATION, BOSTON, MA (Sep 1973 to Present)

Appointments:

Senior Quality Assurance Engineer - Mar 1982
Quality Assurance Engineer - May 1979
Quality Control Engineer - Oct 1975
Assistant Quality Control Engineer - Sep 1973

Clinch River Breeder Reactor Plant, US Dept of Energy (Jun 1981 to present)

As SENIOR QUALITY ASSURANCE ENGINEER, responsible for performance of ASME Section III, Division 2, Level III functions such as approvals of test and inspection procedures and development of training and certification programs for concrete inspectors, plus the duties performed as Quality Assurance Engineer.

As QUALITY ASSURANCE ENGINEER, responsible for quality review and input to technical documents, procedure review and development and the preparation/review of QA inspection plans.

River Bend Station, Gulf States Utilities Co. (May 1979-Jun 1981)

As QUALITY ASSURANCE ENGINEER, responsible for review and approval of QA inspection plans, vendor QA manuals, specification changes, and nonconformance dispositions. Also performed surveillance and audit activities to assure functional and programmatic compliance with project, corporate, client, and regulatory requirements.

Millstone Unit III, Northeast Utilities Service Co. (May 1978-May 1979)

As QUALITY CONTROL ENGINEER, responsible for supervision of the inspection programs for concrete, structural steel, protective coatings, and earthwork.

River Bend Station, Gulf States Utilities Co. (Oct 1975-May 1978)

As QUALITY CONTROL ENGINEER, responsible for field and laboratory testing of concrete and soils.

Shoreham Nuclear Power Station, Long Island Lighting Co. (Sep 1973-Oct 1975)

As ASSISTANT QUALITY CONTROL ENGINEER, tested and/or inspected concrete, cadwelds, soils, and aggregate.

DEPARTMENT OF PUBLIC WORKS, KANSAS CITY, MO (May 1973-Aug 1973 & May 1972-Aug 1972)

As ENGINEERING AIDE, supervised road repair and kept force accounts of work performed.

March 1982

ROUEN, LAWRENCE T.

SENIOR QUALITY ASSURANCE ENGINEER
QUALITY ASSURANCE DEPARTMENT

EDUCATION

University of Missouri (Columbia) - Bachelor of Science in Civil Engineering 1973
Various SWEC Continuing Education Courses

LICENSES AND REGISTRATIONS

Professional Engineer in Civil Engineering - Louisiana
ACI/ASME Level III Inspection Engineer

EXPERIENCE SUMMARY

Mr. Rouen has eight years' experience in the nuclear power plant construction industry. Currently, as Senior Quality Assurance Engineer, he is responsible for quality review and input to technical documents, procedure review and development, and the preparation/review of QA inspection plans. He is also responsible for the development of training programs for Level I and II ASME concrete inspectors and for certification of those inspectors.

Since joining Stone & Webster in 1973, he has gained in-depth experience in testing and inspection of structural activities and in overall quality assurance functions on several nuclear power plant projects.

PROFESSIONAL AFFILIATIONS

None

PUBLICATIONS

None

DETAILED EXPERIENCE RECORD
LUCKS, A. STANLEY 54576

STONE & WEBSTER ENGINEERING CORPORATION, BOSTON, MA (June 1972 to Present)

Appointments:

Chief Geotechnical Engineer - Apr 1978
Assistant Chief Geotechnical - Nov 1976
Group Supervisor and Senior Soils Engineer - Nov 1973
Soils Engineer - June 1972

Geotechnical Division Staff (Nov 1973 to Present)

As CHIEF GEOTECHNICAL ENGINEER (Apr 1978 to Present), responsible for managing the Geotechnical Division staff and facilities. Division responsibilities include geology, seismology, soil mechanics, rock mechanics, foundation engineering, embankment dams, underground facilities, and groundwater hydrology. Geotechnical staff involved in fossil, nuclear, and hydroelectric power projects and studies for advanced technologies, including nuclear waste disposal and energy storage. He directed a feasibility study for a 13 MW high head hydroelectric project located above the Arctic Circle. The project conceptual design called for power tunnels and shafts in permanently frozen rock. Division facilities include a 3,000-sf Geotechnical Testing Laboratory and an extensive computer program library.

As ASSISTANT CHIEF GEOTECHNICAL ENGINEER (Nov 1976-Apr 1978), responsible to the Chief Geotechnical Engineer for control of geotechnical engineering work conducted by Division staff and control of the Geotechnical Testing Laboratory. Served as Division Licensing Representative for the review and approval of Safety Analysis Reports for nuclear power projects.

As GROUP SUPERVISOR (Nov 1973-Nov 1976), was responsible for the supervision of:

The geotechnical design and preparation of specifications for the construction of Rock Island Second Powerhouse. Work included cellular and embankment type cofferdams and controlled blasting for powerhouse excavation, grouting, and earthwork.

Geotechnical work for North Anna Nuclear Power Station, Units 1, 2, 3, and 4. Work included liquefaction studies, design, and construction of drilled caissons, design and installation of dewatering systems.

Geotechnical work for Beaver Valley Nuclear Power Station - Unit No. 2. Work included in situ densification using compaction piles.

Preparation of excavation, backfill and cofferdam specifications, and bid evaluation for Millstone Nuclear Power Station construction.

Geologic work included the mapping of excavations and fault investigations.

Siting Study, New York Generation Study Group (Aug 1973-Sept 1973)

As LEAD GEOTECHNICAL ENGINEER, responsible for geotechnical evaluation of several potential nuclear power plant sites in New York State.

Canal Site, Philadelphia Electric Company (June 1972-Mar 1973)

As SOILS ENGINEER, assisted in site investigation for nuclear power station including preparation of soils report.

Turners Falls Project, Northeast Utilities Service Company, (July 1972-Dec 1972)

As LEAD GEOTECHNICAL ENGINEER, responsible for geotechnical aspects of FPC safety inspection of Turners Falls power canal, dams, and generating stations.

Montague Nuclear Power Station, Northeast Utilities Service Company, (Feb 1973-Sept 1974)

As LEAD GEOTECHNICAL ENGINEER, carried out studies for site selection and investigation for nuclear power plant. Preparation of geotechnical section of PSAR. Work included extensive structural geologic investigation.

Effingham Unit 1, Savannah Electric Power Company (Oct 1972-June 1973)

As LEAD GEOTECHNICAL ENGINEER, prepared of soils report and foundation evaluation for Effingham Unit 1, a fossil fuel power plant. Work included evaluation of bids for piles and setting up pile driving inspection program, design of intake and discharge structures, and unloading dock.

Soils investigation and recommendation of foundation design parameters for 72 miles of transmission lines. Conducted pile load tests to confirm foundation design parameters.

LAMBE ASSOCIATES INC., CONCORD, MA (May 1970-May 1972)

University of Massachusetts, Columbia Point Campus, Boston Bureau of Building Construction

As PROJECT SOILS ENGINEER, author of report on settlement of buildings constructed on a sanitary landfill. Contributed to a report on the problem associated with the generation of methane within sanitary landfills and the design of gas protection systems.

Piney Point Reservoir, Borden Chemical Company, Plant City, FA

As PROJECT SOILS ENGINEER, inspected gypsum tailing dams and phosphatic slime settling ponds. Prepared reports giving design recommendations for new embankments and the maintenance and repair of existing embankments.

Parque Central, Delpre C.A. Caracas, Venezuela

As PROJECT SOILS ENGINEER, participated in the design of earth retaining structures for deep excavation for the construction of the foundations for high-rise buildings in central Caracas. Inspected construction and monitored the performance of slurry trench and sheet pile anchored walls.

140 Federal Street Building, Employers - Commercial Union Insurance Company Boston MA.

As PROJECT SOILS ENGINEER, evaluated a bracing scheme for a deep excavation and determined cause of movements and damage to adjacent structures.

Fort Mead Mine, Cities Service Company, Tampa, FA

Evaluated the stability of four phosphatic slime-settling ponds and investigated the cause of a major failure of an embankment. Designed instrumentation system for critical areas. Author of two reports giving results of evaluation and recommendations for repair.

Freetown SNG Plant, Algonquin Gas Transmission Company, Boston, MA

As PROJECT SOILS ENGINEER, responsible for the site investigation for an SNG plant. Work included an on-site water storage reservoir, two 200,000-barrel naphtha storage tanks, and a barge off-loading facility in addition to the process plant foundations. Author of three reports giving results of site investigation and making recommendations for foundation design and reservoir siting.

Angra Dos Ries Nuclear Power Plant, Furnas Centrals Electrica, Brazil

As SOILS ENGINEER, prepared technical specifications for construction of diaphragm cutoff wall, dewatering, excavation, backfill, and compaction for nuclear power plant.

T. WILLIAM LAMBE & ASSOCIATES, CAMBRIDGE, MA (June 1967-May 1970)

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As CONSULTING SOILS ENGINEER, assisted with design of oil storage reservoir (FORS-3). Performed deformation and stability analyses for embankment and abutment.

Responsible for survey of condition of unstable cliffside and co-author of report making recommendations for stabilizing critical sections where refinery structures were threatened. Design surveillance instrumentation for cliffside.

Designed horizontal drainage system for stabilizing cliffside. Prepared specification for installation of drains.

Participated in surveillance program to monitor performance of earth structures within the refinery including three oil storage reservoirs. Instrumentation included piezometers, inclinometers, strain meters, load cells, temperature sensors, and settlement platforms. A remote data acquisition system was used for data collection.

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Clunas Reservoir, Nairn Joint Water Board, Scotland

As PROJECT ENGINEER, responsible for initial design of reservoir system including hydrology, hydraulics, and geotechnical aspects. Earth dam required 100-ft deep alluvial grouted cutoff. Supervised design of hydraulic model of spillway and soil testing for embankment.

Adam Bridge, Marquis of Bute Estates, Scotland

As PROJECT ENGINEER, author of report on evaluation and recommendations for the repair of the Adam Bridge built in 1740 and subsequently damaged by mining subsidence.

Creightons Green Reservoir, Holywood Water Board, Northern Ireland

As PROJECT ENGINEER, responsible for investigation of leakage through earth dam. Prepared a report giving details of remedial work required.

Black Esk Reservoir, Dumfries County Council, Scotland

As CIVIL ENGINEER, responsible for maintenance and data collection and handling for soil instrumentation installed in earth dam.

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As ASSISTANT CIVIL ENGINEER, assisted with installation and responsible for data processing and evaluation for soil instrumentation in earth dam.

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As ASSISTANT CIVIL ENGINEER, assisted with preparation of hydraulic calculations for claim due to flooding of power station. Conducted inspection of 54 in dia steel pipeline.

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As ASSISTANT FIELD ENGINEER on the construction of the approaches to the Firth of Forth suspension bridge, Inspected construction of reinforced concrete bridges and calculated payment quantities.

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Lochan Breaclaich Hydroelectric Scheme

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Holy Loch Submarine Base, U.S. Navy, Holy Loch, Scotland

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WHATLINGS LTD., GLASGOW, SCOTLAND (Apr 1962-Oct 1962)

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As ASSISTANT FIELD ENGINEER on the construction of two 600-ft transmission towers, responsible for inspection of foundation installation. Foundations included the driving of 120 ft long precast concrete piles. In charge of survey work for transmission line.

March 1982

KILKER, WAYNE E.

SENIOR SOILS ENGINEER
GEOTECHNICAL DIVISION

EDUCATION

University of Minnesota, Minneapolis - B.S. in Civil Engineering 1964
Arizona State University, Tempe, Arizona - M.S. in Soil Mechanics 1970

LICENSES AND REGISTRATIONS

Professional Engineer - Minnesota and Maine

EXPERIENCE SUMMARY

Mr. Kilker, a Senior Soils Engineer in the Geotechnical Division, joined Stone & Webster Engineering Corporation (SWEC) in June 1974. He has over 17 years' experience in Civil Engineering including an advanced degree in soil mechanics, and over 12 years of this experience are in soil and foundation engineering.

As Project Geotechnical Engineer on several coal-fired power plants, Mr. Kilker has been responsible for site investigations, foundation design and coal storage, runoff pond, and ash disposal area design parameters.

As a Lead Geotechnical Engineer for two nuclear power plant sites at Long Island, New York, his responsibilities included supervision of soil engineers in performing site studies, field liaison work, laboratory testing, specification preparation, analytical studies, and report writing. Mr. Kilker was Principal Soils Engineer in charge of evaluating soil conditions and preparing reports on a nuclear power plant site along the Hudson River in New York State.

Mr. Kilker has also performed siting studies for nuclear and hydroelectric facilities, as well as proposal preparation for nuclear/fossil power plants and industrial facilities. He was responsible for modifying specifications for drilling and logging of test holes in deep-seated salt formations.

His experience includes geotechnical work for airports, highways, pipelines, building excavations, retaining structures, and offshore installations. He has evaluated excavation and backfilling operations, piling and drilled pier installations, in situ densification, sand drain placement and groundwater pump tests.

Prior to joining SWEC, Mr. Kilker was also responsible for special investigations such as structural damage claims due to soil conditions, vibrational and impact sources, and fire-related structural distress. He has performed as an expert witness in court, defending several of these studies.

PROFESSIONAL AFFILIATIONS

American Society of Civil Engineers
Boston Society of Civil Engineers
International Society for Soil Mechanics and Foundation Engineering

PUBLICATIONS

"Effect of Change in Effective Stress on SPT N-Values," International Conference on Soil Mechanics and Foundation Engineering, Stockholm, 1981.

DETAILED EXPERIENCE RECORD
KILKER, WAYNE E. 48903

STONE & WEBSTER ENGINEERING CORPORATION, BOSTON, MA (June 1974 to Present)

Appointments:

Senior Soils Engineer - June 1978
Soils Engineer - Sept 1975
Engineer (Soils) - June 1974

MacInnes Power Station, Tampa Electric Company (Oct 1981 to Present)

As PROJECT GEOTECHNICAL ENGINEER, responsible for development of site geotechnical investigation.

Permian Basin Project, Office of Nuclear Waste Isolation (Aug 1981-Sept 1981)

Tailored drilling and logging specifications for use in deep hole studies of salt formations.

Beaver Valley Power Station - Unit 2, Duquesne Light Company (Dec 1981)

Inspection of pile installation for office building.

Indian Point Units Nos. 2 and 3, Consolidated Edison Company (Sept 1981 to Present)

As PROJECT GEOTECHNICAL ENGINEER, responsible for geotechnical input to alternate conceptual designs and cost estimates of angled fish screens.

Malakoff Lignite Generating Station, Houston Light and Power Company (Mar 1981-May 1981)

Responsible for settlement prediction calculation for main plant structures.

Patriot Station, Indianapolis Power and Light Company (Jan 1981-Apr 1981)

Performed pile foundation study including evaluation of pile capacity, pile quantities, and engineering order-of-magnitude cost estimate. Prepared a pile load test program and production pile procurement and installation specification.

Millinocket Mill Coal Utilization Project, Great Northern Paper Company (Apr 1980-Sept 1981)

As PROJECT GEOTECHNICAL ENGINEER, responsible for site geotechnical investigation report and geotechnical design criteria. Evaluated site conditions for foundations, coal storage, coal pile runoff pond, and ash disposal areas. Prepared settlement and lateral earth pressure calculations and assisted in off-site development studies.

Butner, North Carolina Site, Vevey Engineering Works (Mar 1980-July 1980)

Prepared an Engineering Scope of Work for the site geotechnical investigation and coordinated with consultant's activities throughout the work.

AFB Pilot plant, Tennessee Valley Authority, Padukah, Kentucky (Feb 1980-June 1980)

Prepared site development plan specification. Estimated earthwork, roadway, and drainage quantities.

Sears Island Fossil Power Plant Site, Central Maine Power Company (Dec 1979-Jan 1980)

Performed site reconnaissance, prepared conceptual plan, and estimated quantities for development of an off-site ash disposal area.

Patriot Station Site, Indianapolis Power and Light Company (Dec 1979-Apr 1980)

Performed slope stability analysis. Prepared a conceptual design of a braced sheet pile/tieback excavation for a pump house and intake pipe installation. Assisted in preparation of the geotechnical report on the riverfront area of the site.

Mason Station, Central Maine Power Company (Nov 1979-Dec 1979)

Reconaissance of several possible ash disposal sites. Assisted in preparation of site layout and quantity estimates. Responsible for structural evaluation of existing wooden docking facilities.

Yugoslavian Siting Proposals, Departments of Croatia & Slovenia (Sept 1979-Nov 1979)

Responsible for the preparation of the geotechnical portion of reports describing the methodology of siting nuclear power plants as well as presentations to the client.

Site Evaluation Study, Salt River Project, Arizona (Oct 1978-Feb 1979)

PRINCIPAL ENGINEER in charge of evaluating geotechnical characteristics of various sites being considered for pumped storage hydroelectric facility.

Jamesport Nuclear Power Station, Long Island Lighting Company, Long Island, NY (Jan 1977-Dec 1979)

As LEAD GEOTECHNICAL ENGINEER, responsible for the preparation of geotechnical design criteria, groundwater studies, and preparation of specifications. Directed pumping-recharge well test and groundwater cutoff wall feasibility boring investigation. Responsible for design of pumping-injection wall field, groundwater cutoff wall, and deep well-wellpoint dewatering system.

Shoreham Nuclear Power Station. Long Island Lighting Company, Long Island, NY (Mar 1976 to Present)

As LEAD GEOTECHNICAL ENGINEER, responsible for all geotechnical related site activities. Directed field investigations, liquefaction and stability analysis, settlement evaluation, and intake structure stability analysis. Reviewed offshore pipeline installation procedures and inspected backfill operations. Prepared chemical grout and Vibroflotation installation specifications. Responsible for coordination of in situ densification program and verification of adequacy of the installation. Prepared recommendations for intake canal erosion protection by filter cloth and stone. Evaluated quarry stone proposed for canal slope protection. Observed armor and bedding stone installations. Designated subgrade preparation procedures for plant roads.

Montezuma Pumped Storage, Salt River Project, Arizona (Dec 1975-Feb 1976)

Performed economic study of alternate schemes for reservoir, shaft, and cavern size and location. Used finite element technique to study stress-deformation characteristics of rock due to underground cavern construction.

Mushare - Darkuvin Sites, Atomic Energy Organization of Iran Khuzistan, Iran (Oct 1975-Nov 1975)

Test boring and layout survey specification review. Directed boring layout and site boring program.

Rijkswaterstat, Deltadienst - Holland (July 1975-Sept 1975)

Performed stability study using finite element technique for granular soils underlying proposed concrete box caissons subject to repeated wave action.

North Anna Power Station, Virginia Electric and Power Company, Mineral, VA (June 1975)

Directed backfill placement study, optimizing degree of compaction and placement time.

Fossil and Nuclear Plant Sites - New York Station, Power Authority of the State of New York (July 1974-June 1975)

Authored sections of Public Service Commission and Preliminary Safety Analysis Reports describing site soil conditions and the relationship of these soils to the proposed structures. Authored scope of work outlining required laboratory testing of site soil and rock. Managed off-site borrow study for procurement of granular fill. Performed relative cost analysis for alternate foundation schemes. Performed slope stability and settlement analyses of on-site soils. Supervised preparation of soil-cement installation specification.

Charlestown Nuclear Power Plant - Rhode Island, New England Power Company (Aug 1974-Oct 1974)

Prepared test procedure and interpreted field permeability test results. Evaluated sheetpiling feasibility. Prepared specification for installation and test pumping of water well.

TWIN CITY TESTING & ENGINEERING LABORATORIES, ST. PAUL, MN (July 1970-June 1974)

Responsible for geotechnical design criteria for installations such as footings, mats, deep foundations, retaining walls, embankments, airports, and roadways. Inspected excavation and compaction operations, drilled pier caissons, piling, in situ densification, and soil borings. Investigated structural damage claims due to vibrating equipment or blasting. Monitored vibrations. Investigated soils related damage claims such as settlement, floor, and wall failures. Tested structural units such as column forms and airplane wings for certification.

ARIZONA STATE UNIVERSITY, TEMPE, AZ (Jan 1969-June 1970)

As GRADUATE ASSISTANT supervised laboratory sections of soil mechanics and structural mechanics classes. Advised students on course content and problems.

PEACE CORPS, COLOMBIA, SOUTH AMERICA (June 1966-Sept 1968)

Taught the engineering laboratories at a technical university. Planned and directed construction of engineering projects such as secondary roads and simple span bridges in rural Colombia. Received formal training in Spanish language and employed Spanish in daily communications.

ARIZONA HIGHWAY DEPARTMENT, PHOENIX, AZ (Jan 1965-June 1966)

ASSISTANT ENGINEER in highway materials group. Inspected embankment soil placement during construction of a section of Interstate Highway.

TWIN CITY TESTING AND ENGINEERING LABORATORIES, ST. PAUL, MN (Sept 1963-Dec 1964)

Part time

Inspected compacted fills for roadways and foundations. Performed laboratory tests on soils and various construction materials.

December 1981

BARRY, PAUL FRANCIS 03675

STRUCTURAL ENGINEER
STRUCTURAL DIVISION

Education - Graduate of Tufts University with a B.S. Degree (Cum Laude) in Civil Engineering - 1971

Registrations - Professional Engineer - Massachusetts and Indiana

Experience Summary

Mr. Barry is presently assigned as Lead Structural Engineer for the Somerset Station Coal Conversion, retirement of L-Street Steam Station, and Michigan City Ducts Replacement. He is responsible for the preparation of specifications, schedules, and capital cost estimates; and the supervision of designers for the preparation of concrete, steel, and architectural drawings.

Mr. Barry's experience also included assignments as Lead Structural Engineer on synthetic fuel feasibility studies, and more than six years as an engineer on nuclear power plants.

Since joining Stone & Webster Engineering Corporation, he has completed the Career Development Program and has been licensed as a Registered Professional Engineer in the Commonwealth of Massachusetts and the State of Indiana.

DETAILED EXPERIENCE RECORD
BARRY, PAUL F.   03675

July 1971 to date	STONE & WEBSTER ENGINEERING CORPORATION, BOSTON, MASSACHUSETTS

Appointed to position of Engineer in the Structural Division (August 1973).

Appointed to position of Structural Engineer in the Structural Division (September 1979).

Dec. 1979 to date
Operation Services
Northern Indiana Public Service Company
Michigan City Unit 12

Structural Engineer for shoring support of existing air heater to precipitator gas duct

Structural Engineer for replacement of existing air heater to precipitator gas duct with a new high velocity gas duct

Structural Engineer for addition of the boiler room ventilation fan silencer and silencer enclosure

Structural Engineer for a feasibility study of the addition of man-safe dampers in the gas ductwork

Sept. 1981 to date
Operation Services
EVA Service Corporation
Somerset Station Coal Conversion

Lead Structural Engineer for a study to determine the technical and economic feasiblilty to reconvert the Somerset Station to coal. Work includes developing structural arrangements, schedules, and capital cost for the coal yard, particulate collection system, and the fly and bottom ash system.

Sept. 1981 to date
Operation Services
Boston Edison Company
Retirement of L-Street Steam Station

Lead Structural Engineer for the modification required for the retirement of the L-Street Steam Station and the equipment necessary to replace its function. Work includes preparation of specifications and the supervision of designers for the preparation of concrete, steel, and architectural drawing to support an auxiliary boiler and exhaust stack, and for enclosures around existing equipment.

1981-1980 Nov.-Oct.
E. Koppleman
Production of High Grade Solid Fuels from Wood Waste and Peat

Lead Structural Engineer for a study to determine the technical and economic feasibility to convert wood waste and peat into a high grade solid fuel. Work included developing a site plan, conceptual layouts of plant structures, wood handling, peat harvesting, material quantities, and engineering and construction schedules.

1981-1980 Dec.-Oct. Florida Power Corporation
Higgins Gasification/Repowering Project

Lead Structural Engineer for a study to determine the technical and economic feasibility to repower the existing Higgins Station by integrating coal gasification with a combined cycle gas turbine. Work includes developing a site plan, conceptual layout of plant structure, coal handling, material quantities, and engineering and construction schedules.

1979-1975 Nov.-Aug. Virginia Electric and Power Company
North Anna - Units 3 and 4

Structural engineering for the Service Building and Control Room

Structural engineering for caisson foundations, grating and stair tread purchase orders, electrical structures, and architectural details

Civil Engineering

Structural Engineering for Water Treatment Building

Structural responsibility for plant security

1975-1975 Aug.-Jan. Maine Yankee Atomic Power Company
Maine Yankee Atomic Power Station

Structural engineering for circulating water diffuser system, including design of supplementary pipe tiedown structural system and construction liaison

Engineering and specification for design and construction of seismic wall in circulating water pumphouse

1975-1975 Aug.-May Assisted equipment specialist in preparing master specification for purchase and installation of circulating water pipe.

1975-1974 Jan.-June Wisconsin Electric Power Company
Koshkonong Nuclear Power Plant

Structural engineering for the ultimate heat sink, including the service water pumphouse

Structural engineering for circulating water system, including the circulating water pumphouse, natural draft cooling towers, and circulating water piping

Wrote the structural portions of the Description of Work.

Provided structural input for the Preliminary Estimate.

Structural responsibility for plant security

1974-1973 June-Aug. Long Island Lighting Company
Jamesport Nuclear Power Station

Support Engineer

Worked on Description of Work, wrote sections for PSAR, developed service building layout, and engineered the access-egress drawings and coordination for the Design Criteria.

1973-1971 Aug.-July Career Development Program in Structural Division

Completed 6 months as a Quality Control Inspector at Shoreham Nuclear Power Station, Long Island Lighting Company.

Completed 1 month as a Field Engineer on a transmission line for Blackstone Valley Electric Company.

Completed 1 month as an aide in Geotechnical Division.

Assisted in economic studies for environmental report for River Bend Project for Gulf States Utilities.

Assisted in seismic design of Radwaste Building at Haddam Neck for Connecticut Yankee Electric Company.

Designed steel for precipitator support and miscellaneous concrete for Canal Electric Company.

Designed and checked transmission towers and foundations for Savannah Electric Company, Canal Electric Company, Duquesne Electric Company, and Baltimore Gas and Electric Company.

SCOTT, ALFRED B., JR. March 1982

EDUCATION	B.S. in Mechanical and Metallurgical Engineering - 1948 University of Wisconsin	
EXPERIENCE SUMMARY		
Nov. 1976 to date	STONE & WEBSTER ENGINEERING CORPORATION, BOSTON, MA	
Oct. 1980 to date	Chief Construction Engineer	River Bend Station Gulf States Utilities Company St. Francisville, LA
	Concrete coordination for project to enable project to get more uniformity in concrete produced by subcontractor. In addition, working with subcontractor to get better cleanliness, stockpile drainage, adequate stockpile watering, changes in ground hopper and more consistent concrete. Maintaining statistics on concrete batching. Responsible for "Closing of E&DCR's, IS 255 Report and N&D's" for Structural Department.	
1980 - 1980 Oct. - May	Chief Construction Engineer	River Bend Station Gulf States Utilities Company St. Francisville, LA
	Supervising all crafts on Off Plot Area and Temporary Facilities.	
1980 - 1979 Oct. - June	Assistant Resident Engineer	River Bend Station Gulf States Utilities Company St. Francisville, LA
	Area engineering in Area I, which consists of Reactor, Auxiliary Building, Radwaste and Fuel Building. Responsible for construction coordination of all phases of construction.	

1979 - 1978 June - April	Assistant Resident Engineer	River Bend Station Gulf States Utilities Company St. Francisville, LA

Assumed all Resident Engineer responsibilities.

1978 - 1976 April - Nov.	Assistant Resident Engineer	River Bend Station Gulf States Utilities Company St. Francisville, LA

Work directly for the Resident Engineer and in his absence am responsible for all site engineering and coordination with the Cherry Hill Operations Center. Review of Engineering and Design Coordination Reports; incoming mail and documentation to assure proper action and distribution; review and approve contract changes, field work orders, back charges, invoices and construction change orders; and review of preliminary drawings all specifications. Am handling all erosion problems at this time. There are now 25 engineers working in the field or the field office at this time.

1976 - 1972
Sept. - June

BECHTEL POWER CORPORATION, SAN FRANCISCO, CA

Senior Field Engineer	Willow Glen No. 5 Unit Gulf States Utilities Company St. Gabriel, LA

Responsible for the field engineering and installation of an emergency gas pipeline, engineering of a tank farm and all piping. Then I was responsible for the mechanical discipline for the construction of the Number 5 generating unit. Due to lack of experienced personnel, I was also put in charge of all subcontract work. For the last year, I assumed the responsibilities of all field engineering work.

1972 - 1971
June - Jan.

VAN DEUSEN AND COMPANY, PORTLAND, OR

Field Engineer

Responsible for the management and inspection of work being performed by subcontractors in the construction of a Methanol and Phenol Chemical Complex for the Georgia Pacific Corporation at Plaquemine, Louisiana, by local contractors.

1971 - 1970
Jan. - Aug.

GULF COAST ALUMINUM CORPORATION, LAKE CHARLES, LA

Superintendent	Carbon Paste Plant Lake Charles, LA

Responsible for the construction of this plant for the Plant Service Construction Company of Baton Rouge, LA.

1970 - 1968 June - Aug.	MCCARTY CORPORATION, BATON ROUGE, LA	
	Contract Engineer	
	Estimating, bidding and managing work if bid was successful for insulation contracting.	
1968 - 1951 July - March	BECHTEL CORPORATION, SAN FRANCISCO, CA	
1968 - 1967 July - Aug.	Chief Field Engineer	Grass Roots Refinery Antar Oil Company Valenciennes, France
	Responsible for all field engineering and the coordination and management of subcontracts using four national groups.	
1967 - 1966 July - Nov.	Job Engineer	Grass Roots Refinery Texaco Oil Company Convent, LA
	Assumed all job engineering responsibilities when that engineer left the company's employ.	
1966 - 1966 Nov. - May	Supervisor Field Engineer	Grass Roots Refinery Texaco Oil Company Convent, LA
	Responsible for all field engineering.	
1966 - 1965 May - Feb.	Chief Field Engineer	Grass Roots Ammonia Plant Continental Oil Company Blytheville, AR
	Responsible for all field engineering.	
1963 - 1964 Feb. - Oct.	Chief Field Engineer	Home Office Assignment
	Preparation of work planning schedules, manpower studies, critical path studies and rigging diagrams for next assignment.	
1964 - 1962 Oct. - Nov.	Construction Supervisor	Grass Roots Refinery Regent Oil Company Pembrokeshire, Wales, U.K.
	Responsible for construction of all off-plot facilities and all permanent buildings.	
1962 - 1962 Nov. - Jan.	Assistant Superintendent	Grass Roots Chemical Plant Complex Tenneco Chemical Company Houston, TX
	Responsible for the construction of all off-plot facilities.	

1962 - 1961 Jan. - June	Assistant Superintendent	Grass-Roots Plastic Manufacturing Plant Amoco Chemical Corporation Texas City, TX
	Responsible for construction of all phases of the plant.	
1961 - 1959 June - Dec.	Chief Field Engineer	Grass Roots Chemical Plant Hercules Power Company Lake Charles, LA
	Responsible for all field engineering.	
1959 - 1959 Dec. - June	Chief Field Engineer	Refinery Expansion Continental Oil Company Westlake, LA
	Responsible for all field engineering.	
1959 - 1958 June - March	Chief Field Engineer	Refinery Expansion Imperial Oil Company Calgary, Alberta, Canada
	Responsible for all field engineering.	
1958 - 1957 Mar. - Nov.	Supervisor Field Engineer	Titanium Plant U. S. Industrial Chemical Corporation Ashtabula, OH
	Coordinated all extra work requested by the client with construction forces and performed jobsite engineering for completion of the Titanium Plant.	
1957 - 1957 Nov. - Sept.	Supervisor Field Engineer	Zirconium Sponge Plant U. S. Industrial Chemical Corporation Ashtabula, OH
	Special assignment to coordinate all activities to make the sponge plant operable. Worked directly with client personnel to make required changes. Made detailed cost study of this work for client's Board of Directors.	
1957 - 1956 Sept. - Jan.	Supervisor Field Engineer	Butadiene Plant Expansion Petro-Tex Chemical Corporation Houston, TX
	Responsible for all field engineering and start-up.	

1956 - 1955 Jan. - July	Senior Field Engineer	Refinery Expansion Tidewater Associated Oil Company Avon, CA
	Responsible for all field engineering in the construction of expanded off-plot facilities and pipelines.	
1955 - 1955 July - May	Senior Field Engineer	Grass Roots Refinery Shell Oil Company Anacortes, WA
	Temporary assignment to replace a superintendent who had a heart attack. Finished his assignment of construction of Administration Complex for the refinery.	
1955 - 1954 May - Sept.	Senior Field Engineer	Catalytic Reformer Unit Standard Oil Company Torrance, CA
	Responsible for all field engineering.	
1954 - 1953 Sept. - Aug.	Senior Field Engineer	Thermo Catlytic Reforming Unit General Petroleum Corporation Torrance, CA
	Responsible for all field engineering.	
1953 - 1952 Aug. - Oct.	Senior Field Engineer	Propyl Polymerization and Treating Units Gulf Oil Corporation Port Arthur, TX
	Responsible for all field engineering.	
1952 - 1952 Oct. - July	Senior Field Engineer	Natural Gas Compressor Station Tennessee Gas Transmission Company Mercer, PA
	Responsible for all field engineering.	
1952 - 1951 July - Nov.	Senior Field Engineer	Water Treating And Condensate Filtration Units Union Oil Company Lomita, CA
	Responsible for all field engineering.	
1951 - 1951 Nov. - March	Senior Field Engineer	Natural Gas Compressor Station Pacific Gas & Electric Company Needles, CA
	Responsible for all field engineering.	

1951 - 1950 March - Sept.	<u>TEXAS ILLINOIS NATURAL GAS PIPELINE COMPANY, CHICAGO, IL</u> Chief Inspection, Spread 7 and Spread 9 Supervised the activities of pipeline inspectors on these two spreads for the construction and installation of a 30 inch natural gas pipeline from Texas to Illinois.
1950 - 1950 Sept. - Feb.	<u>TEXAS GAS TRANSMISSION COMPANY, OWENSBORO, KY</u> Construction Engineer Responsible for writing engineering procedures for material control, concrete specifications, piping specifications, building specifications, material take-off and requisitioning for added construction.
1950 - 1949 Feb. - Oct.	<u>STONE & WEBSTER ENGINEERING CORPORATION, BOSTON, MA</u> Field Engineer — Texas Gas Transmission Company, Ruston, LA Responsible for field engineering and releted problems in the construction of a natural gas compressor station.
1949 - 1949 July - Feb.	Field Engineer — TetraEthly Lead Plant, Baton Rouge, LA Responsible for field engineering and related problems, preparation of engineering reports, material take-offs requisitioning, craft scheduling and work planning for the lead melting facilities.
1948 - 1948 Nov. - Feb.	<u>AMERICAN STEEL FOUNDRIES, CHICAGO, IL</u> Research Metallurgist Performed research of foundry sands, cereals, resinous and plastic core binders. Studies causes and remedies of faulty castings.
1948 - 1946 Feb. - March	<u>EDUCATION AND PART-TIME EMPLOYMENT</u> Carpenter Journeyman
1946 - 1943 March - Feb.	<u>U. S. ARMY SIGNAL CORPS.</u> Sergeant Radar Mechanic-Construction and Maintenance-Honorable Discharge
1943 - 1939	<u>PART TIME EMPLOYMENT</u> Carpenter Apprentice

HOLSINGER, BARRY L. January 1982

EDUCATION - Stone & Webster Management Studies Program, North Anna Power Station 1974
Stone & Webster Radiological Safety Refresher, North Anna Power Station 1974
Stone & Webster Radiological Safety School, North Anna Power Station 1973
Virginia Department of Highway Troxler Training School, Staunton District, Staunton, Virginia 1969

EXPERIENCE SUMMARY

Mr. Holsinger has performed various functions in the Quality Control and Quality Assurance Departments on nuclear power plant construction for the past eleven years.

In 1970, Mr. Holsinger joined the Stone & Webster Quality Control Division as an inspector performing inspections and tests for the Civil/Structural Discipline at North Anna Station, Mineral, Virginia.

During 1976, Mr. Holsinger became a supervisor assigned to training inspectors and technicians; plan and schedule inspection and testing within the Civil/Structural Discipline at Millstone Unit 2 Nuclear Power Station, Waterford, Connecticut.

Mr. Holsinger was assigned to a TMI modification in 1980 where he assisted in establishing the QA Program requirements, performed the inspection and testing and established files for maintaining those documents required at Prairie Island Nuclear Power Station, Red Wing, Minnesota.

In 1981, Mr. Holsinger joined the Quality Assurance Division to develop and implement a surveillance plan that would insure the Quality Assurance Program implemented and consistent with company policies for the nuclear power plant construction at Nine Mile Unit 2 Nuclear Power Station, Lycoming, New York.

DETAILED EXPERIENCE RECORD
BARRY L. HOLSINGER

September 1970 to Present	STONE & WEBSTER ENGINEERING CORPORATION, BOSTON, MASSACHUSETTS
March 1981 to present	Quality Assurance Engineer - Nine Mile Unit 2 Nuclear Power Station Quality Assurance Engineer assigned at the site to assist the Project QA Manager in monitoring, controlling, and reporting on all site quality assurance activities. Responsibilities include reviewing Engineering and Design Coordination Reports and Nonconformance and Disposition Reports for quality requirements, quality engineering of Civil/Structural activities, interface Client QA concerns and evaluate the effectiveness of the S&W Program.
October 1980 to February 1981	FQC Engineer Assigned to the Northern States Power Company's Prairie Island Nuclear Power Plant in Redwing, Minnesota to develop a Quality Assurance Program and perform inspection in the Civil/Structural Discipline for the Auxiliary Building Post TMI Modifications. Major activities included developing client relations, developing inspection plans for batch plant, drilled-in anchors, structural steel, AWS welding, concrete preplacement and placement inspection, purchasing and receiving, developing and maintaining files of records generated.
April 1977 to October 1980	FQC Engineer - Millstone III Nuclear Power Station Responsible for planning, scheduling, training, and supervision, assuring inspection criteria conformed to job specifications and codes (NRC Regulatory Guides, ANSI, ACI, AWS, AISC, and ASTMs) for the Civil/Structural Discipline. Major activities within the civil/structural discipline included concrete laboratory - mix designs - concrete/aggregate testing, batch plant inspection and testing, reinforcing steel inspection and testing, structural steel inspection, AWS welding inspection, and ANSI painting inspection and testing.
March 1976 to April 1977	Senior Inspector - Millstone III Nuclear Power Station Responsible for the supervision of nine inspectors within the FQC Structural Discipline. Responsibilities include: site lab, concrete and soils test, inspection and documentation.
October 1974 to February 1976	FQC Inspector/Senior Inspector - Millstone III Nuclear Power Station Responsible for all Site FQC Laboratory functions. Duties included initial start up of the site lab, calibration of field and lab concrete and soils testing equipment. Also trained personnel for lab testing, prepared the lab for CCRL certification, compile and revise documentation and records, supervised one inspector and three technicians.

August 1974 to October 1974	Participated in S&W's management studies program, North Anna Power Station
May 1974 to August 1974	<u>FQC Inspector</u> - Surry Nuclear Power Station Audited S&W's Geotech. Division soils investigation program for Units 3 & 4, audit included splitbarrel sampling, Hvorslev tube sampling, Vibrofloation Corporation's densification probes, and Prof. J. H. Schmertmann's static core penetrometer soundings.
March 1974 to May 1974	<u>FQC Inspector</u> - North Anna Power Station Supervisor of concrete and soils testing laboratory on site.
January 1974 to March 1974	<u>FQC Inspector</u> - Gulf States Utilities, Riverbend Station I & II Audited S&W's Geotech. Division soils densification test program.
September 1973 to January 1974	<u>FQC Inspector</u> - North Anna Power Station Performed soils and concrete tests at the site laboratory. Concrete tests consisted of air content, slump, fresh unit weight, cylinder compressive strength, sieve analysis and specific gravity. Soil tests consisted of moisture - density relationship, atterberg limits, hydrometer analysis, sieve analysis and specific gravity.
July 1972 to September 1973	<u>FQC Inspector</u> - North Ana Power Station Performed soil tests and inspection of the service water reservoir. Tests include the Troxler, sand cone, sieve analysis and sampling from test panels. Inspection included subcontractors compliance to specifications and drawings. Assisted and inspected in the Geotech. Division in the installation of piezometers, rock blasting and rock bolting operations for Unit 3 & 4 containment.
September 1970 to July 1972	<u>Quality Control Technician</u> - North Ana Nuclear Power Station, Mineral, VA (VEPCO) Performing soil tests and inspection for N.A.P.S. roads and bridges, dam, and dikes. Tests include Troxler Nuclear Gauge, sand cone and inspection subcontractors to assure compliance to specifications and drawings.
August 1970 to September 1970	Greer Bros. & Young, Louisville, Kentucky <u>Foreman</u> - Supervising and installing drain pipe for Highway construction.
June 1969 to July 1970	Virginia Dept. of Highway, Verona, Virginia Soils testing for interstate highway construction.

May 1969 to June 1969	James Whitmore, Woodstock, Virginia Training as a surveyor. Duties included Rodman, Levelman, Chainman, Transitman, calculations for and plotting survey plats.
February 1969 to April 1969	Aileen Mfg. Company, Edinburg, Virginia Training for plant engineer. Primary duty, operational cost studies.
January 1966 to December 1968	United States Army - honorable discharge Company clerk - Rank E-4
September 1964 to November 1965	Virginia Dept. of Highway, E inburg, Virginia Soils testing for interstate highway construction.

THOMAS R. KUESEL
Senior Vice President
Partner, Principal Professional Associate
Structural Engineer

Education
Yale University, B.E. 1946; M. Eng. 1947

Societies
National Academy of Engineering
American Society of Civil Engineers
American Consulting Engineers Council
The Moles (honorary tunneling fraternity)
British Tunnelling Society
Structural Engineers Association of California
Charter Member, U.S. National Committee on Tunneling Technology (1972-74)

Licenses
New York, California, New Hampshire, Massachusetts, Connecticut, Pennsylvania, Delaware, Maryland, District of Columbia, Virginia, South Carolina, Georgia, Florida, Texas, Ohio, Illinois, Michigan, Colorado, Washington, Hawaii
National Bureau of Engineering Registration, Certificate of Qualification

Mr. Kuesel, who joined Parsons Brinckerhoff in 1947, became a partner and officer of the firm in 1968. He has over 30 years of experience on major structural projects including long-span and movable bridges, tunnels, and complex structures. He has participated in over 80 tunnel projects on five continents. His present responsibilities in the direction of major underground projects include the Fort McHenry Tunnel in Baltimore, Maryland; the Second Downtown Tunnel under the Elizabeth River at Norfolk, Virginia; and the Anacostia River Tunnel of the Washington, D.C., Metro transit system.

Among Mr. Kuesel's past projects are:

• Hard Rock Tunnels: NORAD Combat Operations Center, Colorado Springs; Peachtree Center Station, MARTA transit system, Atlanta, Georgia; Potomac River Tunnel, Washington, D.C. Metro transit system.

• Soft Ground Tunnels: Lexington Market Tunnels, Baltimore transit system; 7th St. Tunnels (Section F-F-2) Washington, D.C. Metro transit system; Red Hook Tunnel, Brooklyn, New York.

• Cut-and-cover Tunnels: Harvard Square and South Cove Stations, Boston transit system, Massachusetts; Lexington Market Station, Baltimore Transit System; Waterfront Station, Washington, D.C. Metro transit system.

• Immersed Tube Tunnels: Second Hampton Roads Tunnel, Hampton-Norfolk, Virginia; 63rd St. Tunnel, New York City; BART Trans-Bay Tube, San Francisco, California.

• Served as Chairman of the Seismic Advisory Boards, as well as Senior Technical Advisor, for the Stanford Linear Accelerator Positron-Electron Project and the San Francisco Ocean Outfall Project

• From 1963 to 1968, directed the design of the San Francisco Bay Area Rapid Transit (BART) System. For four of these years, he was assistant manager of engineering for Parsons Brinckerhoff-Tudor-Bechtel—general engineering consultants for BART—and for one year, he served as project manager based in the San Francisco office of Parsons Brinckerhoff. Mr. Kuesel developed BART's civil and structural design criteria, which included unique provisions for resistance to earthquakes. He also reviewed and approved all plans and specifications for heavy construction contracts.

T.R. Kuesel Professional Experience

Bridges

Date	Project	Location	Bridge Type											Services								
																	Consultation				
			Long Span*	Movable†	Viaduct	Grade Separation	Highway	Rail	Pedestrian	Steel	Concrete	Water	Land	Study	Report	Preliminary Design	Final Design	Design	Construction	Operations	Renovation	Construction Inspection
1947	Montour Railroad Bridge, Pittsburgh Airport Parkway	Pennsylvania				●		●		●			●					●				
1947	Montour Run Bridge, Pittsburgh Airport Parkway	Pennsylvania			●		●			●			●				●					
1947	Route 11 Bypass Bridge, Stanton	Virginia				●	●			●	●		●			●	●					
1948	Hampton Harbor Bridge	New Hampshire		B	●		●			●		●							●			
1948-49	Long Bird Bridge	Bermuda		Sw			●					●				●	●		●			
1948	Boston & Maine Line Railroad Bridge, New Hampshire Turnpike	New Hampshire				●		●					●						●			
1948-52	York River Bridge (George P. Coleman Memo. Bridge)	Virginia	Tr	2Sw			●			●		●				●	●		●			
1949-54	Sunshine Skyway, Lower Tampa Bay	Florida	Tr/G	B	5		●			●	●	●				●	●		●			
1950	Walnut Street Bridge, Harrisburg	Pennsylvania	Tr		●		●			●		●			●						●	
1950	Susquehanna River Bridge, Pennsylvania Turnpike	Pennsylvania			●		●			●		●					●					
1951	Utica Street Bridge, Oswego	New York			●		●			●		●				●	●		●			
1951	Hawk Street Viaduct, Albany	New York	A				●			●			●		●						●	
1951-52	Palma Sola Bridge, Manatee County	Florida		B	●		●			●	●	●				●	●		●			
1951-52	Cortez Bridge, Manatee County	Florida		B	●		●			●	●	●				●	●		●			
1951-52	Longboat Bridge, Manatee County	Florida		B	●		●			●	●	●				●	●		●			
1951-53	Fleming Park Bridge, Pittsburgh	Pennsylvania	Tr/G		●		●			●		●				●	●		●			
1951	Myrtle Avenue Bridge, Jacksonville	Florida	A				●			●			●				●					
1951	Fuller Warren Bridge, Jacksonville	Florida		B	●		●			●		●						●				
1951-54	Savannah River Crossing (Eugene Talmadge Memorial Bridge)	Georgia	Tr/G		2	1	●			●	●	●	●		●	●	●		●	●		●
1952	Rio Hondo Bridge, Harlingen	Texas		VL			●			●		●				●	●		●			
1952-53	Ringling Causeway, Sarasota	Florida		B	●		●			●	●	●				●	●		●			
1952-60	Arthur Kill Bridge, Staten Island	N.Y.-New Jersey		VL				●		●		●			●	●	●		●			
1953	St. Lucie River Bridge, Martin County	Florida		B	●		●			●	●	●					●					
1953	Indian River Bridge, Martin County	Florida		B	●		●			●	●	●					●					
1953-56	Housatonic River Bridge, Connecticut Turnpike	Connecticut	G		●		●			●		●	●			●	●		●			●
1953-56	Stratford Bridges, Connecticut Turnpike	Connecticut				2	●			●			●			●	●		●			●
1953	Lake Champlain Crossing	N.Y.-Vermont	Tr		●		●			●		●		●								
1954-56	Fuhrman Boulevard-Hamburg Turnpike Viaduct, Buffalo	New York	G		●		●			●			●			●	●					
1954-56	62nd Street Bridge, Pittsburgh	Pennsylvania	Tr		●		●			●		●						●	●			
1954	Ohio Street Bridge, Buffalo	New York		B								●			●						●	
1954-55	BART Project, Preliminary Structure Designs	California			●			●		●	●		●		●	●						
1955	Cocheco River Bridge, Spaulding Turnpike	New Hampshire			●		●			●		●	●			●	●					
1955-58	I-84 Bridges, Danbury	Connecticut				14	●			●	●		●			●	●		●			
1956	Avondale Bridge, Clifton	New Jersey		Sw			●			●		●			●						●	
1957-59	Schenectady Interchange	New York	G		●	5	●		3	●			●			●	●		●			
1957-63	Newport Bridge, Narragansett Bay	Rhode Island	Su/Tr		●		●			●		●		●	●	●						
1957-58	I-91 Bridges, Rocky Hill	Connecticut				3	●			●			●			●	●		●			
1958	English Channel Bridge Studies	England-France	Tr		●		●	●		●		●		●	●							
1958	Mystic River Bridge Reconstruction, Boston	Massachusetts			●		●			●								●			●	
1959-61	Grand Central Parkway Reconstruction	New York				7	●			●	●		●			●	●		●			
1959, 74	St. Simon's Causeway, Brunswick	Georgia		2VL			●			●		●		●	●							
1959	Lake Street Bridge, Elmira	New York			●		●			●		●				●	●		●			
1959	Appomattox River Bridge, Richmond-Petersburg Turnpike	Virginia			●		●			●		●	●			●	●		●			
1960	Kenova Bridge	Kentucky, Ohio	Tr				●			●		●		●	●							
1960	Bridge Creek Dam Bridge, Akron	Ohio			●		●			●		●					●					
1960	Burlington Canal Bridge, Hamilton	Ontario		VL			●	●		●		●						●				
1961	Pennsylvania Railroad Bridge, Lock & Dam 41, Louisville	Kentucky, Ohio		VL				●		●		●		●	●							
1961	Ohio River Bridge, Cincinnati Circumferential Highway	Kentucky, Ohio	3 Tr				●			●		●		●	●							
1961	Bergen-Passaic Expressway (I-80)	New Jersey			2		●			●			●				●					

Bridges (Cont'd)

Date	Project	Location	Bridge Type: Long Span*	Movable†	Viaduct	Grade Separation	Highway	Rail	Pedestrian	Steel	Concrete	Water	Land	Services: Study	Report	Preliminary Design	Final Design	Consultation: Design	Construction	Operations	Renovation	Construction Inspection
1961	Albany Riverfront Arterial	New York			●		●			●			●			●	●					
1961	Long Island Sound Crossing	N.Y.-Conn.-R.I.	Su/Tr/G		●		●			●		●		●	●							
1962	Wabash River Bridge, Vincennes	Indiana			●			●		●		●		●	●						●	
1962	Tagus River Bridge, Lisbon	Portugal	Su				●	●		●	●	●		●	●							
1963-68	BART Project – Standard Aerial Structures	California			●			●		●	●		●			●	●		●			●
	– Special Aerial Structures	California			●			●		●	●		●			●	●		●			●
	– Pedestrian Bridges	California				●			2				●			●	●		●			●
1969-71	Marshall Street Viaduct, Richmond	Virginia			●		●			●			●	●	●						●	
1969	Passaic Falls Bridge, Paterson	New Jersey	A				●			●			●	●	●							
1970	Cape Fear Bridge Collision Repair, Wilmington	North Carolina		VL						●		●									●	
1970	Turnagain & Knik Arm Crossings, Anchorage	Alaska			●		●			●	●	●		●								
1970-72	Halawa Interchange Bridges, Oahu	Hawaii				2							●					●				
1971-79	James River Bridge, Newport News	Virginia		VL	●		●			●	●	●		●	●	●	●		●		●	●
1971	James River Bridge, I-295	Virginia	Tr/G				●			●		●		●	●	●						
1971-76	Leigh Street Viaduct (Martin Luther King, Jr. Memorial Bridge), Richmond	Virginia	G		●		●			●			●			●	●		●			
1972	Golden Gate Bridge South Approach Reconstruction	California			●		●			●			●					●			●	
1972-73	Bolivar Roads Crossing, Galveston	Texas	Tr/G		●		●			●		●		●	●							
1972-79	Third Street Bridge, Wilmington	Delaware		B			●			●		●				●	●		●			
1973-77	Berkley Bridge, Norfolk	Virginia		B	●		●			●		●			●					●		
1973-76	Curtis Creek Bridge, Baltimore	Maryland		B			●			●		●				●	●		●			
1974	Route H-3 Windward Viaduct, Oahu	Hawaii	G		●		●				●		●		●			●				
1974	Long Bird Bridge Rehabilitation	Bermuda		Sw			●			●		●			●		●				●	
1974-75	Leigh Street Pedestrian Bridge, Richmond	Virginia				●			●		●		●				●		●			
1974	Buzzard's Bay Bridge, Cape Cod Canal	Massachusetts		VL				●		●		●			●						●	
1974	St. George's Bridge Deck Rehabilitation	Delaware	A		●		●			●			●				●				●	
1975	Fore River Bridge, Quincy	Massachusetts		VL			●			●		●		●	●							
1975, 79	Market Street Bridge, Wilmington	Delaware		B			●			●		●			●						●	
1975	Cedar Creek Bridge, Milford	Delaware		Sw			●			●		●			●						●	
1975-77	Keehi Interchange, Oahu	Hawaii			●		●				●		●		●			●				
1975	Kingston Harbor Bridge	Jamaica		B, F	●		●			●	●	●		●	●							
1976-78	Pelican Island Causeway Reconstruction, Galveston	Texas		B	●		●	●		●	●	●		●	●	●	●				●	
1976	Chelsea Street Bridge, Boston	Massachusetts		B			●			●		●		●	●						●	
1976	MARTA Project – Standard Aerial Station Girders	Georgia			●			●			●		●					●				
1976-77	Congress Avenue Bridge, Austin	Texas			●		●			●	●	●		●	●	●	●					
1975, 79	Alewife Parkway Bridge, Boston	Massachusetts			●		●			●	●		●	●		●	●					
1977	Miami River Crossing, Miami Transit	Florida			●			●		●	●	●	●	●								
1978-80	Saugatuck River Bridge (Northeast Corridor)	Connecticut		B				●		●		●		●	●	●	●				●	
1979	West Seattle Bascule Bridge	Washington		B			●			●		●				●						
1980-81	West Seattle High Level Bridge	Washington	G		●		●			●	●	●	●			●	●		●			
1980-81	Memorial Drive Underpass, Dallas	Texas				●		●			●		●						●			
1980-82	Hood Canal Bridge	Washington		F,D			●					●				●	●					
1981	Prairie du Chien Bridge, Mississippi River	Wisconsin	A				●			●		●									●	
1981	Jefferson Parish Bridge, Mississippi River	Louisiana	Tr		●		●			●		●	●	●	●							
1982	West Seattle Movable Bridge			Sw, B,VL			●			●	●	●		●	●	●						
1982	I-395 Bridge, Bangor	Maine	G				●				●	●				●	●					

Key: *Tr = Truss
G = Girder
A = Arch
Su = Suspension

†B = Bascule
VL = Vertical Lift
Sw = Swing
F = Floating
D = Draw

Numbers listed are numbers of bridges of that type in project.

T.R. Kuesel Professional Experience

Tunnels and Underground Construction

Date	Project	Location	Construction Type								Services								
															Consultation				
			Hard Rock	Soft Ground	Immersed Tube	Cut-and-Cover	Stations	Highway	Rail	Other	Study	Report	Preliminary Design	Final Design	Design	Construction	Operations	Renovation	Construction Inspection
1947	West Rock Tunnel (Wilbur Cross Parkway)	Connecticut	●					●							●				
1950	Fort Ritchie Project	Maryland	●							Defense				●					
1960-61	3 Hardened Military Centers	Nebr., Colo., Ill.	●							Defense		●							
1961	Newport Tunnel	Rhode Island			●			●				●							
1962-63	NORAD Combat Operations Center, Colorado Springs	Colorado	●							Defense				●		●			
1963-73	San Francisco Bay Area Rapid Transit System (BART)	California	2	12	1	6	14		●				●	●		●	●		●
1968	Franconia Tunnels	New Hampshire	●					●				●							
1968	Cope-Lagoa Tunnel, Rio de Janeiro	Brazil	●					●			●								
1968	Sao Paulo Metro	Brazil		●		●	2		●						●				
1968-73	63rd Street Tunnel (East River, New York City)	New York	●		●				●						●	●			●
1969-70	Caracas Metro	Venezuela		●					●						●				
1969	St. Louis Transit System	Missouri	●			●	●		●			●							
1969-75	Red Hill Tunnel (Route H-3, Oahu)	Hawaii	●			●		●				●	●	●					
1969-75	Trans-Koolau Tunnel (Route H-3, Oahu)	Hawaii	●			●		●				●	●						
1970	Messina Straits Tunnel	Italy–Sicily	●		●			●	●			●	●						
1970	Knik Arm Tunnel, Anchorage	Alaska		●	●			●			●								
1970	Project One, Kawasaki	Japan			●			●				●	●						
1970-74	Route 131-C, NYCTA (Long Island Expressway Subway)	New York				5	5		●			●	●						
1970-77	First Hampton Roads Tunnel (Hampton–Norfolk)	Virginia			●	●		●									●	●	●
1970-77	Second Hampton Roads Tunnel (Hampton–Norfolk)	Virginia			●	●		●				●	●	●		●	●	●	●
1971-72	Great Belt Tunnel	Denmark			●				●		●	●							
1971-72	Chelsea River Water Tunnel, Boston	Massachusetts			●					Water			●	●		●			
1971-76	Washington Metro, Section C-4 (Potomac River Tunnels)	Dist. of Columbia	●	●					●					●		●			
1971-73	Forest Park Tunnel (J.F.K. Airport Rail Connection)	New York		●		●			●				●	●					
1971-73	Bedrock Waste Storage Project (AEC Savannah River Plant)	South Carolina	●							Nuclear Waste	●	●							
1971	Saw Mill River Tunnel, Yonkers	New York	●							Flood Control	●	●							
1971-75	Route 131-B, NYCTA (Long Island Super-Express Route)	New York		2		2	1		●			●	●						
1972	Chicago Metropolitan Sanitary District, Underground Treatment Plant	Illinois	●							Waste Treatm't	●								
1972	Kansas City Transit System	Missouri		●		●	●		●			●							
1972	Fort Worth Transit System	Texas	●			●	●		●			●							
1972-75	Penalisa & La Llana Tunnels (Pan American Highway)	Colombia	2					●							●	●			
1972-79	63rd Street Tunnel Approaches (Long Island Rail Road)	New York	●						●							●			
1972	South Cove Tunnel (MBTA, Boston)	Massachusetts				2	1		●						●	●			
1972	Molokai Water Supply	Hawaii	●									●							
1972-73	Bolivar Roads Crossing, Galveston	Texas			●	●		●				●	●						
1973-78	Washington Metro, Section F-2 (7th Street Tunnels, Waterfront Station)	Dist. of Columbia		●		●	1		●					●		●			
1973	Santo Domingo Tunnel	Dominican Rep.			●						●								
1973	Minneapolis-St. Paul Transit System	Minnesota	●				●		●			●							
1973-82	Detroit-Windsor Tunnel	Michigan–Ontario		●	●	●		●									●	●	●
1973-77	Downtown Elizabeth River Tunnel (Norfolk–Portsmouth)	Virginia			●	●		●									●	●	
1973-77	Midtown Elizabeth River Tunnel (Norfolk-Portsmouth)	Virginia			●	●		●									●		
1973-74	Denver Water Supply Tunnels (Eagle-Piney Project)	Colorado	●							Water		●							
1973	Hong Kong Cross Harbour Tunnel	Hong Kong			●												●		
1974, 77	Eastern Suburbs Railway, Sydney	Australia	●						●		●					●			
1974	Perth Railroad Tunnels	Australia		●		●			●		●								

T R Kuesel Professional Experience

Miscellaneous Projects

Date	Project	Location	Description	Services								
								Consultation				
				Study	Report	Preliminary Design	Final Design	Design	Construction	Operation	Renovation	Construction Inspection
1949	Chalmette Wharf, American Sugar Refining Co.	Louisiana	Port Structure				●					
1950-52	Farrel-Birmingham Co., Ansonia	Connecticut	Heavy Industrial Buildings	●	●		●				●	●
1952	Brooklyn-Staten Island Piers	New York	Valuation Report		●							
1954	Garden State Parkway Service Area	New Jersey	Geometric Design				●					
1954	Dunbar Bridge Toll Study, Charleston	West Virginia	Traffic Study					●				
1956	U.S. Department of Defense, Arch-Dome Structures	-	Structural Analysis		●	●						
1958	Penn Turnpike, Northeast Extension, Lehigh Gap	Pennsylvania	Route Location	●	●							
1960	New York City Marine Transfer Station	New York	Solid Waste Facility					●				
1960	Penn Turnpike, New Jersey Connection	Pennsylvania	Route Location	●	●							
1963	Cape May-Lewes Ferry	New Jersey-Delaware	Ferry Terminals		●	●						
1964	Earthquake Investigation, Anchorage	Alaska	Evaluation and Report	●	●							
1966-67	BART Project – Railroad Yards and Shops	California	Vehicle Maintenance Facilities					●				
1966-68	BART Project – Ferry Building	California	Underpinning			●	●		●		●	●
1969	Port of Callao	Peru	Port Structures					●	●			
1970	BART Project – Ferry Plaza Platform	California	Port Structures					●				
1970	Bloomingdale's Department Stores, Paramus	New Jersey	Foundation Design					●				
1970	Dry Dock No. 3, Brooklyn Navy Yard	New York	Dry Dock		●						●	
1971	Earthquake Investigation, San Fernando (Los Angeles)	California	Evaluation and Report	●	●							
1971	Norfolk Shipbuilding and Dry Dock Co.	Virginia	Port Planning		●	●						
1971-72	Potomac Electric Power Co., Washington, Substation	District of Columbia	Foundation Design			●	●		●			●
1972	Madison Beach Club	Connecticut	Foundation Rehabilitation		●						●	
1972	Earthquake Investigation, Santa Rosa	California	Evaluation and Report	●	●							
1973-75	Portsmouth Marine Terminal	Virginia	Port Structure					●	●			
1973	E.I. DuPont de Nemours & Co., Savannah River Plant	South Carolina	Erosion/Siltation Study	●	●							
1974	South Bay Maintenance Center, MBTA, Boston	Massachusetts	Vehicle Maintenance Facilities					●				
1975	South Station Rehabilitation, Boston	Massachusetts	Structural Foundation Design	●	●	●						
1975	Grand Cayman Port	Cayman Islands	Port Facilities			●	●					
1975-77	Superior Coal Transshipment Terminal	Wisconsin	Bulk Materials Handling			●	●		●			●
1975-77	Keehi Pile Test Program, Oahu	Hawaii	Instrumented Field Test			●	●		●			
1975-76	USDOT Tunnel Monitoring Investigations	Massachusetts	Research and Development	●	●							
1975-76	NYC Housing Authority, Effects of Subway Construction	New York	Building Protection	●	●				●			●
1975	Lorain Dike	Ohio	Dredging Disposal			●	●					
1975-77	Cove Point Terminal, Columbia LNG Corporation	Maryland	LNG Facilities						●			●
1976	USDOT Subway Disruption Study	Massachusetts	Research and Development	●	●							
1976-77	Elizabeth River Tunnel EIS Study	Virginia	Environmental Report	●	●							
1976-80	MARTA Project, Five Points Station Roof	Georgia	Structural Design					●	●			
1978	Hartford Civic Center Roof Failure	Connecticut	Evaluation and Report	●	●						●	
1978-79	White Plains Parking Garage	New York	Structural Design					●				
1978-79	Jourdan Road Terminal, New Orleans	Louisiana	Port Facilities					●				
1978-79	Ft. McHenry Tunnel EIS Study	Maryland	Environmental Report	●	●							
1979	OTEC Seacoast Test Facility	Hawaii	Ocean Thermal Energy			●	●					
1980-82	Kemper Arena Roof Failure, Kansas City	Missouri	Evaluation and Report	●	●						●	
1981	Niagara Frontier Transportation Authority, Buffalo	New York	Floating Slab Track	●	●			●	●			
1981-82	Ft. McHenry Dredge Spoil Containment Facility	Maryland	Dredging Disposal					●	●			●
1982	Vancouver Light Rail Transit System	British Columbia	Rail/Structure Interaction		●			●				

Tunnels and Underground Construction (Cont'd)

Date	Project	Location	Construction Type								Services				Consultation				
			Hard Rock	Soft Ground	Immersed Tube	Cut-and-Cover	Stations	Highway	Rail	Other	Study	Report	Preliminary Design	Final Design	Design	Construction	Operations	Renovation	Construction Inspection
1974	Singapore Mass Transit Study	Singapore		●		●	●		●			●							
1974	Taipei Underground Railway Report	Taiwan				●	●		●			●	●						
1974	North Link Railroad Tunnels	Taiwan	●						●			●							
1974-75	Nassau County Sewer Outfall	New York			●					Waste Water		●						●	
1975	Baytown Tunnel (Houston Ship Channel)	Texas			●	●		●									●		
1975	Los Angeles Transit "Starter Line"	California		●			●		●		●	●							
1975	Circulating Water Tunnels, Comanche Peak Nuclear Power Station	Texas	●							Water				●		●			●
1975-77	Chicago Urban Transit District	Illinois		●		●			●		●				●				
1975, 79	Queens Midtown Tunnel, New York City	New York	●	●		●		●										●	
1975, 79	Brooklyn-Battery Tunnel, New York City	New York	●	●		●		●										●	
1975-82	Baltimore Transit, Lexington Market Section	Maryland		●		●	●		●					●		●			
1975-77	Cove Point Tunnel	Maryland			●	●				LNG						●			●
1975-82	Metropolitan Atlanta Rapid Transit System (MARTA)	Georgia	●	●		●	3		●					●	●	●			●
1976	Orinoco River Railroad Tunnel	Venezuela		●	●	●			●		●								
1976-78	Harvard Square Station (MBTA, Boston)	Massachusetts				●			●				●	●					
1976	South Cove Tunnel, Phase 2 (MBTA, Boston)	Massachusetts		●											●				
1976-77	National Waste Terminal Storage Project (ERDA Oak Ridge)	Tennessee	●							Nuclear Waste	●	●							
1976-78	Positron-Electron Project, Stanford Linear Accelerator Center	California		●		●				Physics Research			●	●		●			
1977	Detroit Railroad Tunnel	Michigan Ontario		●	●	●			●		●	●							
1977-81	San Francisco Southwest Ocean Outfall Project	California		●	●	●				Waste Water			●	●					
1977	Miami Transit System, Miami River Crossing Study	Florida			●	●			●		●								
1977-78	Underground Nuclear Power Plant Study	California	●							Nuclear Power		●							
1977-78	Conceptual Designs – OWI Geologic Storage Facilities	Various	●							Nuclear Waste	●	●	●						
1977-82	Second Downtown Tunnel (Norfolk-Portsmouth)	Virginia			●	●		●					●	●					
1978	North Terrace Pedestrian Tunnel, Adelaide	Australia				●				Pedest'n					●				
1978-82	Ft. McHenry Tunnel, Baltimore	Maryland			●	●		●					●	●		●			
1978-79	Fells Point Tunnel, Baltimore	Maryland			●			●			●	●							
1978	Miami River Highway Tunnel	Florida			●	●		●			●	●							
1978	Brisbane River Tunnel	Australia			●	●		●			●	●	●						
1979-82	Mt. Lebanon Tunnel, Pittsburgh (NATM Demonstration)	Pennsylvania	●						●				●	●					
1979	MUNI Turnaround, San Francisco	California				●			●						●				
1979-82	Washington Metro, Section F-4 (Anacostia River Tunnel)	Dist. of Columbia		●	●	●			●					●					
1979	Sydney Harbor Tunnel	Australia	●		●	●		●			●								
1980	Bangkok Tunnel	Thailand			●			●				●	●						
1980	Red Hook Tunnel, Brooklyn	New York		●					●		●	●				●	●		
1980-82	Westway	New York				●		●							●				
1980	Kaohsiung Harbor Tunnel, Taiwan	Taiwan			●			●			●								
1980	Detroit LRT Tunnels	Michigan		●		●	●		●		●								
1980-82	Glenwood Canyon Tunnels	Colorado	●					●			●	●	●	●					
1981	Caracas Metro – Caricaou Line	Venezuela	●	●		●	●		●			●	●						
1981-82	Mt. Lebanon Tunnel, Pittsburgh (Conventional Design)	Pennsylvania	●			●			●				●	●					
1981-82	Rogers Pass Tunnel	British Columbia	●	●		●			●				●	●					
1982-	Caracas Metro–La Paz-Plaze Italia	Venezuela		●		●	2		●						●				
1982	Los Angeles Transit – Wilshire Line	California	●	●		●	●		●				●						

W. THOMAS MEREDITH
President, Parsons Brinckerhoff Construction Services, Inc.
Technical Director of Construction Management
Principal Professional Associate

Education
College of William and Mary
Catholic University
University of Maryland

Societies
Society of American Military Engineers
American Institute of Plant Engineers

License
West Virginia

Mr. Meredith has extensive experience in the management of all types of construction projects. He is responsible for directing the worldwide business and technical activities of Parsons Brinckerhoff in the field of construction management.

Prior to assuming his current position, he served as project manager for the Philadelphia Center City Commuter Rail Connection, a 1.7-mile, four-track commuter link that will combine approximately 560 miles of the former Penn Central and Reading Railroads into one integrated system. A major element of the project is a modern transit station above which will rise a shopping mall and office towers. Parsons Brinckerhoff, in joint venture with two other firms, is providing comprehensive construction management services to the City of Philadelphia in building the 80 percent federally funded project. The $308 million connection is expected to begin revenue service in January 1984.

Previous Experience
As vice president for engineering/construction and assistant general manager of the Tumpane Company in Al Khobar, Saudi Arabia, he was responsible for the direction and implementation of the engineering and construction management of the Peace Hawk V Housing and Community Support Program in Saudi Arabia. This program encompassed the design and construction of three separate, but integrated, communities. The project involved complete site development, including modern water, sewage, electrical, telephone, and cable television systems.

Mr. Meredith served as vice president of a consulting engineering firm in New York City immediately following his retirement as a Brigadier General from the U.S. Air Force in 1973. During a military career that spanned 30 years, he held positions with both the U.S. Army Corps of Engineers and the U.S. Air Force. His responsibilities included:

- The operation, maintenance, and repair of the entire Department of Defense real property inventory, valued at $14 billion, as well as management of the 25 million acres of real estate; management of an annual expenditure of $3.5 billion and a work force of 250,000 persons, encompassing all Army, Navy, Marine and Air Force installations worldwide. Acted as principal for the development of life cycle construction costing and value engineering for the Department of Defense.

- The direction of the entire civil engineering effort encompassing the operation, maintenance, repair, design, and construction of a major Air Force Command comprised of 20 major Air Force installations worldwide with a 6,800-man work force and an annual operating and construction budget of $55 million.

- The post of commander of the USAF Combat Engineering Force supervising the construction in Vietnam of four major air bases, major expansion of eight existing air bases, directing aircraft shelter and realignment programs as well as air base, road, and bridge damage from enemy action.

- The development of concepts that provided two Air Force combat engineering forces: PRIME BEEF and RED HORSE; also responsible for implementing these concepts and for the realignment of the entire Air Force civil engineering force worldwide, including the establishment of training and career development programs and direction of the organization, training, equipping, and force deployments; also developed modular construction design concepts for total base facilities.

- The management of the USAF operations and maintenance function, encompassing a real property inventory of $16.1 billion, with an annual cost of $975 million and employing 110,000 people; developed management systems, engineering performance standards, preventive maintenance and work force control systems and techniques, and cost evaluations.

- The design and construction of facilities for all U.S. Forces in the United Kingdom; program varied from complex communications and missile facilities to family housing, with a working value of $250 million; direct responsibility and supervision of 140 professional engineers.

LOUIS G. SILANO
Vice President
Principal Professional Associate
Structures Division Manager
Structural Engineer

Education
Columbia University, B.S.C.E., 1951; M.S.C.E., 1955

Societies
American Society of Civil Engineers, Fellow
Tau Beta Pi, Alpha Phi Delta
Column Research Council

Licenses
New York, Rhode Island, Virginia, Georgia

Since he joined the firm in 1951, Mr. Silano has engineered and managed many complex multidisciplinary projects. As manager of the Structures Division and deputy technical director for major structures, Mr. Silano is responsible for structural projects including design of tunnels, movable, fixed and long-span bridges, bridge rehabilitation, port structures and mass transit structures.

Mr. Silano has held major responsibility for the following notable engineering projects:

Tunnels
• Project manager and project engineer for the Second Hampton Roads Bridge-Tunnel Crossing, including a 7,000-foot-long immersed tube tunnel, two ventilation buildings constructed on man-made islands, 1,500 feet of open approaches, and 8,800 feet of trestle approaches connecting Hampton and Norfolk, Virginia.

• Project manager responsible for the preliminary engineering design of a 2,300-foot immersed tube tunnel, including ventilation buildings, for a crossing of the Chao Phya River in Bangkok, Thailand.

• Project manager responsible for the preliminary engineering design of a high-level bridge and sunken tube tunnel alternate for a crossing of Bolivar Roads in Galveston, Texas.

• Project engineer for the preliminary engineering design of a 2,100 foot sunken tube tunnel including ventilation buildings, for Project One in Kakogawa, Hyogo, Japan for Kawasaki Heavy Industries, Ltd.

• Principal-in-charge of the design of 1,700-foot-long water intake tunnel to be constructed by the sunken tube method in Bahrain, in the Persian Gulf, for Hyundai Construction Co. Ltd.

• Participated in the design efforts for the following immersed tube tunnels: 63rd Street Tunnel, New York City; Second Downtown Elizabeth River Tunnel between Portsmouth and Norfolk, Virginia; Brisbane River Tunnel Crossing in Brisbane, Australia; and the Trans-Bay Tubes in San Francisco, California, part of the Bay Area Rapid Transit System.

• Participated in the design efforts for the following projects, all of which included bored tunnels through rock: subway section design for the Washington, D.C. Metropolitan Area Transit Authority's Metro; water circulating tunnels at the Tusi Power Plant in Texas; and a tunnel boring machine feasibility study for the New York City Transit Authority.

Bridges
• Project manager for the Fremont Bridge, Portland, Oregon. Responsible for engineering of this project including the design and erection of both the foundation and the superstructure of the world's largest three-span stiffened tied arch. The Fremont Bridge is a double-decked, eight-lane structure having an orthotropic upper deck and a main span 1,200 feet long. The structure received worldwide acclaim for the unique methods employed in erecting the main span and was awarded the AISC Prize Bridge Award in the long-span category for 1974.

• Project engineer for the Newport Bridge, Narragansett Bay, Rhode Island. Responsible for the engineering design and preparation of the contract plans for New England's largest suspension bridge. This 1,600-foot suspension span employs many novel features unique to suspension bridges. Among these are shop-prefabricated parallel-wire strands, pipe-frame anchorages, plastic cable wrap and all-welded steel towers.

Mr. Silano's experience also includes participation in design of many large bridges, including the Arthur Kill Bridge, Staten Island, New York; Savannah River Crossing, Georgia; Fleming Park Bridge, Pittsburgh, Pennsylvania; Myrtle Avenue Overpass, Jacksonville, Florida; and the Prospect Expressway, Brooklyn, New York. Prior to joining the firm in 1951, he was employed by the Port Authority of New York and New Jersey.

Teaching Experience
The Cooper Union, New York—Strength of Materials Laboratory.

BIRGER SCHMIDT
Senior Professional Associate
Geotechnical Engineer,
Project Manager

Education
Danish Technical University, Civil Engineer, 1960
University of Illinois, M.Sc., 1962
University of Illinois, Ph.D., 1969

Societies
American Society of Civil Engineers
Danish Society of Civil Engineers
International Society of Soil Mechanics and Foundation Engineering
International Society of Rock Mechanics
American Underground Association
British Tunnelling Association
Project Management Institute

Licenses
California, Massachusetts, New York, FEANI (European Federation of Engineers)

Dr. Schmidt has over 20 years of experience, principally in soil and rock mechanics, foundation and underground engineering, and in project management.

He is at present project manager for the firm's efforts on the design of the Nuclear Waste Repository in Basalt for the Department of Energy. This multi-billion dollar facility is intended to receive spent nuclear fuel and nuclear waste from other sources for deep underground, permanent storage. It will include over 100 miles of storage chambers in basalt at a depth of some 3,700 feet, five access shafts, and elaborate surface facilities.

In addition to this effort, Dr. Schmidt contributes to national nuclear waste policy-making on thermal rock mechanics, waste retrievability, and exploration and monitoring methods and requirements.

Dr. Schmidt was the resident project manager for the Positron-Electron Project (PEP) at the Stanford Linear Accelerator Center. Parsons Brinckerhoff provided design and construction management services for the conventional facilities of this $78 million project, including 7,000 feet of tunnel construction; a number of large experimental halls, including one underground; landscaping, drainage, and roadworks; and electrical and mechanical systems.

For the firm's San Francisco office, Dr. Schmidt also:

- Completed alternatives studies and cost estimates for San Francisco's Richmond Transport Sewer Tunnel.
- Consulted and provided design review for San Francisco's Southwest Outfall Sewer, on soil mechanics, tunnel engineering and seismic effects, including complex liquefaction problems.
- Reviewed for BART a MUNI project involving construction of an underground track turn-around just above BART running tunnels in soft Bay mud.
- Provided consultation and quality assurance review on two State of Washington bridge projects, the Hood Canal Bridge and the West Seattle Bridge.

Dr. Schmidt previously headed the Geotechnical Department in the firm's New York office, responsible for quality control, management, administration and service procurement in geotechnical exploration, analysis, design and reports. Some of the projects on which he worked are as follows:

- Immersed tunnels: Second Hampton Roads Tunnel (full geotechnical services); Third Elizabeth River Tunnel (exploration services); Chao Phya Immersed River Tunnel, Bangkok (feasibility), and others. Toured Europe to study European construction practices.
- Bridges: Martin Luther King, Jr. Memorial Bridge, Richmond, Virginia (full geotechnical services); James River Bridge, Virginia; and Canje River Bridge, Guyana.
- Rapid transit: WMATA, Washington, Section F2 twin soil tunnels and station; Baltimore Rapid Transit, Lexington Market tunnels and station (both geotechnical and underground design services).

• Highways and highway structures: Turnpikes in New Jersey and Virginia; coastal road in Guyana; expressway feasibility in Singapore.

• Marine facilities: Marine terminals in Portsworth, Virginia; Belize, Grand Cayman Island; and Ponce, Puerto Rico; coal terminal in Superior, Wisconsin; dredge containment facility at Lorain, Ohio; shipyard facilities in Fiji and Denmark; and shore protection in Samoa.

• Nuclear waste storage: Savannah River Plant, Liquid Nuclear Waste Storage Facility, S.C., including extensive geophysical and deep boring exploration, and sophisticated rock mechanics analyses.

• Research: For the Department of Transportation, two research projects on soil exploration improvements and on improved methods of monitoring underground construction. At the University of Illinois, tunnel lining design research and related work. At the Danish Geotechnical Institute, research into fundamental soil characteristics and project oriented research and model studies.

Dr. Schmidt has also provided design and consulting or management services on such projects as tank farms, grouting applications, floating dry docks and shipways, deep and shallow foundations of many kinds, earth dams, slope stratification and retaining walls.

Dr. Schmidt has lectured at the University of Illionis, the Danish Technical University, and Stanford University.

Publications
Dr. Schmidt has published over twenty papers and other publications; a selection follows:

• Design Problems for Underground Nuclear Waste Disposal in Basalt, Rapid Excavation and Tunneling Conference, San Francisco, May 1981.

• With W. Grantz, Settlements of Immersed Tunnels, Journal of the Geotechnical Engineering Division, ASCE, Vol. 105, No. GT9, September, 1979, pp. 1031-1047.

• With G.W. Clough, Design and Performance of Excavations and Tunnels in Soft Clay—A State-of-the-Art Report, International Symposium on Soft Clays, Bangkok, 1977.

• Monitoring Soft Ground Tunnel Construction—Urban Transportation Tunneling—A Handbook of Rational Practice for Planners and Designers, Handbook prepared for Urban Mass Transportation Administration, UMTA-MA-06-0025-76-6, 1976.

• With C.J. Dunnicliff, An Engineering Approach to Monitoring the Performance of Soft Ground Tunnels During Construction, Rapid Excavation and Tunneling Conference, San Francisco, V.I., 1974., pp. 377-396.

• Exploration for Soft Ground Tunnels—A New Approach, ASCE, Engineering Foundation Conference: Subsurface Exploration for Underground Excavation and Heavy Construction, Henniker, N.H., 1974, pp. 84-96.

• With C.J. Dunnicliff, Construction Monitoring of Soft Ground Rapid Transit Tunnels, Report prepared for Urban Mass Transportation Administration UMTA-MA-06-0025-74-13 I and II, 1974.

• Settlements and Ground Movements Associated with Tunneling in Soil, Ph.D. Thesis, University of Illinois, 1969.

• With D.U. Deere, R.B. Peck, J.E. Monsees, Design of Tunnel Liners and Support Systems, Report of USDOT, OHSGT, Contract 3-0152, 1969.

MICHAEL J. ABRAHAMS
Manager, Structures Department
Senior Professional Associate
Structural Engineer

Education
Bowdoin College, B.A., 1960
Columbia University School of Engineering, B.S., 1963; M.S., 1964

Societies
American Concrete Institute
American Society of Civil Engineers
Structural Stability Research Council, Chairman Task Group 3, Beam Columns

Licenses
New York, Georgia, Washington

Recently appointed manager of the North Atlantic Region's Structures Department, Mr. Abrahams has participated in many bridge, tunnel, and building projects:

Recent Projects

- Project engineer directing preparation of design and specifications for a new powerhouse for the Groveville Mills hydroelectric plant, Beacon, New York. This 0.8-MW facility will also include a revised penstock and revision to the tailrace.

- Deputy project manager for Hood Canal Bridge, Washington — a 6,470-foot-long floating, prestressed concrete bridge with a 600-foot-long lift draw section to allow ship passage. Responsible for design of bridge superstructure as well as movable bridge operating machinery, electrical and mechanical designs, specifications, and technical supervision of three subconsultants.

- Project manager for recent investigation of collapse of Kemper Arena, Kansas City, Missouri. A large portion of the roof of this 17,600-seat arena collapsed during a violent storm. As project manager, directed firm's investigation of collapse including structural and hydrological analysis, wind tunnel testing, consultation with consultant, and report preparation. Also project manager for subsequent review of rebuilt arena; review resulted in letter report to client.

- Project engineer for two highway bridges in upstate New York — part of infrastructure improvements associated with the Prattsville Pumped Storage Project of the Power Authority of the State of New York.

- Design of several fixed and movable highway bridges: the Third Street (bascule) Bridge in Wilmington, Delaware; the James River (vertical lift) Bridge in Newport News, Virginia; and the Martin Luther King, Jr., Memorial Bridge, in Richmond, Virginia.

- Review of shop drawings and erection procedures for the Curtis Creek Bridge, Baltimore, Maryland.

- Inspection, preparation of reports, rating, and design of repairs for fixed and movable bridges. Projects include a 544-foot-long vertical lift railroad bridge, Buzzards Bay, Massachusetts; a 60-foot-long swing bridge, Milford, Delaware; and a 3,235-foot causeway with a 160-foot bascule span, Galveston, Texas.

- Design and consultation during construction work on a 3,000-car precast prestressed concrete parking garage project in White Plains, New York. The garage includes three buildings and a high-level bridge.

- Project engineer in charge of plans and specifications for the plaza roof of the main subway station in Atlanta, Georgia. The roof is a large precast, post-tensioned, concrete structure erected using segmental construction techniques.

- Design engineer for prestressed box beams on Congress Avenue Bridge, Austin, Texas.

- Project engineer in charge of design and detailing of tunnel portions of the Second Downtown Elizabeth River Tunnel between Norfolk and Portsmouth, Virginia.

- Supervision of design and preparation of drawings for the ventilation buildings and certain portions of the sunken tubes of the Second Hampton Roads Bridge/Tunnel crossing in Virginia.

- Design engineer, preliminary design of mushroom piers, Keehi Interchange, Oahu, Hawaii.

• Structural analysis of a waterfront station, Section F-2, of the Washington Metropolitan Area Transit Authority (WMATA) Subway System.

• Acted as consultant to a major insurance company regarding an investigation into the collapse of the Hartford, Connecticut, Coliseum roof.

• Member of a five-man team of bridge experts who toured six European countries to assess state-of-the-art of prestressed segmental concrete bridges. Project was sponsored by International Road Federation at request of Federal Highway Administration.

• Model studies conducted at the U.S. Corps of Engineers Experimental Station at Vicksburg, Mississippi, on the placing operations for the 63rd Street Tunnel in the East River, New York City.

• Review of erection procedures of the Fremont Bridge in Portland, Oregon. The erection included lifting the 6,000-ton center span 160 feet into place — the world's largest lift of this type.

• Participation in feasibility studies for various sunken-tube tunnels, high-level bridge crossings, and tracked air-cushion vehicle guideways.

Previous Experience

• Served with the U.S. Peace Corps in the Philippines as a Civil Engineer attached to a Philippine Government Agency. Directed the survey, design, and construction of self-help projects such as schools, water supply, and irrigation systems.

• Structural engineer with a major design-construction firm. Involved in the design of iron-ore processing plants and mines. Work included the design and detailing of ore storage buildings, reclaim tunnels, and a stressed skin conveyor support system.

Publications

• Coauthor, "Record Span for Record Lift — The Fremont Bridge," awarded first prize by James F. Lincoln Arc Welding Foundation, 1974.

• Coauthor, "A Report on the Design and Construction of Segmental Concrete Bridges in Western Europe—1977," U.S. Department of Transportation, Washington, D.C., 1978.

Awards

• Honor Award, New York Association of Consulting Engineers, 1982, Structural Design of Five Points Station Roof.

• Annual Award, Prestressed Concrete Institute, 1982, Five Points Station Roof.

AHMET GURSOY
Senior Professional Associate
Senior Structural Engineer

Education
Technical University of Istanbul, B.S.C.E.. 1952
Illinois Institute of Technology, M.S.C.E., 1956

Societies
American Society of Civil Engineers

Licenses
New York

Mr. Gursoy's experience in the design of tunnels and other underground structures spans over 20 years. Since he joined Parsons Brinckerhoff in 1956, he has managed and coordinated several complex multi-disciplinary design projects and studies. Recently he served as the engineering manager in charge of pioneering work for the preconceptual design studies of the nuclear waste disposal facilities in deep geological formations of salt, granite, basalt, and shale for the Office of Waste Isolation, Department of Energy.

Mr. Gursoy is currently the manager of Subsurface Facilities and Shafts for the Basalt Waste Isolation Program, being designed by Parsons Brinckerhoff in joint venture for Richland DOE. He supervises rock mechanics; geology; engineering for the mine, shafts, tunnels, shaft stations and head frames; and engineering for surface mine support facilities, including rock handling, mine operations and change house, and mine ventilation.

Mr. Gursoy's additional experience in underground facilities projects includes:

- Project manager in charge of the firm's participation in the design of the Fort McHenry Tunnel, a S600-million project to be constructed under Baltimore Harbor in Baltimore, Maryland. Design features of the federally-funded project include two large ventilation buildings for an eight-lane roadway; cut-and-cover sections; depressed open-approach sections; and two sets of twin, circular, sunken-tube sections. Mr. Gursoy managed the joint venture design office of up to 50 engineers and coordinated the design effort with six Baltimore subcontractors.

- Overall engineering manager for the conceptual design of the nuclear waste disposal facilities in deep geological formations. As engineering manager, he directed pioneering work in conceptual design studies and cost analysis for a matrix of four different waste types and four different geological media—basalt, granite, shale, and salt—for the Office of Waste Isolation, Oak Ridge, Tennessee. He also coordinated the work of 30 Parsons Brinckerhoff engineers with that of other consultants and the client. He devised special underground layouts in basalt and granite to expedite construction under the stringent schedule imposed.

- Project engineer responsible for the ventilation building and open approaches for the 7,000-foot-long sunken tube tunnel of the Second Hampton Roads Bridge-Tunnel Crossing in Virginia.

- Responsible for the final design of a bored subway tunnel under the Potomac River for the Washington (D.C.) Metropolitan Area Transit Authority (WMATA) system.

- Participated in work on the 63rd Street Tunnel in New York City, which is used by both the city subway system and the Long Island Rail Road.

- Assisted in design of a sunken tunnel ventilation building in Kakogawa, Hyogo, Japan.

- Was involved in the design of the ventilation building for a section of the Bay Area Rapid Transit (BART) system in San Francisco.

Mr. Gursoy also has extensive experience in the structural design of bridges, including:

- Supervised design for the preliminary study and cost estimate of the Bolivar Roads Crossing in Galveston, Texas, a 15,000-foot-long water crossing, and for the James River Bridge in Newport News, Virginia, a water crossing 2,400 feet long with a 450-foot main span lift bridge.

• Responsible for preliminary design of an orthotropic bridge carrying Interstate 295 over the James River in Richmond, Virginia.

• Directed the final design of the Fremont Bridge in Portland, Oregon (the world's longest tied arch bridge).

• Principal designer of the Newport Bridge in Jamestown, Rhode Island. Responsible for the preliminary and final design of the two-mile-long water crossing, which contains a 1,700-foot suspended main span, approach spans, towers, and cables.

Publications

• "Lateral Winds on Side Spans of Suspension Bridges," *Journal of Structural Division, ASCE,* 1968. Received "Best Paper Award" of 1968

• "Bosporus Bridge Spans Two Cultures," *Consulting Engineer*, December 1974

• "Conversion of Art and Engineering," *Consulting Engineer*, November 1970

Teaching Experience

Mr. Gursoy taught soil mechanics at the Illinois Institute of Technology in 1957.

ROBERT J. HILL
Assistant Vice President
Deputy Director — Construction Management
Manager of Construction Operations

Education
Manhattan College, B.C.E.

Socieites
The Moles — Education Committee
American Arbitration Association — Member Commercial Panel
American Public Transportation Association — Vice Chairman Construction Committee
Municipal Engineers Society, City of New York
Society of American Military Engineers
Archdioces of New York — Cardinal's Campaign Construction Committee

License
New York

Mr. Hill is deputy director of the Construction Division. He has had extensive experience in the design, estimating, and construction of mass-transit projects, major tunnels, marine structures, underground, transportation, and plant projects. His design and contract experience is complemented by his former employment in the heavy construction industry. Mr. Hill's assignments since joining the firm include:

• Project director for construction management of the New York City Department of Environmental Administration's Interim and Accelerated Sludge Management Plan, a $450-million program of adding new pollution control plants, and modifying existing ones, in order to comply with federal E.P.A. regulations requiring cessation of ocean dumping.

• As manager of the Washington Metropolitan Area Transit Authority Claims Services contract, he was responsible for developing WMATA defense of litigation arising from contractor claims from overall mass-transit construction.

• Project manager of design-construct contract for the $120-million Brisbane Downriver Crossing, Australia, which includes a major vehicular tunnel, ventilation buildings and equipment, approach structures, and other appurtenant buildings.

• Project manager for the Barnum House restoration, a renovation of an historic building for Section 8 housing and commercial space.

• Responsible for determining the overall program cost for the Waste Isolation Facilities Program, U.S. Energy Research and Development Administration. Work involved the cost estimating and expert testimony for Congressional appropriations of multi-billion-dollar mined underground facilities throughout all suitable geologic formations in the United States.

• Project manager of design of Massachusetts Bay Transportation Authority (MBTA) project to extend a rapid transit line in the Greater Boston metropolitan area including passenger stations and parking garage.

• Responsible for the design of the MBTA South Cove Tunnels in soft ground.

• Assistant project manager for design of Second Downtown Tunnel Project, Norfolk, Virginia, and for design during construction of Second Hampton Roads Bridge-Tunnel. These projects were major sunken-tube type tunnels and included ventilation, administration, and other support buildings as well as the associated mechanical and electrical equipment.

• Involvement in Fort McHenry Tunnel Project in Baltimore, Maryland.

• New York office project manager for the Pittsburgh Light Rail Transit System.

• Project engineer for conversion of submarine berthing facilities from conventional to nuclear fleet use.

Previous Experience
• President of a construction firm involved in heavy marine and underground construction.

• Contracts administrator for a consortium of international constructors engaged in the construction of Water Tunnel No. 3, New York City. This was the largest tunnel construction project performed and involved 13 miles of deep tunnel, 17 shafts, and three major underground chambers. His responsibilities included administration of the prime and subcontracts. As part of his responsibility, he was responsible for placement and utilization of disposal materials in associated land development projects in the Greater New York metropolitan area, as well as involvement in preparation of major litigation for the joint venture.

- Project superintendent for an international firm where he was responsible for the following projects: Cross Bay Bridge, which included the erection of the largest precast, prestressed concrete girders to date; Port Elizabeth, a major containerport; and Bowline Point Power Plant. Involved in the construction of the 63rd Street Tunnel Project. Head estimator bidding on major projects throughout the eastern United States.

- For New York City Department of Public Works, served as section engineer in charge of sewer construction for a major portion of Queens County. Previously, involved for the Department in construction of Contract 4B, North Branch Intercepting Sewer, Newtown Creek PCP, and Hunts Point Pollution Control Project.

- Employed by major contractors on large urban highway, bridge, miscellaneous building, and underground construction.

GEORGE A. MUNFAKH
Professional Associate
Head, Geotechnical Department

Education
University of Aleppo, Syria, B.S. 1967
Louisiana State University, M.S. 1970; Ph.D. 1973

Societies
American Society of Civil Engineers – Member of the National Committee on Placement and Improvement of Soils
Transportation Research Board – Member of the Committee on Soil Stabilization
International Society of Soil Mechanics and Foundation Engineering
Chi Epsilon, Phi Kappa Phi

License
Syria

Dr. Munfakh is responsible for the firm's geotechnical work in New York and other regional offices. In this capacity, he directs geotechnical investigations, planning, design, and construction services for projects undertaken in virtually every engineering field. These include marine structures such as ports and offshore contained disposal facilities; subsurface facilities such as mass transit systems and tunnels; and surface facilities such as bridges, highways, buildings, material handling facilities, and airports.

Specific examples of his experience are:

Marine Facilities

• Directed the geotechnical investigations and design of the Fort McHenry Tunnel in Baltimore, and the related dredge disposal facility at Canton/Seagirt. Assisted in the evaluation of several sites, as well as the engineering for the Canton/Seagirt Facility.

• Supervised the geotechnical design of the Jourdan Road Terminal Project in Louisiana which includes the use of a stone column-reinforced earth system behind a pile-supported deck. This innovative design was tested through a prototype field failure test designed and conducted by the Geotechnical Department. Supervised preparation of back-up areas, including the use of sand and prefabricated wick drains.

• Performed all geotechnical investigations and design related to the Lorain, Ohio offshore contained dredge disposal facility, including determination of the consolidation properties of the disposed material and the influence on the size of the facility.

• Supervised the design of a containment dike for dredge disposal materials at the Portsmouth Naval Shipyard in New Hampshire. Assisted in the preparation of the earth fill contract.

• Reviewed geotechnical work performed for the Westway Project in New York, including the prototype earth fill contract.

• Provided geotechnical investigations and design for the Portsmouth Marine Terminal in Virginia, the Port of Suez in Egypt, and the Jeddah Port Complex in Saudi Arabia.

• Directed geotechnical design of an offshore port in Belize, Central America, including a man-made island and access trestle supported on 54-inch precast, prestressed concrete cylinder piles installed in difficult limerock formations. Supervised vertical and horizontal pile load test programs including instrumentation of test piles.

• Planned site investigation in the Samoa Islands; designed seawalls, breakwaters, and revetments; and conducted an environmental impact study including cost and damage estimates.

Subsurface Facilities

• Directed geotechnical investigations and design of several trench tunnels including the Second Downtown Elizabeth River Tunnel in Virginia; the Fort McHenry Tunnel in Baltimore, Maryland; and the Char Phya River Crossing in Bangkok, Thailand.

• Senior geotechnical engineer for a Baltimore Rapid Transit System project. Responsible for the exploration program, geotechnical analysis, and design recommendations for the Lexington Market Tunnel Section.

• Provided geotechnical services for the following rapid transit projects: WMATA F-2b Station in Washinton, D.C.; South Cove Tunnel in Massachusetts; WMATA L-1 in Washington, D.C.; and Harvard Square Station in Cambridge, Massachusetts.

• Geotechnical group supervisor for the underground strategic petroleum reserve project in Louisiana and Texas.

Surface Facilities

• Led all soils investigations, analysis, and recommendations for the master plan of Sadat City in Egypt and the final design required for Phase I implementation of the plan. Also, responsible for geotechnical recommendations involving master plan preparation for Kano University, Nigeria.

• Directed the subsurface exploration program and geotechnical design for bridges and highway embankments of the Governor Driscoll Expressway in New Jersey and I-495 in Massachusetts. Also, performed feasibility studies for the Nacote Creek Bridge in New Jersey including several fixed and movable alternatives.

• Directed the geotechnical investigations and design of several facilities at the Albany County Airport in New York including terminal building extension, air cargo building, runways, and access roads. Poor soils were stabilizaed by vibroflotation.

• Reviewed all geotechnical studies performed for the Cerrejon Coal Project in Colombia including mine facilites, coal handling systems, port facilities, and 150 kilometers of railroad and port-mine road with 26 bridges including one major crossing consisting of 20 spans.

• Provided geotechnical services for several material handling facilities including the St. James Coal Terminal in Louisiana, the Detroit-Edison Coal Unloading Facility in Michigan, the Chesapeake Coal Pier in Virginia, the Cleveland Iron Ore Pellet Terminal in Ohio, and the Dundee Cement transshipment facility in Louisiana.

Previous Experience

As a research associate at Louisiana State University, Dr. Munfakh headed research projects dealing with densification and stabilization of organic soils. He also served as a foundation designer with consulting engineers in Syria.

Publications

Dr. Munfakh's publications on improvement of soft soils include:

• "Stone Columns at Jourdan Road Terminal, Port of New Orleans." Paper presented at the American Association of Port Authorities Seminar on Environmental Planning and Engineering at Seaports, 1981.

• "The Effects of Densification on the Engineering Characteristics of Organic Soils," *Engineering Research Bulletin* No. 113, Vols. I and II, Louisiana State University, 1973.

• "Lime Stabilization of Organic Soils," *Highway Research Record* No. 381, Highway Research Board, 1972.

• "Stabilization of Organic Soils with Lime" (co-author), *Engineering Research Bulletin* No. 103, Louisiana State University, 1970.

• "Geotechnical Aspects of the Second Downtown Elizabeth River Tunnel." Paper presented at the Thirteenth Annual Southeastern Transportation Geotechnical Conference, Virginia Beach, Virginia, 1981.

Awards

Michael Claus Memorial Award for Excellence in Research, 1973.

WALTER C. PARISH
Civil/Structural Engineer

Education
Georgia Institute of Technology, B.S.C.E., 1957
College of William and Mary, Graduate Studies

Licenses
Maryland

Mr. Parish's experience includes design for support systems for underground and open-cut excavating; support for buildings, utilities, railroads, highways, streets, and spans; and consultant to contractors. Currently Mr. Parish functions as administrator of geotechnical services for the Pittsburgh Light Rail Transit System. He is project engineer for the Mt. Lebanon Tunnel portion of the LRT and supports the structural department in soil loading and bearing values in the department's design of box and station structures. He also serves as the contract administrator for building protection in the central business district.

Other projects to which he has brought his skills include:

- Design of support systems for underground and open-cut excavations as well as support for buildings, utilities, railroads, highways, and streets in the Washington, D.C. area.

- Project engineer for construction projects in Maryland, South Dakota, Georgia, and Puerto Rico ranging in value to $10 million. These include: marine construction at Calvert Cliffs, Maryland; nuclear plant and construction of channel, harbor, and landfill for Sun Oil petrochemical complex, Yabucoa, Puerto Rico.

- Responsibility for engineering of various projects, including construction of four artificial islands and excavation and backfill of tunnel trenches for the Chesapeake Bay Bridge-Tunnel Crossing. Enlarging and deepening portions of the Chesapeake and Delaware Canal near Summit, Delaware.

- Management of shop fabrication scheduling for most economic use of steel shops required for structural units of buildings and bridges.

- Design of embankment support, temporary bridges, and support piers for three high speed AMTRAK to N.Y. rail lines at New Carrollton Metro Station.

- Participation in the design of underground support systems on about 50 percent of all Washington, D.C. Metro contracts, involving cut-and-cover construction. Designed support system passage of Metro subway under Rock Creek. Designed several re-support systems for contractors on D.C. Metro where failure had occurred in existing system. Designed rock support systems for open cuts at Metro's Cleveland Park, Van Ness, and Bethesda Stations. Designed bridges, support piles, embankment, and utility duct support for two Conrail lines, leading from D.C. to Virginia, at the intersection with the Metro structure.

- Design of pipe insertion cofferdam for L.N.G. terminal at Cove Point, Maryland.

- Design of plug and 4-pipe hydraulic bypass for existing 18-foot square sewer tunnel in Chicago, Illinois.

- Design of cofferdams for construction of abutments for B&O Railway bridge over future extension of Maryland Route 28 in Rockville, Maryland.

- Design of decking and temporary bridges for street and highway support over open-cut excavations throughout the Washington Metropolitan area.

- Numerous analyses and independent studies made in conjunction with other designs.

YALCIN TARHAN
Senior Professional Associate
Senior Supervising Engineer

Education
Robert College, B.S.C.E., 1958
New York University, M.S.C.E., 1961

Licenses
Georgia, New Jersey, New York, Virginia

Mr. Tarhan is a structural engineer who has managed or performed detailing, estimating, and design for such varied projects as dams and energy-related structures, flood-control structures, bridges and tunnels, and varied types of buildings.

His projects include:

- Project manager for the underground petroleum storage site at Bryan Mound, Texas, as part of the Strategic Petroleum Reserve Program of the U.S. Department of Energy. Storing petroleum in caverns mined from salt deposits required providing for the environmentally safe handling of large quantities of petroleum, water for solution mining, and salt brine. The project included design- and construction-phase architecture/engineering services for pump foundations and for water intake structures, with all associated appurtenances. These included an innovative fish ladder, motor control center buildings, oil separators, open water channels, division channels, brine disposal pipe lines, levee penetrations, sheet-piling walls, and dikes.

- Structural engineer responsible for recommendations of structural measures to improve flood control protection in the Wyoming Valley of the Susquehanna River, Pennsylvania, including dikes, levees, sea walls, and other structures.

- Designer of flood control structures, including walls and sheetpiling walls, as part of the Saw Mill River flood control project, Yonkers, New York.

- Project manager of a sunken tube tunnel project under the Elizabeth River between Norfolk and Portsmouth in Virginia. The project has eight double plate steel tubes, cut-and-cover cast-in-place concrete sections, ventilation building and pump rooms, sea walls, and sheetpile bulkheads. The project is under construction, and Mr. Tarhan coordinates Parsons Brinckerhoff field personnel as well as all design department efforts.

- Project engineer of a complex, 3,000-car garage, which is part of a development in downtown White Plains, New York. The project has two underground levels involving deep foundation in addition to six aboveground floors. Mr. Tarhan coordinated work between the developer's engineers and architects as well as other departments of Parsons Brinckerhoff.

Previous Experience
Prior to his employment with Parsons Brinckerhoff, Mr. Tarhan participated in the design of earth fill dams, spillways, valve control structures, and water inlet structures for contractors performing turn key projects for the Turkish State Department of Water Works.

KANG HUANG
Civil Engineer

Education
National Taiwan University, B.S., Civil Engineering, 1959
Oklahoma State University, M.S.C.E., 1963
Columbia University, Engineer Mechanics, 1971

Societies
American Society of Civil Engineers

Licenses
Massachusetts, New York, Florida
Office of Civil Defense Fallout Shelter Analyst and Protective Construction Designer

Mr. Huang brings to Parsons Brinckerhoff 20 years experience in civil engineering, with particular emphasis on structural design and computer applications.

For Parsons Brinckerhoff, he has served as:

- Project engineer for the Mt. Lebanon Rock Tunnel in Pittsburgh, Pennsylvania. Responsible for the design of a 2,500-foot, twin single-track concrete-lined tunnel and two ventilation structures for Pittsburgh's new light rail transit system. The two design alternatives include a fully concrete-lined tunnel design constructed by conventional drill-and-blast methods and a New Australia Tunneling Method (NATM) design utilizing a shotcrete lining.

- Project engineer for the Anacostia River Tunnel for the Washington Metropolitan Area Transit Authority (WMATA). Responsible for alternative designs of either a trench tube with cut-and-cover box tunnel sections or a shield-driven tunnel utilizing compressed air or earth pressure balancing construction techniques.

- Senior engineer for the Second Downtown Elizabeth River Tunnel in Norfolk, Virginia. Responsible for the design of the immersed tube sections of this tunnel.

Previous Experience
As project design engineer for the New York City Transit Authority responsible for a major new subway route, his work included stress analyses of rock-tunnel lining and design of subway stations, underground structures, railroad alignments, ventilation shafts, steel structures, concrete arches, and reinforced concrete. He also developed, for all design purposes, computer programs involving finite element and matrix methods in the analysis of structures, soil mechanics, rock mechanics, alignment geometry, and clearance.

Mr. Huang's previous experience also includes:

- Supervising engineer for the Miami subway system. In charge of developing and designing prestressed elevated special structures.

- Project manager for sections of the WMATA subway system which include a rock tunnel with underground stations and a soft-ground tunnel with above-ground station and prestressed concrete bridge.

- Senior engineer responsible for stress analysis of a thin-shell prestressed concrete containment for a nuclear reactor under static and dynamic loads using a finite element computer program.

- Civil engineer for the New York City Department of Parks, responsible for field investigations of structural failures. Recommended remedial procedures for restoration and renovation of existing structures and supervised construction of buildings, sewers, and retaining walls.

- Senior civil engineer in charge of development and design for portions of the Boston Rapid Transit System. Responsible for all aspects of structural design and preparation of contract plans for subway tunnels, buildings, bridges, and special structures.

- Senior civil engineer, responsible for developing foundations for major structures, including underpinning; pile design and testing; soil-bearing capacity and pressure analysis; and interpretation of soil testing data. Also designed sewage and storm drainage systems and developed special computer programs for theoretical analysis and practical design of civil and structural engineering projects.

- Structural engineer responsible for the geometric layout of highway vertical intersections and for the design of highway and railroad bridges in steel, reinforced and prestressed concrete; earth-retaining structures; sheeting and bracing cuts; alteration of existing structures; and small dams and culverts.

NACHUM SECKER
Senior Structural Engineer

Education
Technion (Israel Institute of Technology), B.S.C.E., 1951
Massachusetts Institute of Technology, Fulbright Scholar, 1955

Licenses
New York

During his 30 years as a structural engineer, Mr. Secker's responsibilities have encompassed all phases of project management and cost estimation for the design and construction of heavy industrial projects including nuclear facilities such as power plants and waste isolation facilities. Since joining the firm, Mr. Secker has worked on several major projects including the design of a deepwater berthing facility at Adabiyah, Egypt, for which he served as project structural engineer. A part of the rehabilitation of the Port of Suez, the design utilizes prestressed concrete cylinder piles and cast-in-place reinforced concrete deck.

Mr. Secker's recent responsibilities include:

- Project bridge engineer for the Sudan Railway Rehabilitation and Modernization study. His work on this study required many visits to Sudan, as well as his being stationed there for a period of approximately 18 months. He traveled extensively over the entire Sudan Railway network, consisting of more than 5,500 kilometers of track, and inspected approximately 500 bridges of varied types and spans. The project construction cost estimate was over $70 million.

- New York project manager of an international joint venture for a comparative study of a bridge or tunnel crossing of the Chao Phya River in Bangkok, Thailand, with a construction cost estimate of over $170 million.

- Project engineer for a feasibility study for delivery of coal and cooling water to a fossil power plant in the Great Lakes are for Detroit Edison Company, and for a proposal to develop a coal transhipment and processing facility in the Hampton Roads area in Virginia.

In addition, Mr. Secker was responsible for the coordination and development of cost estimates and nuclear-related aspects of the design of surface facilities for the following projects:

- Preliminary conceptual design of a multi-million-dollar waste isolation facility for Union Carbide's Office of Waste Isolation and the U.S. Department of Energy. Project included costing 17 different alternatives as to the geological host media (salt, shale, granite, and basalt), the nuclear fuel waste cycle, and alternative waste packaging. Design of the canistered waste (remote-handling) receiving buildings, the low-level (drum-handling) receiving building, the radioactive waste building, the control building, and all other special facilities.

- Preparation of a joint description and study of nuclear waste depositories with western European countries for the International Nuclear Fuel Cycle Evaluation (INFCE) in Vienna, Austria. These descriptions covered different reactor strategies for light-water reactors, heavy-water reactors, and fast-breeder reactors for spent fuel. The estimated cost for capital construction for the INFCE depository was $553 million, and for one year of operation the cost estimate varied from $90 to $105 million, depending on the assumed annual installed nuclear power capacity and the reactor strategy used.

- A study of the feasibility, methods, and costs of exhuming highly radioactive wastes from a disposal site at West Valley, New York, for the U.S. Department of Energy. The total costing of $20 million for the exhumation of approximately 46,000 cubic yards of contaminated earth involved careful consideration of the hazards involved and of unique enclosure construction and handling.

Prior Experience
Mr. Secker's background includes positions as chief structural engineer and head of the structural department of an engineering firm specializing in industrial work. He has also been a supervising structural engineer responsible for the costing and design of NASA facilities for the moon project and for heavy industrial facilities. Major responsibilities included:

• Senior project engineer for a consulting engineering firm. Assignments included supervision of design and construction of bulk (coal and iron ore) materials handling facilities, iron ore mining concentration, and pelletizing facilities.

• Senior structural engineer for the Jamesport Nuclear Power Units Nos. 1 and 2. Responsible for design and engineering of the turbine and generator building, and water intake structures facility.

• Associate and chief engineer for a firm which provided consulting services to Bethlehem Shipbuilding and Steel Division for various waterfront and heavy industrial projects including a finger pier at Sparrows Point, Maryland, unique in its length, design, and methods of construction; mooring facilities for a dry dock at San Francisco, California; and shipyard facilities at Beaumont, Texas.

• For a consulting engineering firm, senior group leader responsible for commercial and residential design as well as special engineering projects.

• Chief engineer for a multidisciplined architectural engineering firm. Assignments included design and engineering of commercial facilities and several cement plants.

• Project engineer on military and nuclear projects for the Department of Defense, State of Israel.

• For a consulting engineering firm, structural designer of fossil fuel plants, paper mills, and varied waterfront projects.

VAHAN TANAL

Deputy Manager, Geotechnical Department
Senior Geotechnical Engineer

Education
Robert College, Istanbul, Turkey, B.S.C.E., 1969
University of Wyoming, M.S.C.E., 1971

Societies
American Society of Civil Engineers
International Society of Soil Mechanics and Foundation Engineering
The Society of American Military Engineers
Sigma Tau

As senior geotechnical engineer, Mr. Tanal serves as project manager for geotechnically intensive projects, and directs the planning, design, construction control and field inspection for tunnels, airports, ports and harbors, highways, oil storage facilities, bridges and other projects. As deputy manager of the firm's geotechnical department, he assists in administration, oversees project staffing and management of budgets and schedules. His most recent project responsibilities include:

- Construction consultations and shop drawing review of the dredged spoil containment site, for the Fort McHenry sunken tube tunnel in Baltimore, Maryland.

- Final design and preparation of the contract documents for the dredged spoil containment structure and related facilities, for the Fort McHenry Tunnel. The project consists of a 5,000-foot-long steel containment structure, enclosing about 120 acres of water area, and includes elutriate treatment facilities and settling basins.

- Dredged spoil disposal site selection and engineering feasibility studies for the Fort McHenry Tunnel. The studies investigated upland and waterfront sites in the greater Baltimore area.

- Geotechnical design and field investigations for the surface facilities at four underground oil storage sites in Texas and Louisiana, for the Department of Energy Strategic Petroleum Reserve Program. In this program, oil is stored in naturally formed and leached salt caverns several thousand feet below ground.

- Rehabilitation of the existing port, and master planning for a new port at Port of Suez, Egypt.

- Dredged spoil disposal facilities for the Portsmouth Naval Shipyard in Portsmouth, Maine.

Other responsibilities with Parsons Brinckerhoff included the geotechnical analyses of a man-made island for the Second Hampton Roads Bridge-Tunnel crossing; the design analyses for the l'Enfant-Pentagon Route and the Branch Route of the WMATA bored tunnels; retaining structures for the LIRR-JFK rail rehabilitation; geotechnical analyses for a port expansion in Ponce, Puerto Rico; and geotechnical explorations for the Canje River Bridge in Guyana.

Previous Experience
As a project manager with another major U.S. engineering firm, Mr. Tanal managed various site selection, waste disposal, port facilities, construction control, and plant expansion projects for oil, chemical, mining, metal processing and other industrial companies. Some of his major project responsibilities included:

- Retaining structures for red mud tailings storage for the Martin Marietta Alumina Plant in St. Croix, U.S. Virgin Islands.

- Containerport for Hess Oil Virgin Islands Corporation in St. Croix, U.S. Virgin Islands.

- Tank farm and future refinery for Hess Oil Virgin Islands Corporation in St. Lucia, West Indies.

- Proposed clean boiler fuel demonstration plant for COALCON in New Athens, Illinois.

- Tank farm and berthing facility for Metropolitan Petroleum Company in Jersey City, New Jersey.

• Venalum and Alcasa aluminum plants for Reynolds International in Puerto Ordaz, Venezuela.

• Organics plant expansion for Stauffer Chemical in Mount Pleasant, Tennessee.

Earlier, Mr. Tanal served as principal investigator in the design of a proposed floating nuclear power plant to be located 2.8 miles offshore of Atlantic City, New Jersey. In that capacity he was in charge of:

• Foundation, stability, and settlement analyses of the main breakwater, the closure breakwater and the mooring caissons.

• A study of commercially available geotechnical monitoring devices and their installation methods to assess their adaptability for use in the corrosive ocean environment.

• Planning and execution of an installation of piezometers in the ocean bottom and a telemetry data gathering system, to monitor wave-induced pore pressures in stratified subsoils.

• The development by the Charles Stark Draper Laboratories of a triaxial movement sensor to simultaneously monitor vertical and lateral deformations in foundation soils at inaccessible locations in the ocean environment.

Mr. Tanal's field engineering training included construction projects in Istanbul, Turkey and the Brussels Metro construction in Belgium.

Publications

Mr. Tanal is a coauthor of the technical paper, "Instrumentation for Wave Induced Pore Pressures," presented at the ASCE Specialty Conference, Ocean Engineering III, at the University of Delaware in June 1975. Paper was published in the conference proceedings.

CHU-PING TU
Senior Structural Engineer

Education
National Tsing-Hua University, China, B.S.C.E., 1939
Cornell University Graduate School, Courses in Structural Design and Architecture
Division of Physical Research, Bureau of Public Roads, Washington, D.C..
Advanced Studies in Soil Mechanics
M.I.T. Special Program in Coastal Wave Dynamics, 1976

Mr. Tu has over 38 years of professional engineering experience in research, planning, and design of structures. Since joining the firm in 1965, he has been responsible for the design and investigation of many major bridge piers and footings, tunnels, retaining walls, bulkheads, and waterfront and offshore structures.

His projects include several that have won awards or are generally acknowledged as innovative. For example, Mr. Tu was project engineer as well as principal structural designer of the Coal Transshipment Terminal at Superior, Wisconsin, a project which won three major awards — the Outstanding Civil Engineering Achievement Award for 1977 from the American Society of Civil Engineers, and the awards for design excellence from the National Society of Professional Engineers and the Consulting Engineers Council of New Jersey. He was also principal structural designer of the four main piers with footings for the award-winning Fremont Bridge in Portland, Oregon.

Mr. Tu's responsibilities on numerous other major projects include:

• Senior structural engineer responsible for the preliminary design of drydocks and piers for Trident Submarine Support Complex in Bangor, Washington.

• Senior structural engineer pioneering the design of a circular and eliptical breakwater in Jacksonville, Florida, for protecting offshore floating nuclear power plants against hurricane waves and impinging ships.

• Consultant responsible for the piling layout and deck structure of the new pier extension in Adabiyah, Egypt.

• Project engineer and principal designer of the car positioner building and car dumper pit in the Iowa Gateway Terminal along the Mississippi River at Keokuk, Iowa.

• Consultant responsible for the piling layout and deck structure of the ARDM mooring and utility pier with trestle approach in Kings Bay, Georgia.

• Senior structural engineer responsible for the investigation and design of caissons, cofferdams, formworks, and procedures for the construction of all the substructures (footings and piers) for the Newport Bridge, Rhode Island.

Other Projects
Mr. Tu served as principal structural designer on the following:

• Twin tunnel across the Potomac River, its ventilation shaft and pumping station, as well as the underpinning of existing buildings for the subway construction in Washington, D.C.

• Offshore ventilation structure caisson linking the Trans-Bay Tunnel and the subway tunnel in San Francisco, California.

• Open approaches of the Second Hampton Roads Bridge-Tunnel crossing in Virginia.

• Bulkheads and retaining structures for the construction of the Elizabeth River Second Downtown Tunnel in Virginia.

• Abutments and bulkheads for the Parsonage Creek Bridge, Long Island, New York.

• Seawall with adjoining waterfront roadway and bridge pier in Albany, New York.

• Forest Park Tunnel and approaches for extending the Long Island Rail Road's service to J.F. Kennedy International Airport in New York City.

Previous Experience
Before joining Parsons Brinckerhoff, Mr. Tu was associated with a consulting firm in New York, in charge of the planning, and preliminary and final design of over 30 waterfront facilities including bulkheads and piers in shipyards and terminals.

DANIEL J. WALLACE
Professional Associate
Senior Structural Engineer

Education
City College of New York, B.S. in Architectural Engineering, 1961

Societies
American Society of Civil Engineers — Chairman of Metropolitan Section
Transportation Group Executive Committee

Licenses
District of Columbia, New York, South Carolina

Mr. Wallace, a professional associate, is a chief transportation structural engineer and a senior project manager in the Atlantic Region of Parsons Brinckerhoff. He has significant experience in the design of trench, rock, earth, and mixed-face tunnels using concrete, steel, and shotcrete. His experience also includes the administration and coordination of design, staging, cost estimating, and development of construction operations for major transit projects. The transit structures which he has space-proofed, conceptually developed, and designed include: passenger stations, ventilation shafts, traction power substations, and pumping stations.

Mr. Wallace has engineering experience in all major modes of rail rapid transit: light rail transit, heavy rail transit and railroads.

His current projects for the firm include:

- Project manager for the final design of WMATA Branch Route Section F4 (Metro's new tunnel under the Anacostia River, Washington D.C.). Contract documents are being prepared to construct this crossing by use of either trench tube, mined tunnel or cast-in-place concrete structures built within cofferdam structures.

- Manager for transit planning and engineering for the Baltimore, Maryland North Corridor/Metro Center Transit Alternatives Analysis Study, which includes light rail and exclusive busway.

- Project manager for the final design of the Pittsburgh, Pennsylvania light rail transit system's Mount Lebanon tunnel. Contract documents are being prepared to construct this rock tunnel by either drill and blast techniques or by tunnelling machine.

Mr. Wallace has contributed his administrative and design skills on these major transit projects:

- Task manager for conceptual and preliminary engineering studies for the Pittsburgh, Pennsylvania light rail transit system's Mount Lebanon tunnel, Mount Washington tunnel and Central Business District Underground Alternatives.

- Project manager for the construction of the Washington Metropolitan Area Transit Authority (WMATA) system's Branch Route Section F2 tunnels and passenger station.

- Project manager for design/construction services for the Baltimore Rapid Transit system's Lexington Market subway station and tunnels.

- Deputy project manager for the final design of WMATA Section F2 consisting of twin subway line tunnels (earth) and a waterfront station complex (cut-and-cover construction).

- Deputy project manager for the final design of the Baltimore Rapid Transit Lexington Market section tunnels (earth) and passenger station (cut-and-cover construction) in Baltimore, Md.

- Senior structural engineer for the final design of Section C-4 of WMATA, which consists of twin subway tunnels under the Potomac River (rock, mixed-face, and earth).

- Senior structural engineer for the studies, final design and construction for the 63rd Street Tunnel under the East River (4-track, 2 level, trench tube) in New York City for use by both the New York City subway and Long Island Rail Road.

- Structural designer for the Trans-Bay Tube in San Francisco for the Bay Area Rapid Transit (BART) system.

- Structural designer for the Metropark Railroad Station, Woodbridge, New Jersey.

• Study of the feasibility of using tunnel boring machines for construction of Route 131-A (rock tunnels) of the New York City subway system.

Transportation projects in which Mr. Wallace participated in the 1960's include:

• Senior structural designer for Long Island Rail Road modernization studies.

• Extensions of the New York City subway system, East River Tunnel, and Cross Brooklyn Expressway, all in New York; Shawmut Tunnel, Boston, Massachusetts; and I-93, Franconia Notch, New Hampshire.

Mr. Wallace has served as structural designer of several building projects. These include: the Norad Combat Operations Center Defense Facility, Colorado; Atlantic City Expressway Toll and Administration Buildings, New Jersey; and Bloomingdale's Furniture Store, Bergen County, New Jersey.

Before joining the firm, Mr. Wallace participated in engineering studies, estimates of future functional needs, and preparation of design drawings for the John F. Kennedy, LaGuardia, Newark and Teterboro Airports for the Port Authority of New York.

WILLIAM KAM
Senior Structural Engineer

Education
University of Tennessee, B S.C.E. 1950; M.S.C.E. 1952
Columbia University, 1952-54, credited with class work toward Doctor of Engineering Science

License
New York

Mr. Kam is a structural design engineer with more than 24 years of professional engineering experience. Since joining the firm in 1962, he has had major responsibility in structural design, checking contract drawings and shop drawings for bridges of all types, both fixed and movable. Mr. Kam has worked on several of the world's largest tunnels and bridges.

Mr. Kam's projects for the firm include:

Tunnels
- Principal designer of tube joints, including all steelwork in the tubes, and jacking devices of the Fort McHenry Tunnel in Baltimore, Maryland. Mr. Kam developed wedges for the correction of tube alignments on this world's longest double-bore highway tunnel.

- Principal designer of the tube joints of the Al Khorbar water tunnel in Saudi Arabia.

- Principal designer for the cut-and-cover section and the joints between tubes for the Second Downtown Elizabeth River Tunnel in Virginia. He checked contract and shop drawings of the tubes and the approach bridges.

- Reviewer of shop drawings pertaining to structure adequacy of the Hong Kong Cross Harbor Tunnel.

- Principal designer of all steelwork in typical tube, typical joint between tubes, and other major steelwork, including the survey tower, for the second Hampton Roads Tunnel in Virginia.

- Principal designer of all steelwork in typical tubes, joints between tubes, and closure tube; and provided dynamic analysis for San Francisco BART Tunnel, the world's longest subaqueous railroad tunnel.

Bridges
- Principal structural and machinery design engineer for the renovations of the South Market Street Bridge in Wilmington, Delaware, the Saugatuck River Bridge in Connecticut, and the Kernwood Bridge in Massachusetts.

- Structure design engineer for restoration of the Hood Canal pontoon bridge in Seattle, Washington, the world's largest floating bridge. Mr. Kam also checked shop drawings.

- Design engineer on major bridge projects of varied types, including the James River vertical lift bridge and the Martin Luther King, Jr., Memorial Bridge, a curved steel continuous box girder bridge, both in Virginia; the Curtis Creek Bridge in Maryland and Third Street Bridge in Wilmington, Delaware, both double-leaf bascule bridges; the Cape Fear River vertical lift bridge in North Carolina; the Parsonage Creek prestressed concrete bridge in New York; the prize-winning reconstruction of the historic Congress Avenue Bridge in Austin, Texas; 28 prestressed concrete bridges of the Atlantic City Expressway in New Jersey; and miscellaneous composite stringer bridges of the Garden State Parkway in New Jersey.

- Principal designer of lower level and other major steelwork for the Fremont Bridge in Portland, Oregon, the world's longest two-decked tied arch bridge.

- Principal designer for the preliminary design and layout of suspended spans for the Newport Bridge in Rhode Island. He was also principal designer for final design of towers, plate girder, and simple truss spans, including miscellaneous major steelwork, and was in charge of final shop drawings for towers and trusses.

Other Major Structures
- Principal structure designer of two Bloomingdale Department Store buildings in New Jersey.

- Structural engineer for several hardened structures, including NORAD; Ft. Richie; and the American Telephone and Telegraph site in New York State.

Previous Experience
Before joining Parsons Brinckerhoff, Mr. Kam was associated with several nationally known consulting engineering companies. He participated in the design of many types of bridges and other structures:

- Movable bridges of all types, including the Delair Bridge in New Jersey, which is the world's third-longest-span vertical lift bridge.

- Suspension bridges, including the Walt Whitman in Pennsylvania, the Throgs Neck in New York, the Verrazano Narrows in New York, at the time the longest single suspension span in the world, and the lower level of the George Washington Bridge in New York.

- Long span cantilever and continuous bridges, including the Captree Bridge in New York and the Gold Star and Thames River Bridges in Connecticut.

- Other structures. Mr. Kam was the assistant project engineer for the bridge truss of the proposed radio telescope in Sugar Groves, West Virginia, and also has experience in the design of fossil-fueled power plants, industrial buildings, and school buildings.

'STANLEY MAGER
Structural Engineer

Education
City College of New York, B.C.E., 1947

Societies
American Society of Civil Engineers, Member

Licenses
New York

A structural engineer with Parsons Brinckerhoff since 1950, Mr. Mager has over 30 years of professional experience. He has participated in both the design and the checking of numerous tunnels, bridges, and other major structures.

His various assignments include:

Tunnels

- Principal design engineer on the Second Downtown Eilzabeth River Tunnel, Portsmouth, Virginia, including layout and design of sunken tubes, suspended tunnel ceiling, and waterproofing of the cut-and-cover tunnel sections.

- Project engineer on ceiling and wall repairs for the Brooklyn-Battery and Queens-Midtown tunnels, New York City. Mr. Mager was responsible for preparing contract plans and specifications.

- Principal designer on the Second Hampton Roads Tunnel, Virginia. His responsibilities included layout and design of sunken tubes and suspended tunnel ceiling, and checking design and detailing of open approaches.

- Senior designer on the Potomac River crossing of the Washington, D.C., "Metro" subway. He was responsible for checking the design and for performing layout and detailing of reinforcing steel on ventilation shafts and addits. He also checked tunnel layout and clearance geometry.

- Senior designer on the Lexington Market Line of the Baltimore subway. He was responsible for layout and design of cross passages, and checking tunnel layout and clearance geometry.

- Designer on the San Francisco Trans-Bay Tube, including sunken tubes and the ventilation building, which was an unusual design constructed as a floating caisson sunk into place.

- Senior engineer in the preparation of a comprehensive engineering report on a proposed twin-tube vehicular tunnel in Japan entitled Project One. He was responsible for the preparation of comparative designs and estimates for circular vs. rectangular tube sections. He prepared the design analysis of circular tubes for the report.

Bridges and Other Major Structures

- Principal designer for the Newport Suspension Bridge, Newport, Rhode Island. He was responsible for the design of the tower anchorage, cable bent, cable saddles, stiffening truss details, floor, and lateral systems, and for checking shop drawings.

- Principal designer for the Fremont Bridge, Portland, Oregon, the world's longest tied arch bridge. He was responsible for the design of the main bearings, arch details and splices, and cable hangers, as well as checking truss details and shop drawings. The Fremont Bridge won the American Institute of Steel Construction's award as the most beautiful long-span bridge built in 1974.

- Principal checker of the lift span and towers for the James River Bridge, Newport News, Virginia.

- Designer of bascule piers for the Curtis Creek Bascule Bridge, Baltimore, Maryland.

- Design engineer on the White Plains Garage, White Plains, New York. His responsibilities included structural analysis and design of the main building framing and of the spiral ramps.

Consumers
Power
Company

J A Mooney
Executive Manager
Midland Project Office

General Offices: 1945 West Parnall Road, Jackson, MI 49201 • (517) 788-0498

July 26, 1983

Mr J J Harrison
US Nuclear Regulatory Commission
Region III
799 Roosevelt Road
Glen Ellyn, IL 60137

MIDLAND ENERGY CENTER
MIDLAND DOCKET NOS 50-329, 50-330
REACTOR BUILDING SETTLEMENT
FILE: C-27, 0505.7 SERIAL: 23879

Pursuant to our discussion on July 18, 1983, enclosed please find the crack summary information which was provided to your legal counsel on July 20, 1983.

JAMooney

JAM/JNL/bjw

CC RJCook, Midland Resident Inspector, w/a
JGKeppler, Administrator, NRC Region III, w/a
DSHood, US NRC, w/a
RBLandsman, US NRC, Region III, w/a
OL/OM Service List, w/a

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& EAST SHAFTS
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7-14-83

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BY W. Chiagamatody for R. Kent Date 7-14-83

7220-200 REV 9/82

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STARTUP SYSTEMS OR AREAS ≤ 90 DAYS OF TURNOVER, OR AFTER TURNOVER IF AFFECTED BY CHANGES:

System/ Area No.	Hardware Change (Yes or No)
NA	

7220-242

7220-C-198-2857-1 TOTAL # OF PAGES - 11

ALE: ONE
EQUALS ONE FOOT

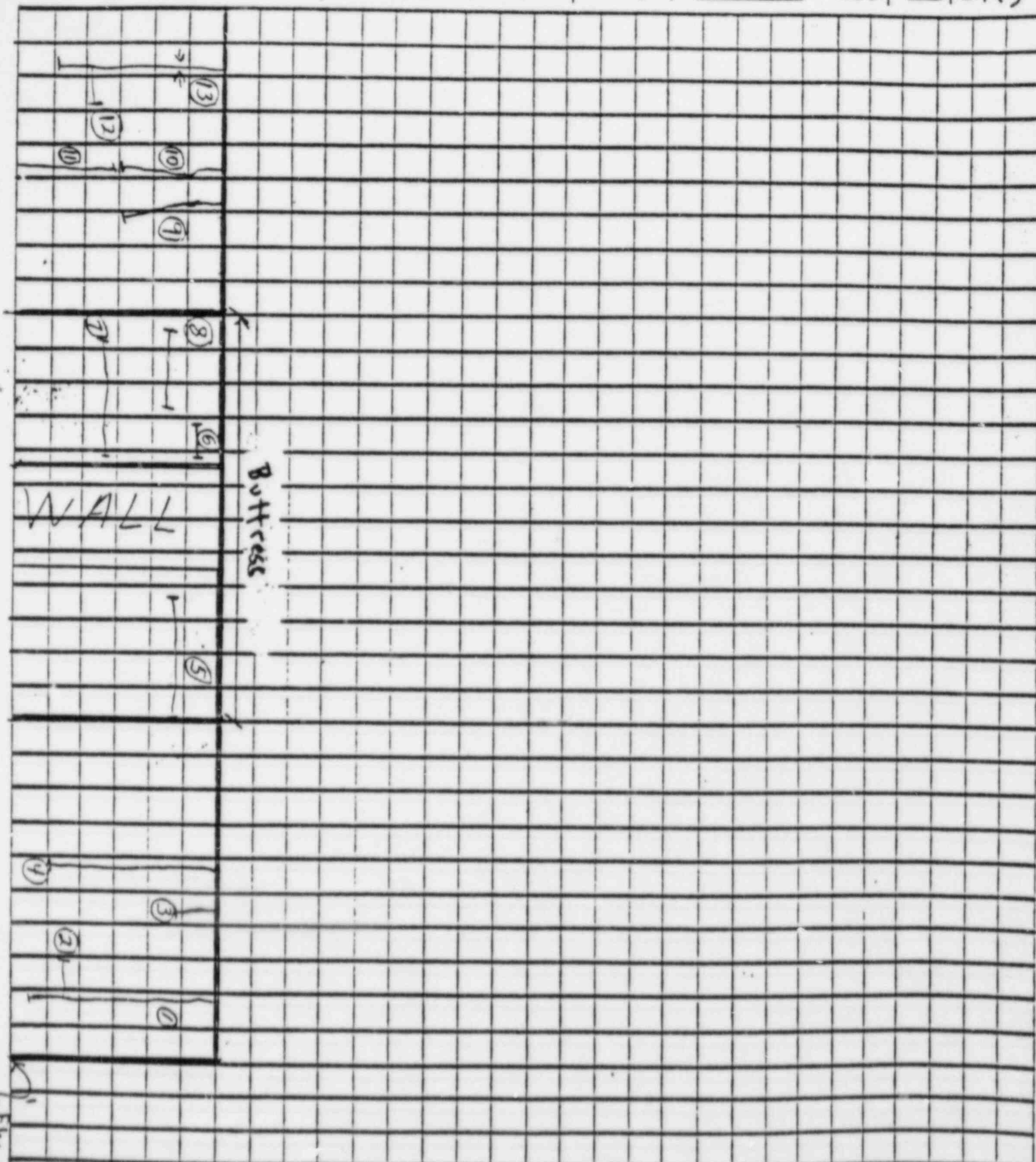
AUXILIARY BUILDING

DATE 7/12/83

REVIEWED BY M [illegible]
WJE (LEVEL II INSPECTOR)

ELEVATION 591'6"
COMPARATOR NO. C1-1
CALIBRATION DUE DATE N/A

LOCATION Unit #2 East
SEQUENCE NO. 1
SURVEYED BY [illegible]



7220-C-198-2857-1

TAB... 43T

MEASURED CRACK WIDTH SUMMARY

591'6" Unit #2, East

CRACK NO.	DATE											
	C1-1 7/12/83											
1	H.L.											
2	H.L.											
3	H.L.											
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7220-C-198-2857-1

SCALE: ONE EQUALS ONE FOOT

AUXILIARY BUILDING

DATE 7/11/83

ELEVATION 591' 6"

LOCATION Unit #1 West

COMPARATOR NO. C1-1

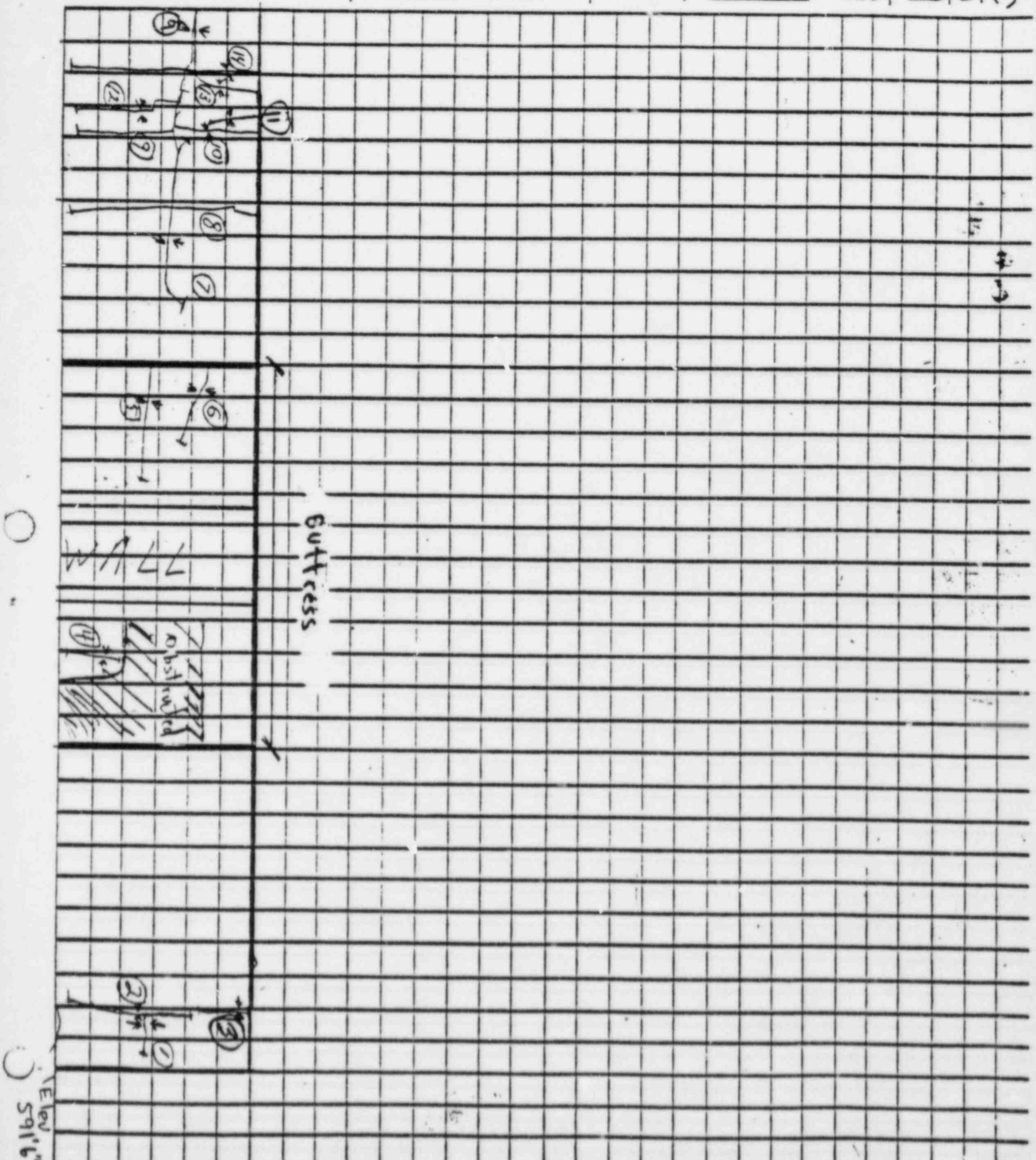
SEQUENCE NO. 1

CALIBRATION DUE DATE N/A

SURVEYED BY SK

REVIEWED BY

WJE (LEVEL II INSPECTOR)



7220-C-198-2857-1

TABLE 43T

MEASURED CRACK WIDTH SUMMARY

591'6" Unit #1, West

CRACK NO.	DATE											
	L1-1 7/11/83											
1	H.L.											
2	.005											
3	H.L.											
4	H.L.											
5	H.L.											
6	H.L.											
7	H.L.											
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10	H.L.											
11	.005											
12	H.L.											
13	H.L.											
14	H.L.											

SCALE: ONE ■ EQUALS ONE FOOT

AUXILIARY BUILDING

DATE 7-12-83

ELEVATION 591'-6"

COMPARATOR NO. C1-1

CALIBRATION DUE DATE NA

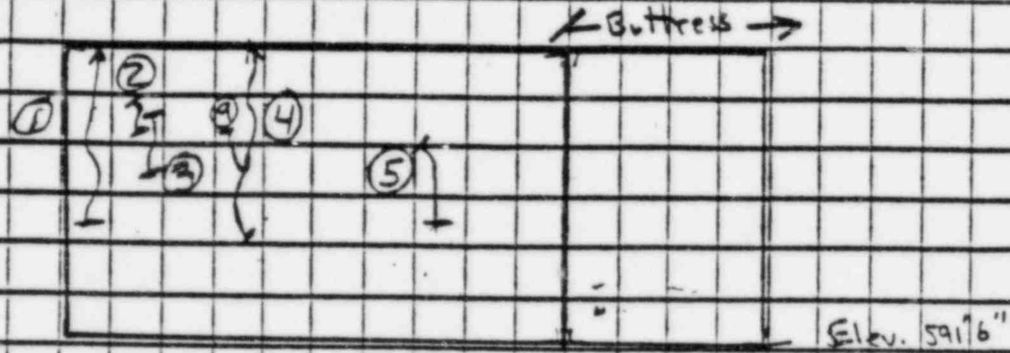
LOCATION UNIT 2 NORTH

SEQUENCE NO. 2

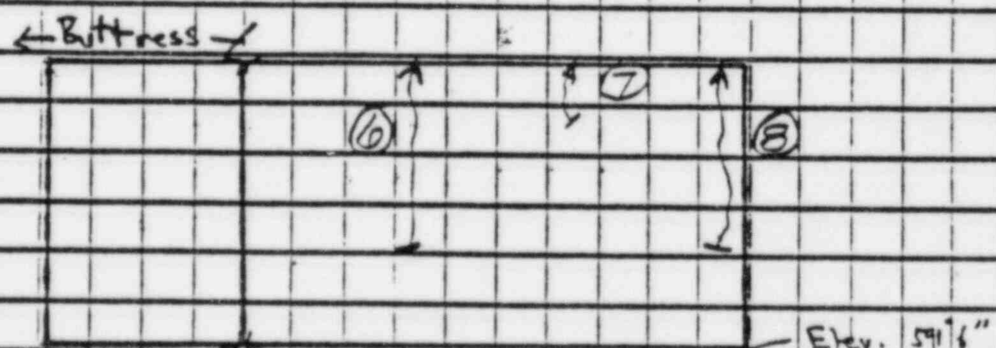
SURVEYED BY [illegible]

REVIEWED BY M Coma

WJE (LEVEL II INSPECTOR)



WEST SIDE



EAST SIDE

TABLE 43T

MEASURED CRACK WIDTH SUMMARY

591' L". UNIT 2, NORTH

CRACK NO.	DATE 4-1 7/12/83											
1	HL											
2	HL											
3	HL											
4	HL											
5	HL											
6	HL											
7	HL											
8	HL											
9	HL											

7220-C-198-28571

SCALE: ONE ■ EQUALS ONE FOOT

AUXILIARY BUILDING

DATE 7-12-

ELEVATION 591'-6"

COMPARATOR NO. C'-1

CALIBRATION DUE DATE NA

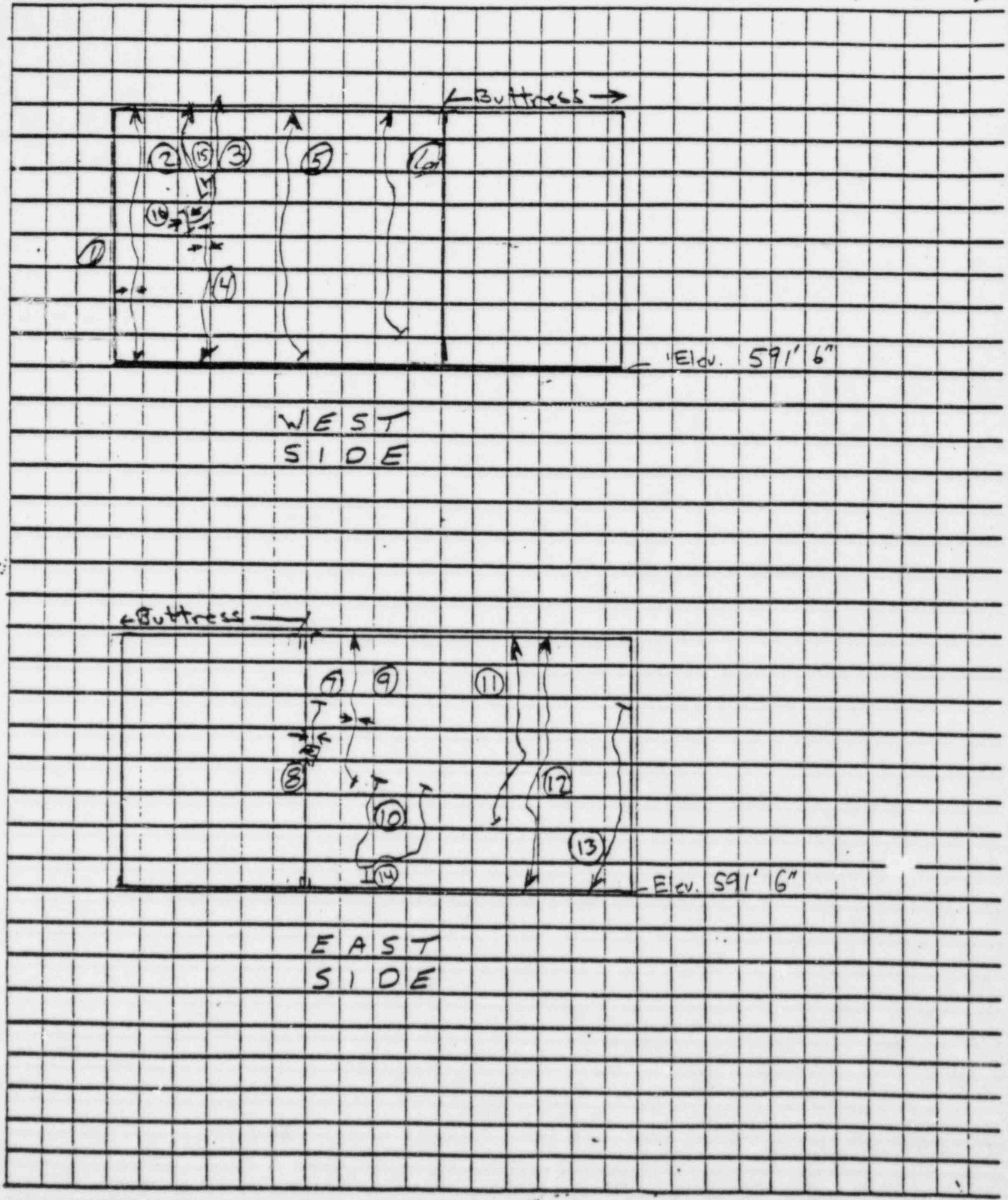
LOCATION UNIT 1 NORTH

SEQUENCE NO. 1

SURVEYED BY [illegible]

REVIEWED BY M [illegible]

WJE (LEVEL II INSPECTOR)



7220-C-198-2857-1

8

TAB... 43T

MEASURED CRACK WIDTH SUMMARY 591' 6". UNIT 1 NORTH

CRACK NO.	DATE											
	C1-1 7/12/83											
1	.015											
2	HL											
3	HL											
4	.010											
5	HL											
6	HL											
7	.010											
8	.010											
9	.010											
10	HL											
11	HL											
12	HL											
13	HL											
14	HL											
15	HL											
16	.010											

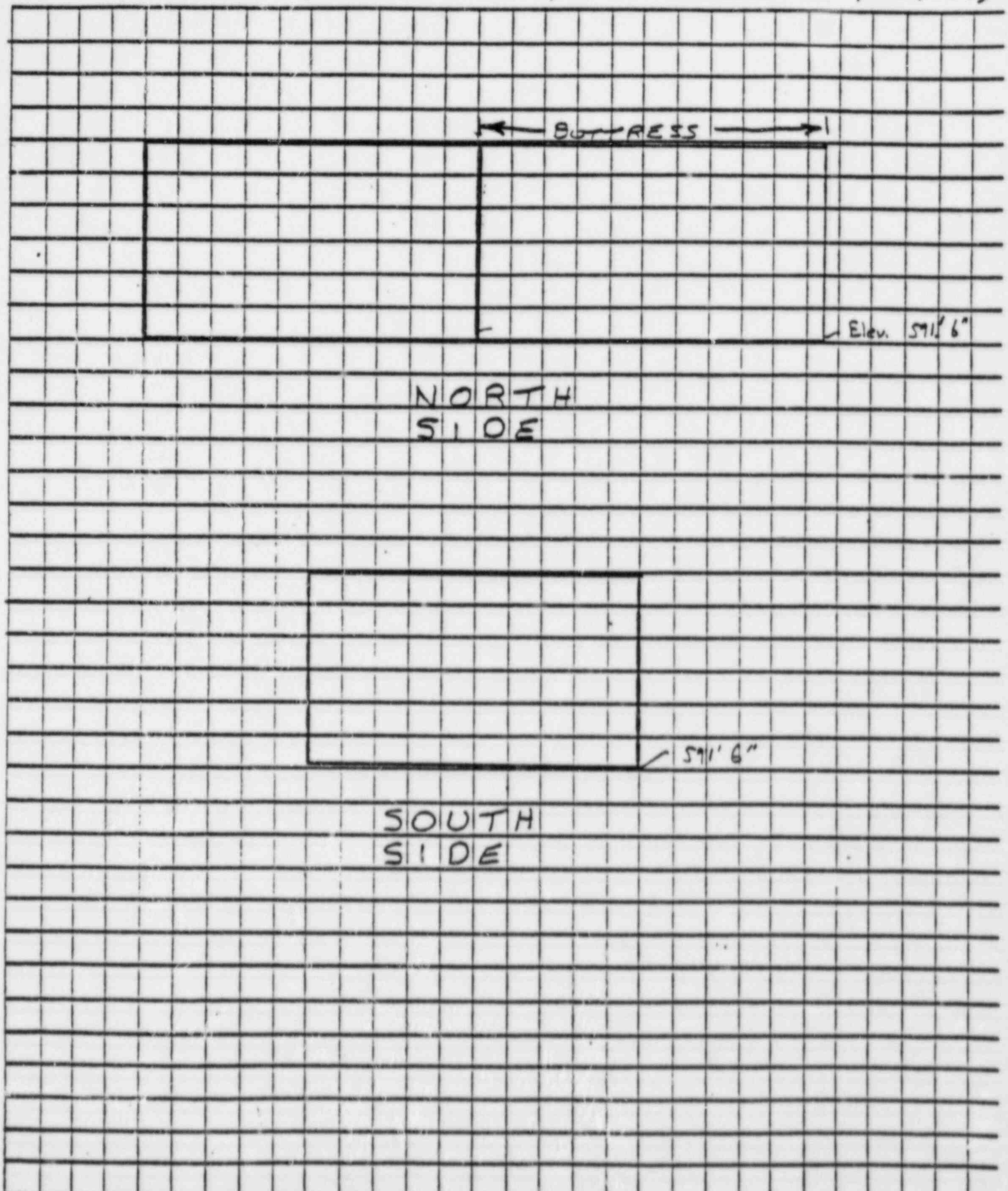
SCALE: ONE ▪ EQUALS ONE FOOT | AUXILIARY BUILDING | DATE 7-13-8

REVIEWED BY M Conn

ELEVATION 591'-6 | LOCATION Unit 2 West

COMPARATOR NO. C2-1 | SEQUENCE NO. 1 | WJE (LEVEL I

CALIBRATION DUE DATE NA | SURVEYED BY | INSPECTOR)



TAB... 43T

MEASURED CRACK WIDTH SUMMARY

59'6" · Unit # 2 , west

CRACK NO.	DATE											
	7/15/85											
	No cracks											

PRELIMINARY NOTIFICATION OF EVENT OR UNUSUAL OCCURRENCE--PNO-III-83-49 Date: June 17, 1983

This preliminary notification constitutes EARLY notice of events of POSSIBLE safety or public interest significance. The information is as initially received without verification or evaluation, and is basically all that is known by the staff on this date.

Facility: Consumers Power Company
Midland Nuclear Power Plant
Docket Nos: 50-329, 50-330
Midland, MI 48640

Licensee Emergency Classification:
____ Notification of Unusual Event
____ Alert
____ Site Area Emergency
____ General Emergency
xx Not Applicable

Subject: WORKER LAYOFF

The Mergentime Corporation, the contractor hired to perform remedial soils work on a portion of the Auxiliary Building at Midland, announced that it will lay off 40 workers from its workforce of 105 persons.

The layoff, which involves 22 laborers and 18 machine operators, apparently has been caused by the NRC's delay in approving the next stage of remedial soils work. The licensee's request for authorization to continue remedial soils work is still under review by the NRC.

The Mergentime Corporation plans to lay off 20 workers June 17, and the remainder on June 18, 1983. At the present time, eight underpinning piers, out of a proposed total of 57, have been installed. Pier installation is the method selected to correct the problem of poor soil compaction under a section of the Auxiliary Building.

Neither the licensee nor Region III plans a news announcement.

The State of Michigan will be notified.

The Resident Inspector was notified of this event at 12:30 p.m. (CDT) on June 17, 1983. This information is current as of 3 p.m. (CDT), June 17, 1983.

Contact: R. Gardner (signed R. Gardner), FTS 384-2524 — R. Warnick (signed R Warnick), 384-2575

~~8311040075~~

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Rev. 1/19/83

Consumers
Power
Company

J A Mooney
Executive Manager
Midland Project Office

General Offices: 1945 West Parnall Road, Jackson, MI 49201 • (517) 788-0774

June 9, 1983

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orig + 3

Mr J J Harrison
Midland Project Section
U S Nuclear Regulatory Commission
Region III
799 Roosevelt Road
Glen Ellyn, IL 60137

MIDLAND ENERGY CENTER GWO 7020
LOAD TEST FOR PIER W11
File: 0485.16 UFI: 42*05*22*04 Serial: CSC-6735
70*01

During the NRC visit of May 11 and 12 at the Midland Site, data for the load test at Pier W11 was presented. The applicant believed that based on the data from Pier W11 as well as other prototype piers that the apparent soil modulus, E, value shown was consistent with the design assumptions for the permanent underpinning design. However, the NRC believed that an E value of 1500 ksf and a differential settlement of ½" between the electrical penetration area/control tower and the main auxiliary building was appropriate. Hence the NRC asked Consumers to look at the following options:

A. Review the building capacity for an E of 1500 ksf or differential settlement of ½" and provide results of shear strength from unconsolidated undrained triaxial tests on representative samples taken within 1½ feet of the bearing stratum of some piers (to confirm the design ultimate bearing capacity of 44 ksf).

B. Increase the jacking load so that the remaining differential settlement after lock off is ¼" and provide results of triaxial tests as discussed in option (A).

C. Hold the jacking load on the permanent wall long enough so that the remaining differential settlement after lock off is ¼" and provide results of triaxial tests as discussed in option (A).

D. Perform another pier load test. One of the requirements is that friction between the pier and surrounding soil is eliminated. The NRC may provide additional requirements if the applicant chooses this option. One of the additional requirements may be to hold the duration of each load increment longer. In this case provide results of triaxial tests as discussed in option (A).

~~8309070214~~

JUN 20 1983

E. Perform a plate load test for a plate of 18" minimum size. The plate should be loaded so as to reach failure load (or the assumed ultimate bearing capacity) of the soil. The results of this test should demonstrate that a minimum E value on the order of 3000 ksf is achieved. If this test is performed to ASTM standards, NRR Washington need not review the test procedure further. Region III can give approval directly before the test is performed.

Consumers Power Co. was to indicate to the NRC which option they would adopt. Based on CPCo review of the options we wish to adopt option (A). We have conducted a parametric study for the auxiliary building with reduced spring values in order to achieve the suggested differential settlement of ½". The reduced spring constants correspond to apparent modulus values less than 1500 ksf. These reduced springs induced differential settlements of 0.47" for the EPA (relative to the Main Auxiliary Building) and 0.44" for the control tower (relative to the Main Auxiliary Building).

Our review of the analysis, based on the structural design criteria, has indicated that:

1. The existing structure south of column row G is adequate.

2. The permanent underpinning wall reinforcement as designed remains unchanged.

3. All control towers Piers including CT1, CT3, CT11 and CT12 also remain unchanged.

4. The connection between the EPA and the Main Auxiliary Building and the Control Tower underpinning and the building may have some minor effect. The final design of these connections is underway.

5. Based on review of the most critical settlement loading combination of the main building north of column row G, there are a couple of localized areas at elevation 634' and 659' which are slightly overstressed. We believe that a more detailed evaluation will demonstrate these areas to be adequate. In case these areas can not be shown to be adequate, the appropriate repairs will be made.

The calculations performed for the above study are available for review. We have also reviewed option (C) i.e. holding the jacking load longer. Since the parametric study shows that the structure can take a larger differential settlement than originally assumed, we believe the present acceptance criteria for final lock-off of the permanent foundation should be redefined. The present acceptance criteria for lock off, referenced in SSER section 3.8.3.1 page 3-9, is as follows:

1. Reaching secondary consolidation on the semi-log plot.

2. Settlement increment of .05" in last 30 days.

3. Settlement increment of .01" in last 10 days.

The second criteria translates to ¼" additional total settlement for 40 years after lock off. The differential settlements corresponding to these criteria would be even smaller since the Main Auxiliary Building will also be settling during this time.

Since a study has been performed per option (A), items 2 and 3 of the acceptance criterian should be redefined as follows:

2. .05" in last 15 days. (This translates to ½" additional total settlement for 40 years after lock off.)

3. .01" in last 5 days.

Based on the above, Consumers Power Company believes that the structure is satisfactory for the lower E value for control tower and EPA and we have therefore decided not to perform a new load test. Based on the capacity of the structure, we would also redefine the acceptance criteria for the lock off of the permanent wall. We also commit to provide NRC with results of the triaxial tests.

JAMooney

JAM/KBR/klm

	ACTION	INFORMATION
J W Cook, P26-336B		
R A Wells, MPQAD		
A J Boos, Bechtel Ann Arbor		
J A Mooney, P14-115A		
J E Brunner, M-1079		
J R Schaub, P14-305		
R C Bauman, P14-312B		
W R Bird, P14-418A		
J K Meisenheimer, Midland		
A R Mollenkopf, P14-408A		
D B Miller, Midland		
F W Buckman, P24-624A		
D B Budzik, P24-517A		
N J Saari, Midland		
D F Lewis, Bechtel Ann Arbor		
R W Huston, Consumers Power Company 7910 Woodmont Avenue Suite #220 Bethesda, Maryland 20014		
R L Tueteberg, P24-505		
NRC Correspondence File, P24-517		
Mr. Mike Miller Isham, Lincoln & Beale 3 First National Plaza, Suite #5100 Chicago, IL 60602		
Isham, Lincoln & Beale 1120 Connecticut Avenue N.W. Washington, D.C. 20036		
R M Wheeler, Midland		
A E Blocher, Midland		
T R Thiruvengadam, P14-400		
Neil Swanberg, Bechtel Ann Arbor		
Mr. Ron Callen Michigan Public Service Commission 6545 Mercantile Way Lansing, MI 48909		

File: 0485.16 UFI: 42*05*22*04

Consumers Power Company

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J A Mooney
Executive Manager
Midland Project Office

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DPRP		SLO	
DRMA		RC	
DRMSP			
DE			
ML			
OL		FILE	

September 7, 1983

Mr J J Harrison
Midland Project Section
U S Nuclear Regulatory Commission
Region III
799 Roosevelt Road
Glen Ellyn, IL 60137

MIDLAND ENERGY CENTER GWO 7020
REMEDIAL SOILS DISCUSSIONS BETWEEN CPCo AND NRC
File: 0485.16 UFI: 42*05*22*04 Serial: CSC-6871
70*01

This letter is to confirm discussions with Region III's Dr. Landsman and Mr. Gardner and with Mr. Wheeler and Mr. Wieland of CPCo on September 1, 1983. The following agreements were reached:

1. Dr. Landsman indicated the NRC concurrs with the Engineering logic change which allows the drifts from Kc-2 to Kc-3 and Kc-10 to Kc-11 to be constructed before Piers Kc-3 or Kc-10 are jacked.

2. Dr. Landsman concurred with eliminating the activity entitled "Construct Concrete Invert and Layback Soil Kc-2 to Kc-3" and "Construct Concrete Invert and Layback Soil Kc-11 to Kc-10" from the work activity list.

3. It was pointed out to Dr. Landsman that a Consumers Power letter, serial CSC-6863, dated 8/25/83, has the incorrect activity number to "Install Pier W13", due to a typing error. The number for this activity should be 165054035 instead of 165053035. This incorrect number was also in an NRC approval letter, dated 8/29/83. For the purposes of documentation, it was agreed that the NRC approval letter dated 8/29/83 authorized the activity "Install Pier W13" and an additional authorization letter from NRC is not required.

4. On September 1, 1983, a discussion was held between Dr. Landsman and our Mr. Puhalla in which Dr. Landsman concurred with relocating LS-10 from an interior piezometer to an exterior piezometer, and also concurred with FCR C-6556 to Drawing C-1320 which deletes wells 555, 561, 576 from the dewatering schedule and adds them to the piezometer schedule.

If you have any questions please contact this office.

JAMooney

8309150002

JAM/RMW/klm

SEP 12 1983

GEOTECHNICAL ENGINEERS INC.
1017 MAIN STREET · WINCHESTER · MASSACHUSETTS 01890 (617) 729-1625

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RC		DRMA	
PAO		SCS ✓	
SGA		ML	
ENF		File	

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MEMORANDUM OF TELEPHONE CONVERSATION

Landsman

Date of Memo: December 20, 1983
Date of Telcon: December 16, 1983
Participants in Telcon: NRC: Ross Landsman, Joseph Kane
GEI: Steve Poulos

Topic: Control of Movements of EPA Structure During Underpinning Operations Midland Nuclear Plant

During the past week Consumers Power Company had recommended that the jacking loads under the east and west ends of the EPAs be increased above those specified in SER and supplements.

After lengthy discussions regarding the behavior of the EPA, CT, and AUX buildings to date, the following values of $\Delta 1$ and $\Delta 2$ (differential movements) were obtained for the period August 1982 through December 15, 1983.

	W2	W3	E3	E2
$\Delta 1$ (mils)	33 Up	35 Down	17 Down	60 Up
$\Delta 2$ (mils)	72 Up			50 Up

The above data indicate that due to the jacking loads applied to date under the ends of the EPAs, which is the only location of jacks at present, the ends of the EPAs have been lifted up relative to the Control Tower by 72 mils (west side) and 50 mils (east side).

The conclusions reached during the telephone conversation, for use at least until the forthcoming site visit in January 1984, are:

DEC 27 1983

1. Maintain $\Delta 1$ between 150 mils down and 50 mils up, and

2. Maintain $\Delta 2$ between zero mils down and 50 mils up.

3. Jacking loads may be altered to maintain above criteria, subject to the structural capacity of the EPA being sufficient to accommodate the loads.

The above conclusions will be transmitted by Dr. Landsman to the applicant for immediate use during the interim period between now and the site visit in January 1984.

It was stressed that the intent of the underpinning process was to cause a minimum of flexure in the EPAs and CT relative to each other and to the main AUX building during installation of the jacks. The loads in the jacks are secondary to this requirement, so long as the structures supported can withstand the jacking loads. If the structure needs to be supported with more load to prevent flexure, then jacks must be added in accordance with plans developed during the Audits in 1982 and 1983.

GEOTECHNICAL ENGINEERS, INC.

Steve Poulos

Steve J. Poulos
Principal

SJP:tr

Original: R. C. Herrick, FRC
Copies: J. Kane, NRC
R. Landsman, NRC
R. Samuels

Consumers
Power
Company

James W Cook
Vice President - Projects, Engineering and Construction

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

April 19, 1982

Harold R Denton, Director
Office of Nuclear Reactor Regulation
US Nuclear Regulatory Commission
Washington, DC 20555

MIDLAND PROJECT
MIDLAND DOCKET NO 50-329, 50-330
SUMMARY OF SOILS-RELATED ISSUES
AT THE MIDLAND NUCLEAR PLANT
FILE: 0485.16, 0485.18 SERIAL: 16629
ENCLOSURES: SUMMARY OF SOILS-RELATED ISSUES
AT THE MIDLAND NUCLEAR PLANT

As a result of recent discussions between the NRC Staff management and Consumers Power Company management, it was concluded that a summary report addressing all of the soils-related issues at the Midland Nuclear Plant would be beneficial in completing the Staff's extensive review of the remedial actions proposed with regard to these issues. The enclosed report is a technical summary which provides a history of the soils problem at the Midland plant and a discussion of the design and construction details concerning the remedial measures for the diesel generator building (DGB), auxiliary building, service water pump structure foundation, permanent dewatering system, and underground utilities. The quality assurance program for the underpinning activities is also discussed. Finally, the enclosed report presents the status of design, licensing, and construction of the remedial activities for the various affected structures and utilities on the Midland site.

It is our expectation that this report will serve several purposes. Our objective in providing this technical report is to summarize the soils-related remedial measures for use in the NRC's staff management review and as an introduction to this topic for the Adivsory Committee on Reactor Safeguards (ACRS) Subcommittee.

We believe that this report, together with all the other exhaustive soils-related information provided to the NRC Staff, should assist the Staff in completing its review, issuing a Safety Evaluation Report (SER) on the soils remedial actions and in providing its concurrence on remaining items of soils-related construction. In further support of this continuing effort, we are providing by separate correspondence reference document tabulations of the detailed information available to the Staff. These tabulations of the

oc0482-0065a100

~~8204260172~~

reference information available to the Staff are arranged to correspond to the areas of review identified in those Standard Review Plans pertinent to the Midland soils issues.

James W. Cook

JWC/RLT/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
DCFischer, ACRS, w/a (6)
SGadler, w/o
JHarbour, ASLB, w/o
GHarstead, Harstead Engineering, w/a
RWHernan, NRC, w/a
DSHood, NRC, w/a (2)
DFJudd, B&W, w/o
JDKane, NRC, w/a
FJKelley, Esq, w/o
RBLandsman, NRC Region III, w/a
WHMarshall, w/o
JPMatra, Naval Surface Weapons Center, w/a
WOtto, Army Corps of Engineers, w/o
WDPaton, Esq, w/o
SJPoulos, Geotechnical Engineers, w/a
FRinaldi, NRC, w/a
HSingh, Army Corps of Engineers, w/a
BStamiris, w/o

SUMMARY OF SOILS-RELATED ISSUES

AT THE

MIDLAND NUCLEAR PLANT

April 19, 1982

~~8204260176~~

SUMMARY OF SOILS-RELATED ISSUES

AT THE

MIDLAND NUCLEAR PLANT

EXECUTIVE SUMMARY

Consumers Power Company, the applicant for an operating license for the Midland Nuclear Plant, has been engaged in a comprehensive program to resolve soils-related issues identified during plant construction.

Excessive settlement of the diesel generator building (DGB), resulting from inadequately compacted plant fill, was identified in July 1978. Since then, extensive exploratory tests and studies have been conducted to determine the exact cause and extent of this problem. Subsequently, other soils-related problems have been identified.

In addition to the soils-related issues, remedial actions are necessary to correct a problem affecting the two borated water storage tank (BWST) foundations. Failure of the design to consider nonuniform loading led to overstressing during a load test. This condition was aggravated by the soils conditions.

Together with the architect-engineer, Bechtel Associates Professional Corporation, and numerous other renowned consultants, the Applicant has performed comprehensive and detailed analyses in order to develop satisfactory remedial actions for identified problems.

Throughout this process, the Applicant has maintained an extensive dialogue with the NRC staff through technical reports, responses to questions, meetings, and direct presentations. Concurrence has been received on many of the analyses and remedial design concepts while others are still under review.

The status of soils-related issues as of April 1982 at the Midland Nuclear Plant can be summarized under the following programs:

- o The settlement problem of the DGB has been essentially resolved by preloading the area in and around the building to achieve accelerated consolidation of plant fill which supports the building.

- Adequately compacted fill under portions of the auxiliary building and feedwater isolation valve pit (FIVP) will be resolved by constructing underpinning under the auxilary building and replacing the existing backfill under the FIVP. When completed, the new foundations will carry the loads to the undisturbed natural soils underlying the site. These new foundations will meet newly established seismic design criteria promulgated by the NRC.

- Inadequately compacted fill under the overhang portion of the service water pump structure will be resolved by constructing underpinning similar to that under the auxiliary building.

- Design problems associated with the BWST foundation will be resolved by the preload of the valve pit, which has been completed, and reinforcing the old ring beam with a new concentric ring beam.

- Potential liquefiable pockets of backfill supporting some Seismic Category I structures and utilities will be resolved by providing a permanent plant dewatering system.

- The adequacy of all underground Seismic Category I utilities will be ensured by a variety of actions ranging from acceptance of existing facilities to complete replacement.

- Concerns relating the the quality assurance program for the unique underpinning have been resolved by developing a special quality assurance plan for this work.

This report provides a brief history of the soils-related problems at the Midland plant and presents design and construction details of the remedial measures developed to address these problems. It is intended for use in NRC management reviews and as an introduction to this topic to the Advisory Committee on Reactor Safeguards.

SUMMARY OF SOILS-RELATED ISSUES

AT THE

MIDLAND NUCLEAR PLANT

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SUMMARY OF SOILS-RELATED ISSUES

AT THE

MIDLAND NUCLEAR PLANT

BACKGROUND

A construction permit for Midland Plant Units 1 and 2 was issued by the Atomic Energy Commission on December 15, 1972. Soils-related problems were first identified in July 1978 when the settlement monitoring program detected excessive settlement of the diesel generator building (DGB). The building had settled 3.5 inches at the point of greatest settlement, compared to design predictions of 3 inches for the 40 years of expected plant operation. Shortly thereafter, the Applicant verbally reported the matter to the NRC site inspector, and formally reported it under 10 CFR 50.55(e) in September 1978.

The plant design called for the placement of foundations for certain structures and portions of others on approximately 30 feet of compacted fill material overlying the natural material of the site. Specifications governing the placement and compaction of fill material required typical controls over moisture content, lift thickness, compactive energy, and in situ testing by the traditional soils engineering methods. As was later determined, controls in the areas of both placement and testing were deficient.

Soil placement activities were conducted largely from 1975 to 1977. In August 1977, some settlement was detected for one of seven foundation grade beams of the administration building. This is a nonsafety-related structure that houses plant offices. The settlement was investigated by conducting test borings in the near vicinity and by load testing the remaining grade beams. In addition, two borings outside the immediate area of the failure were taken. The results of the investigation, which was completed in September 1977, demonstrated adequately compacted soils, apart from those directly beneath the beam that had settled.

The foundation construction of the DGB, for which construction was started in October 1977, rests entirely on plant fill material. The Applicant's initial response after discovering the settlement problem in 1978 was to halt DGB construction, pending investigation. Drs. R.B. Peck and A.J. Hendron, Jr., renowned soils consultants, were retained.

The Applicant also initiated a soils boring program, which was later extended to the entire site and resulted in over 350 soil borings. The NRC, for its part, initiated an investigation that continued into the early part of 1979.

Based on results of soil boring samples taken from under the DGB, the Applicant concluded that the soil beneath the DGB was inadequately compacted. The consultants recommended in November 1978 that the Applicant "preload" or "surcharge" the structure. This involved placing a 20-foot layer of sand around the perimeter of and within the structure to accelerate settlement, or more accurately, to "consolidate" the fill material. In the consultant's opinion, a significant advantage of the preload process is its self-verifying nature. That is, when the preload is complete and effective, settlements under the structure approach a straight line on a settlement-versus-log-time graph. In addition, excess pore pressures are dissipated, a fact which can be observed directly by piezometer measurements.

After a through review of the options available, the Applicant elected to institute a surcharge loading program, which subsequently was started in January 1979. In early November 1978, the NRC staff was advised that preloading was the recommended remedial action for the DGB. The staff visited the site in December of that year. Although the staff expressed no opinion at the time, it later objected to the Applicant's actions on grounds that the staff had not been provided adequate acceptance criteria before application of the preload. In the December meeting, the staff indicated that if the Applicant implemented the preload, the Applicant would be proceeding at its own risk.

In August 1979, results from the preload indicated to the satisfaction of the Applicant and its consultants that the criteria for reaching secondary consolidation had been achieved. Accordingly, the Applicant began removing the surcharge in August 1979. The removal operation was completed within a month.

Meanwhile in 1979, while the preload was in place, the results of an extensive boring program elsewhere on the site showed inadequately compacted soil under the electrical penetration areas of the auxiliary building and under a portion of the cantilevered section of the service water pump structure (SWPS), i.e., the portion of the structure that rests on plant fill. Neither building had undergone unusual or excessive settlement. Nevertheless, the Applicant decided to underpin portions of both structures to obtain adequate predictability of structural behavior under design conditions.

The possibility of liquefaction of inadequately compacted sandy soils during seismic conditions also was studied. Grouting of localized sand pockets was considered. However, the Applicant decided upon a permanent dewatering system, because demonstrating that all sand pockets had been successfully grouted was considered difficult and because a dewatering system was both practical and conclusive.

Summary of Soils-Related Issues
at the Midland Nuclear Plant

The NRC staff review of the Applicant's soils proposals was delayed by the Three Mile Island accident. Late in 1979, the NRC staff retained the U.S. Army Corps of Engineers as its consultant. On December 6, 1979, the staff issued an order halting all remedial construction until such time as the Applicant could prove to the staff that its proposed and completed remedial actions were technically sound.

During 1979, the Applicant had responded at length to two sets of 10 CFR 50.54(f) requests. However, the staff did not find the responses adequate. The Applicant requested a hearing and voluntarily agreed not to undertake further remedial construction without concurrence of the NRC staff, although a request for a hearing suspended the effect of the staff order. As a result of the hearing, staff concurrence has been secured on the dewatering system, portions of the auxiliary building underpinning, and certain other work.

In June 1980, the staff, still not assured that the preload had brought about secondary consolidation of the fill under the DGB, requested a series of borings to demonstrate, among other things, that the preload had accomplished its purpose. The staff also asked for borings at other locations, including the cooling pond dike. The Applicant's consultants advised against the borings because they believed errors inherent in this approach would lead to unpredictable results of little or no value. Because the staff believed that the information relied upon by the consultants was ambiguous, the NRC staff maintained its view and the Applicant took the requested borings against the advice of its consultants. Subsequently, the staff has come to believe that the borings confirm the Applicant's predictions of future settlement of the DGB.

The next event of major consequence occurred on October 14, 1980, when the staff changed its position concerning seismic criteria for the Midland site safe shutdown earthquake (SSE). The new staff position, which was announced by a letter, was a departure from criteria approved by the NRC when a construction permit for the plant was issued.

At the staff's request, the Applicant has agreed to revise its underpinning proposals for the SWPS and auxiliary building in order to incorporate this new criteria as a design basis.

The previous underpinning scheme for the SWPS used drilled piles attached to the overhang portion of the structure by corbels. This was found lacking under the heightened seismic loads. A new scheme making use of walls that extended from the structure's original walls to the undisturbed natural material under the cantilevered portion was adopted.

Regarding the auxiliary building, a scheme involving caissons under the electrical penetration area was also abandoned because of increased seismic loads in favor of a wall extending under the electrical penetration area and control tower. The modified schemes were developed in mid-1981 and were presented to the NRC staff in September 1981. The NRC staff has concurred with the concept of the new underpinning schemes.

To resolve the seismic issue raised in the staff's October 1980 letter, the Applicant proposed a site-specific response spectrum (SSRS) for the design of structural remedial work and for a seismic margin analysis of existing structures. The staff has concurred with this proposal. With regard to the auxiliary building underpinning proposal, the staff agreed to conduct its review in four phases to avoid construction delays associated with obtaining staff concurrence. In late 1981, after the staff approved Phase 1, the Applicant started excavations for the access shaft for the underpinning.

During 1981, the Applicant discovered a problem with the borated water storage tank (BWST) foundations. These foundations, which consist of a concrete ring beam and valve pit, are placed on fill. A structural design error resulted in overstressing the ring beam, creating cracks and the potential for yielding of reinforcing steel. To resolve this problem, the Applicant decided to reinforce the old ring beam with a new concentric ring beam to be constructed after preloading the valve pit. The NRC staff has concurred with this remedial concept.

Because of the widespread nature of the fill problems, the Applicant conducted additional plant fill analyses and proposed remedial measures for underground piping located in plant fill around the site. In some cases, existing pipes were proven adequate by analysis. In other instances, the Applicant opted to excavate and rebed pipes. The NRC staff has concurred with the decision regarding which pipes are to be rebedded. The Applicant has also committed to replace a portion of the piping due to an inability to reach agreement with the NRC staff on the acceptance criteria for that portion of the existing piping.

Hearings have been conducted on some aspects of the soils problem and the resulting remedial work. This includes the auxiliary building, the BWST and its foundation, the cooling pond dike, underground piping, and the proposed SSRS. The NRC staff has conducted extensive reviews into the preload plan and its effect on the DGB. In addition, the staff conducted extensive audits on the SWPS and auxiliary building during early 1982.

Since the inception of the soils issues, the Applicant has provided the staff with substantial information through 10 CFR 50.55(e) reports, responses to 10 CFR 50.54(f) questions, technical reports, and direct presentation in meetings. The Applicant has participated in over 50 meetings with the staff on soils-related issues. The 10 CFR 50.54(f) responses alone occupy over 11 volumes of material.

Because of the complexity of these soils-related issues, a summary of the technical details of the remedial work and the quality assurance program applied to the work are presented in seven parts, as follows:

Part I	Diesel Generator Building
Part II	Auxiliary Building and Feedwater Isolation Valve Pit
Part III	Service Water Pump Structure Structure
Part IV	Borated Water Storage Tanks
Part V	Permanent Dewatering
Part VI	Underground Utilities
Part VII	Quality Assurance

PART I: DIESEL GENERATOR BUILDING

1.0 INTRODUCTION

The diesel generator building (DBG) is a reinforced concrete structure with three crosswalls that divide the structure into four cells; each cell contains a pedestal to support a diesel generator unit. The building is supported on continuous footings that are founded at el 628' and rest on backfill that extends down to approximately el 603' (see Figure I-1).

In July 1978, approximately 60% of the building was completed and the pedestals were already in place. The recorded settlements of the building at that time exceeded those which should be anticipated under normal conditions. It appeared that the building was settling due to the consolidation of the backfill and was supported along the north portion by four electrical duct banks acting as vertical piers and resting on the natural soil below the fill.

The Applicant decided to halt construction while an exploration program was initiated to determine the quality of the backfill. Drs. R.B. Peck and A.J. Hendron, Jr. were retained as consultants to advise on the selection and the execution of any remedial action.

The exploration program confirmed that the backfill did not meet the specified compaction requirements at all points and that the fill consisted of cohesive soil, granular soil, and lean concrete. The backfill ranged from very soft to very stiff for cohesive soil and from very loose to dense for granular soil. At the time of the exploration, the groundwater level ranged from el 616' to el 622', and the cooling pond, located 275 feet south of the building, had water level at approximately el 622'.

2.0 REMEDIAL ACTION

After review of settlement observations and results of an exploration program, it was decided that remedial action was necessary and several options were evaluated. Based on consultants' recommendations, it was decided to surcharge the area within and around the building.

The purpose of the surcharge was to accelerate the settlement so that under the operating loads of the structure future settlement would be within tolerable limits. Furthermore, the procedure would permit a conservative and reliable estimate of the future settlement. Before the surcharge was placed, the duct banks were separated from the building and soil instrumentation was installed (see Table I-1).

Surcharging consisted of placing 20 feet of sand above grade (el 634') with the geometry shown in Figure I-1. The surcharge was added in two principal increments as shown by the idealized load history in Figure I-3. Surcharge was effectively begun on January 26, 1979. At the same time, construction of the remainder of the building was resumed and approximately 94% of the structural dead load was completed by the time the surcharge reached maximum level. The cooling pond level was also raised to el 627'. Removal of the surcharge started on August 15, 1979, when it had been determined that primary consolidation of the soil had been achieved.

The Applicant and its consultants have concluded that the surcharge has consolidated the fill beneath the DGB such that the future settlement can be predicted. The Applicant has included this prediction in a structural reanalysis of the building and concludes the DGB is capable of meeting its design requirements over the operating life of the Midland plant.

The NRC staff has concurred with the prediction of future settlement. Discussions with the staff on the structural reanalysis of the building are continuing.

3.0 DATA INTERPRETATION - SETTLEMENT PREDICTIONS

Figure I-3 is a typical plot of settlement versus time for a point on the DGB, along with piezometer readings, cooling pond elevation changes, and the idealized surcharge load history. The same settlement data points have been replotted as settlement versus the logarithm of time as shown in Figure I-2. This semi-log plot shows the typical consolidation behavior with primary consolidation completed and the secondary consolidation beginning at approximately 100 days from the start of surcharge placement. This typical behavior permitted extrapolations to be made to forecast the building settlement during its service life under the conservative assumption that the surcharge remains in place for 40 years. Results of this extrapolation are shown in Figure I-4.

Upon surcharge removal, the building showed the expected rebound of about 0.2 inch. Following rebound and until the start of dewatering in September 1980, the building showed a maximum settlement of 0.1 inch. This is less than the range of 0.2 to 0.5 inch which was predicted on the basis of the previously mentioned straight line extrapolation. Following dewatering activities, the building settled 0.4 to 0.5 inch (see Figure I-5) due to lowering the groundwater table from approximately el 620' to el 595' and the resulting settlement of the fill and natural soil. This range is about half of that predicted on the basis of theoretical calculations.

4.0 SOIL EXPLORATION AFTER SURCHARGE

At the request of the NRC, 11 soil borings were drilled in the DGB area during April and May 1981 as a part of additional soil investigation. Details of this investigation program were coordinated with the NRC staff and its consultants, the Army Corps of Engineers. The results of the field investigation and laboratory testing programs were provided to the NRC staff and its consultants.

4.1 SETTLEMENT CALCULATIONS

At the request of the NRC, one-dimensional consolidation tests were performed on the samples to provide an estimate of maximum past consolidation pressure. The maximum past consolidation pressures interpreted from the laboratory tests showed a scatter predictable for consolidation laboratory tests on heterogeneous fill. The data showed some of the interpreted maximum past consolidation pressures were lower than would have been expected after surcharging; a greater number were higher. Based on the assumption that the lower maximum past consolidation pressures interpreted from the laboratory tests demonstrated that parts of the fill had not achieved full primary consolidation under surcharge loading, a settlement analysis was made to estimate future primary consolidation under the DGB loading. This analysis predicted future primary consolidation settlement values ranging from 0 to 0.4 inch. Because this range is on the same order as that measured as a result of dewatering, the settlements predicted by this analysis were replaced with actual measured settlement values shown in Figure I-5. During the meeting with the NRC staff on February 23, 1982, the settlements calculated on the basis of consolidation tests and measured settlements were discussed and the staff concurred with using measured dewatered settlements plus predicted 40-year secondary consolidation settlements to represent future settlements for the structure.

4.2 BEARING CAPACITY

The results of the strength tests on cohesive soils obtained after surcharging provided shear strength parameters required for evaluation of the factors of safety against bearing capacity failure under static and seismic conditions. The factor of safety against a static bearing capacity failure is greater than 5, compared to the minimum acceptable value of 3. The factor of safety against a bearing capacity failure for combined static and earthquake loads consistent with an SSE of 0.12g is greater than 2.7, compared to the minimum acceptable value of 2.

5.0 EARTHQUAKE SETTLEMENT OF SAND

On the basis of standard penetration tests conducted before surcharge, it is estimated that the settlement of sand due to earthquake ground shaking would be about 0.25 inch.

6.0 DYNAMIC PROPERTIES OF BACKFILL

Seismic cross-hole testing was performed at two locations within the DGB during November and December 1979 to determine the shear wave velocity of the fill for seismic analysis. The data showed the shear wave velocity can be represented by a value of 500 ft/sec from ground surface to el 615' and by a value of 850 ft/sec from el 615' to el 600'.

7.0 SURCHARGE EFFECTIVENESS

Figure I-6 presents a comparison between the pressures that existed during surcharge and those expected during the operating life of the structure. This comparison shows that at all depths the pressures that existed during surcharge exceeded those that are expected while the structure is operational. This comparison confirms that the settlements predicted on the assumption that the surcharge remains in place 40 years (see Figure I-4) are conservative in that all loads added after surcharge removal, including those due to permanent dewatering, were less than the surcharge loading at all depths.

8.0 STRUCTURAL REANALYSIS

At the conclusion of the surcharge program, a structural reanalysis of the DGB was performed. This reanalysis accounted for the actual settlement which had ocurred since the removal of the surcharge, and for the additional settlement predicted to occur over the 40-year life of the plant.

This reanalysis proceeded by defining the acceptance criteria for the structure. These acceptance criteria differ from the acceptance criteria used in the original analysis and design of the structure and set forth in the FSAR only in the addition of four load combinations that include the effect of settlement. These additional load combinations are described in Section 8.1.

8.1 STRUCTURAL ACCEPTANCE CRITERIA

Because of the settlement problem, a structural reanalysis of the DGB was performed in accordance with the structural acceptance criteria which are consistent with FSAR Subsection 3.8.6.3, with settlement effects included as outlined in the response to NRC

Requests Regarding Plant Fill, Question 15 (Revision 3, September 1979). In accordance with an NRC staff request, an additional comparative analysis was performed on the DGB in accordance with the load combinations of ACI 349-1976 as supplemented by Regulatory Guide 1.142.

8.1.1 Diesel Generator Building Analytical Model

The structural reanalysis of the DGB uses a finite-element model. The required load combinations were applied to this model and the resulting forces were investigated for compliance with the structural acceptance criteria. The DGB was modeled as an assemblage of plate, beam, and boundary elements to represent soil.

8.1.2 Structural Adequacy Computations

The final structural reanalysis of the DGB indicated that in no case was the maximum allowable rebar stress exceeded. In nearly 70% of the structure, the tornado load combination produced the largest rebar stress levels. (The largest rebar stress value calculated was 39.15 ksi.)

8.2 LICENSING STATUS

During the meeting of February 24, 1982, the NRC staff, in its review of the testimony being prepared for the public hearings, requested additional analysis of the DGB. In particular, the staff was concerned that settlement stresses induced in the structure prior to and during the surcharge program may be significant. Consequently, an additional analysis is presently being performed to establish rebar stress values which existed prior to surcharge removal.

8.3 CONCLUSIONS

The DGB is a massive, reinforced concrete structure with extensive reserve strength. The structural reanalysis performed on the DGB verifies that the integrity of the structure will be maintained under the most critical load combinations. Based on the analysis performed, it can be stated that the settlement has had minimal effect on the structure, and it can be concluded that the DGB will safely perform its intended function over the operating life of the Midland plant.

9.0 CONCRETE CRACKS

A set of electrical duct banks located beneath the building foundation initially acted to restrain the even movement of the structure during fill settlement. A systematic crack pattern was observed in walls resting on the duct banks. Cracks in walls that do not rest on duct banks are attributable to restrained volume changes during curing and drying of the concrete. Cracks were first mapped after the duct banks were separated from the DGB and prior to surcharge placement. Another crack mapping of the DGB was performed after surcharge removal to acertain the effect of surcharge.

The concrete cracks within the DGB were formally addressed in the response to Question 29 of the NRC Requests Regarding Plant Fill. In this response, the cause and significance of the concrete cracks in all structures were presented. Subsequently, during the NRC structural technical audit of April 1981, further discussion was held concerning the effects of the cracks and the additional rebar stress resulting from the concrete cracks. To evaluate the additional rebar stresses associated with the concrete cracking, a number of analytical approaches have been used and the results forwarded to the NRC in the response to Question 40 of the NRC Requests Regarding Plant Fill. These results indicated that because these stresses are strain-induced secondary stresses, they do not affect the ultimate strength capacity of the cracked member.

In response to an NRC request for a nonlinear, finite-element analysis to evaluate the effects of cracks on the integrity of the DGB, an additional computer analysis of the DGB was performed. This analysis was perfomed using a finite-element program, Automated Dynamic Incremental Nonlinear Analysis (ADINA), which is a three-dimensional, nonlinear program capable of considering concrete crushing, cracking, crack widening, and reinforcement yielding. The east wall of the DGB was selected for the ADINA analysis. A crack was modeled into the east wall, and the ADINA analysis was performed for two governing load combinations. The analysis indicated that the effect of concrete cracks was localized and minor in nature. The results of this ADINA analysis were submitted to the NRC followed by meetings with the NRC staff to discuss these results.

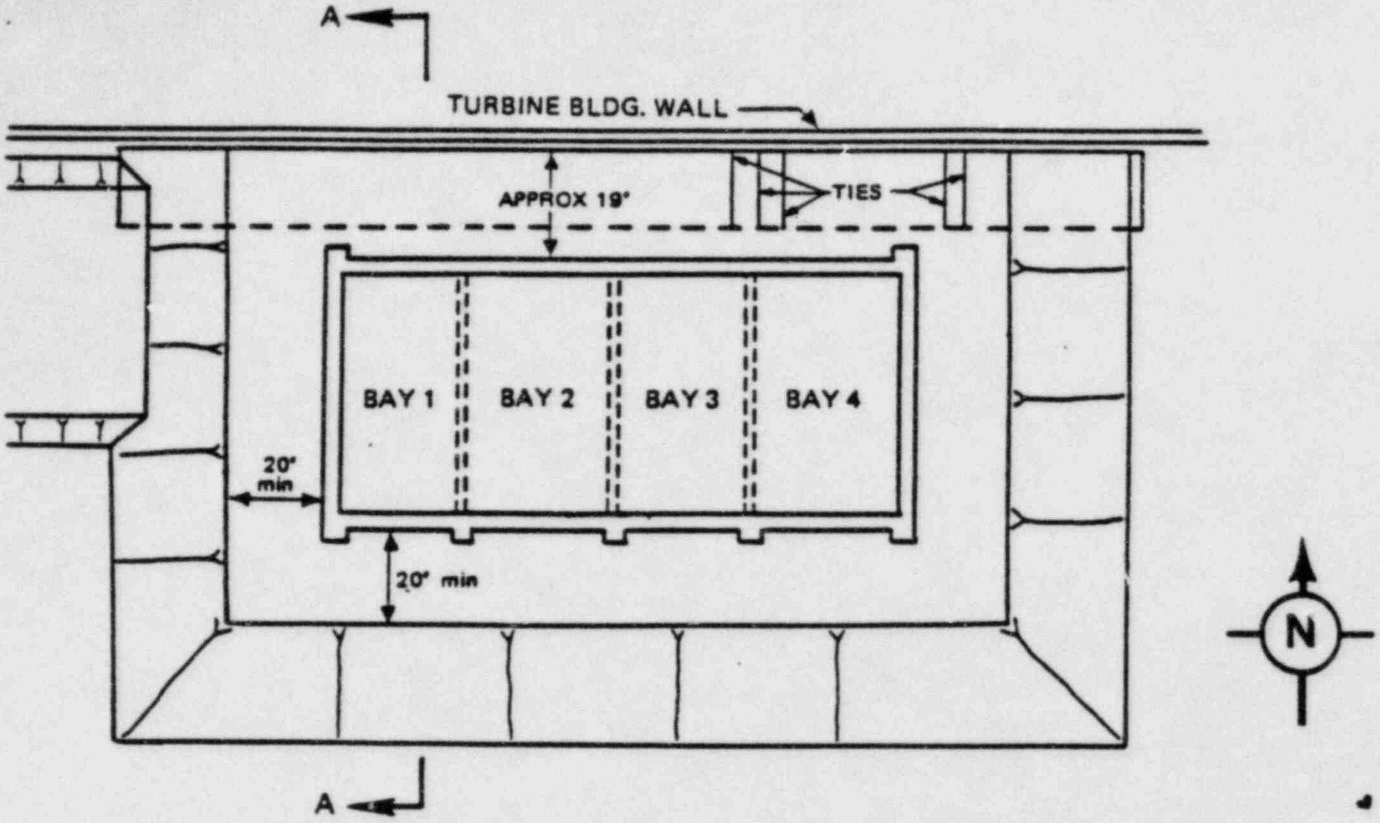
To address additional staff concerns, further evaluation of the existing concrete cracks was performed by Dr. Mete Sozen of the University of Illinois and Dr. W. Gene Corley of Portland Cement Association. The consultants agree that the DGB is capable of withstanding the loads it was initially designed for, despite the existence of concrete cracks. A report addressing the evaluation of cracks by the consultants has been presented to the NRC staff; three meetings have subsequently been held to discuss the crack report. A report on a crack repair program by Portland Cement Association for all cracks in all structures will be submitted to the staff in the near future. Furthermore, crack mapping for the DGB continues at approximately yearly intervals.

A final resolution of the crack issue is still pending with the NRC staff.

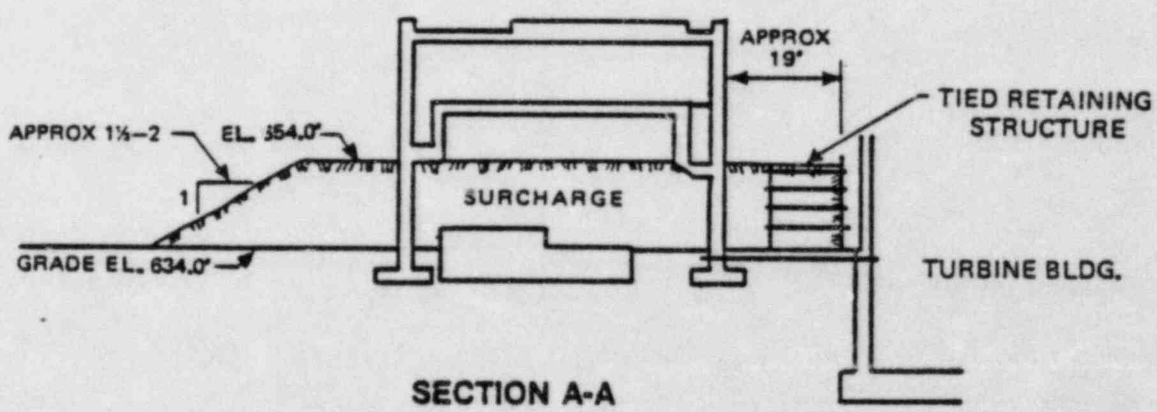
TABLE I-1

DIESEL GENERATOR BUILDING INSTRUMENTATION

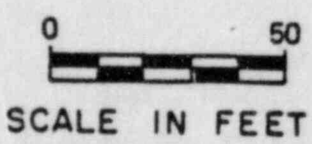
Type	Number
Building Settlement Markers	28
Settlement Plates	52
Borros Anchors	60
Deep Borros Anchors	4
Sandex Gages	5
Piezometers	48



PLAN



SECTION A-A

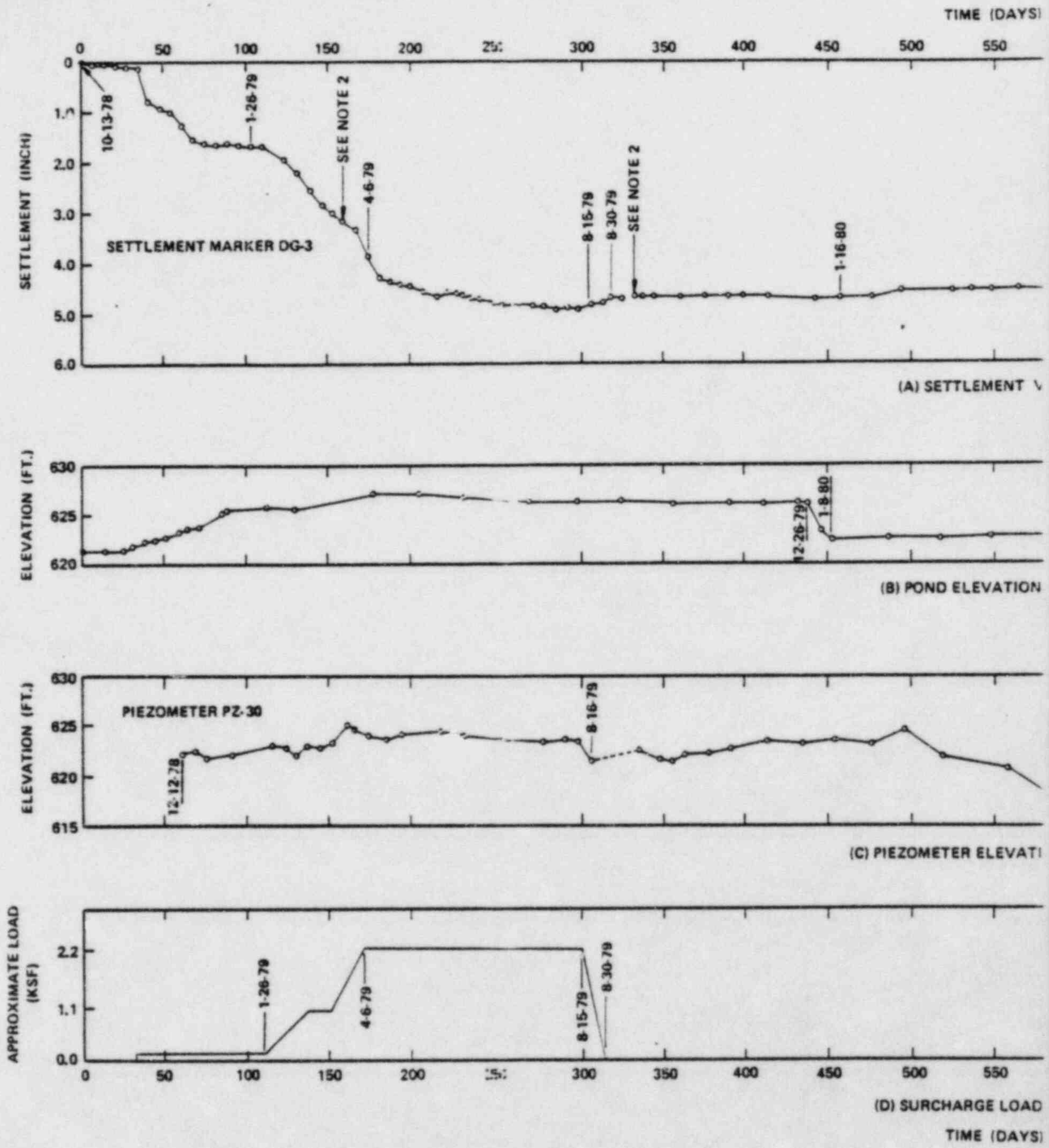


CONSUMERS POWER COMPANY
MIDLAND UNITS 1 AND 2

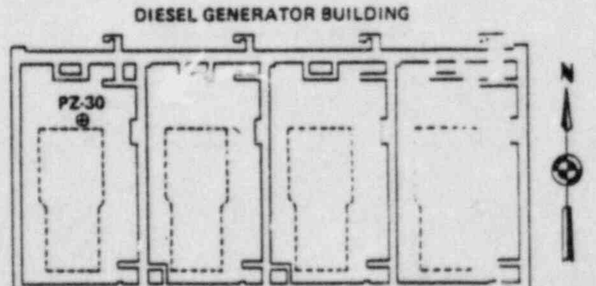
GENERAL LAYOUT OF SURCHARGE LOAD DIESEL GENERATOR BUILDING

FIGURE I-1

G-1986-41



DIESEL GENERATOR BUILDING



LOCATION PLAN
(NOT TO SCALE)

NOTE:

1. O
m

2. Tl
fr
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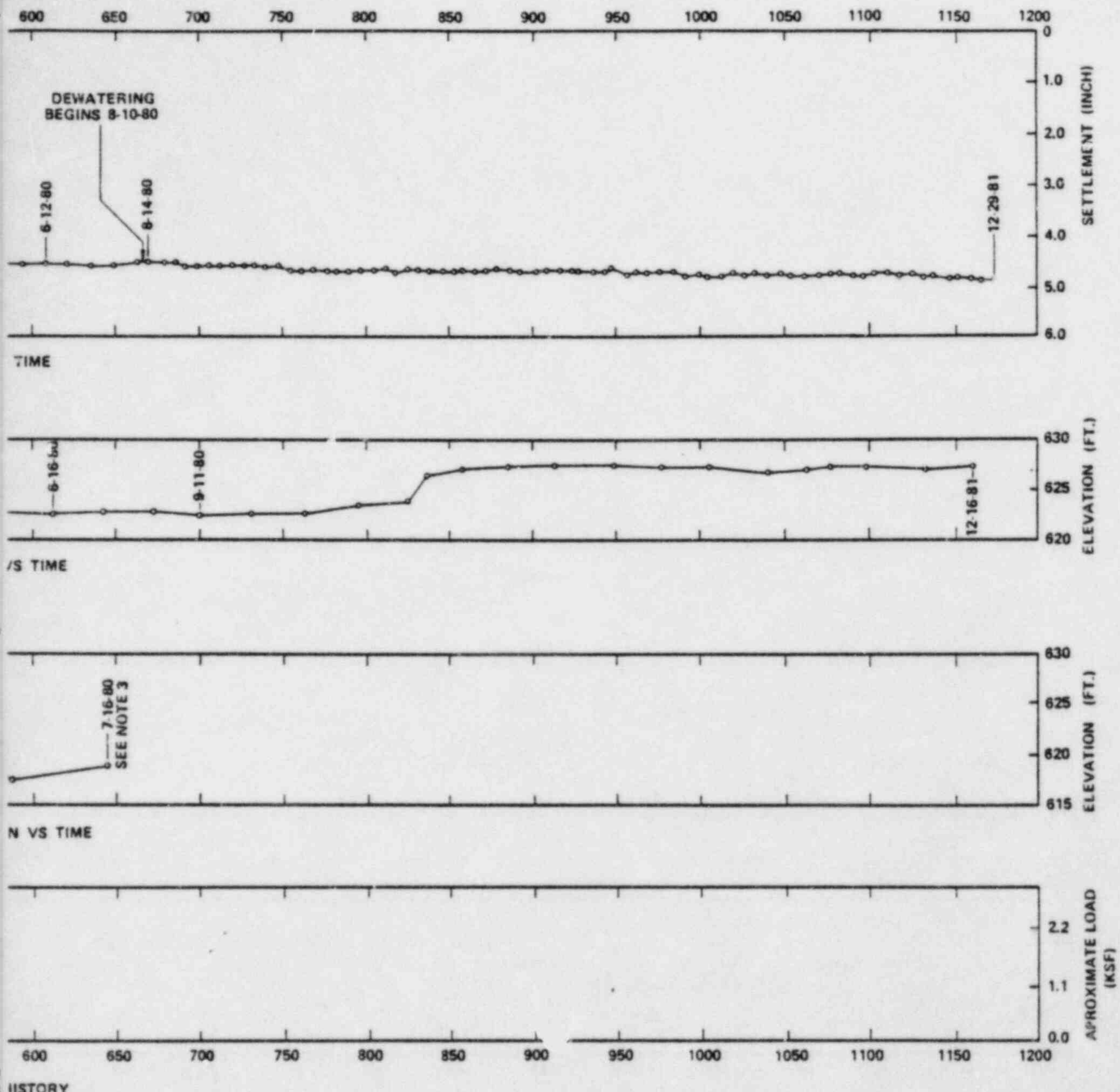
3. Pi



10-13-78 the measured settlement at
rker DG-3 was 2.592 inch.

permanent marker could not be monitored
m 3/22/79 to 9/14/79 due to surcharge.
nporary markers at elevation 664'-0" were
d during this period to estimate the settlement
the permanent markers. On 9/14/79 the
tlement was again based directly upon the
rmanent markers.

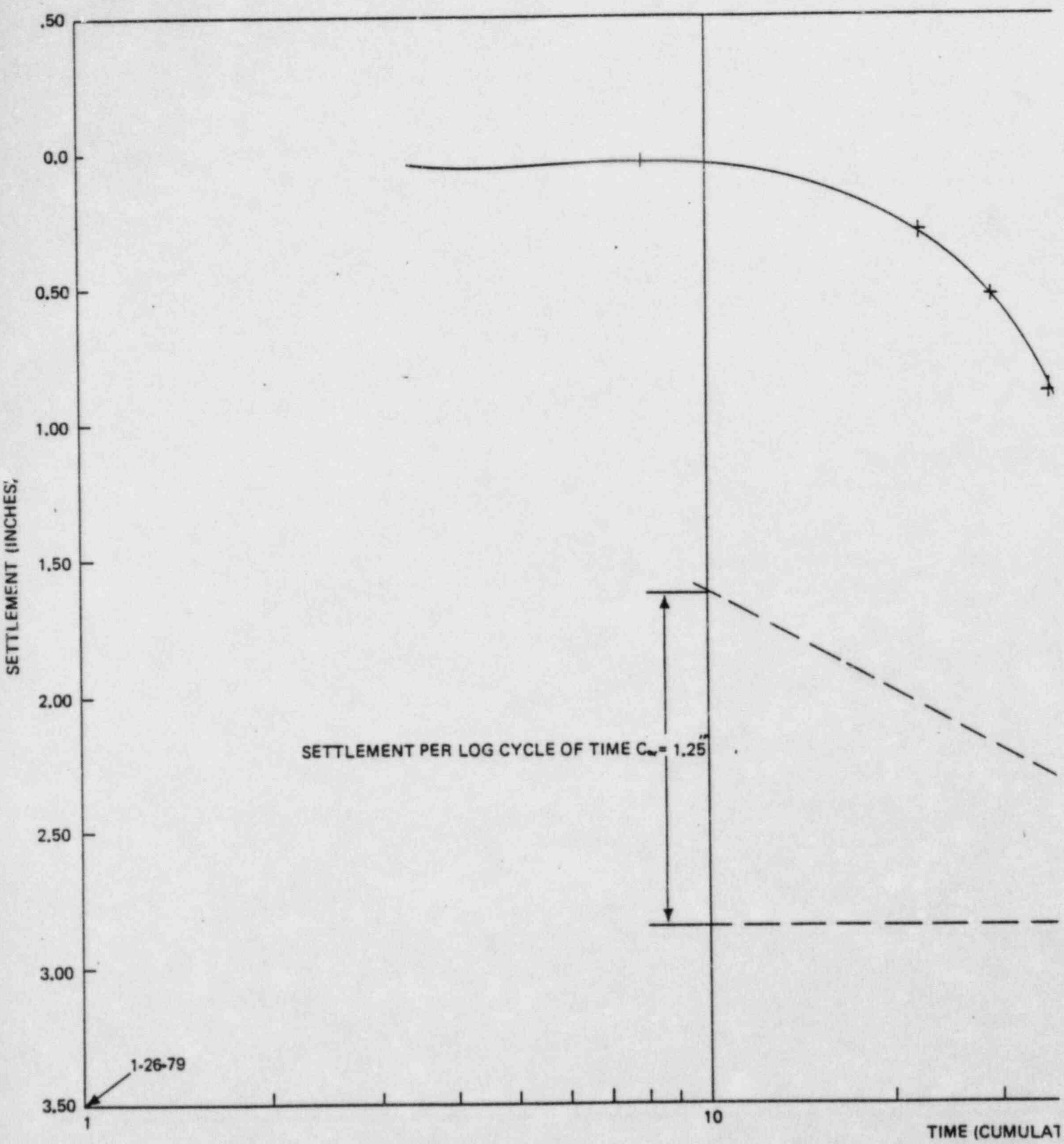
tometer destroyed.

CONSUMERS POWER COMPANY
MIDLAND UNITS 1 AND 2

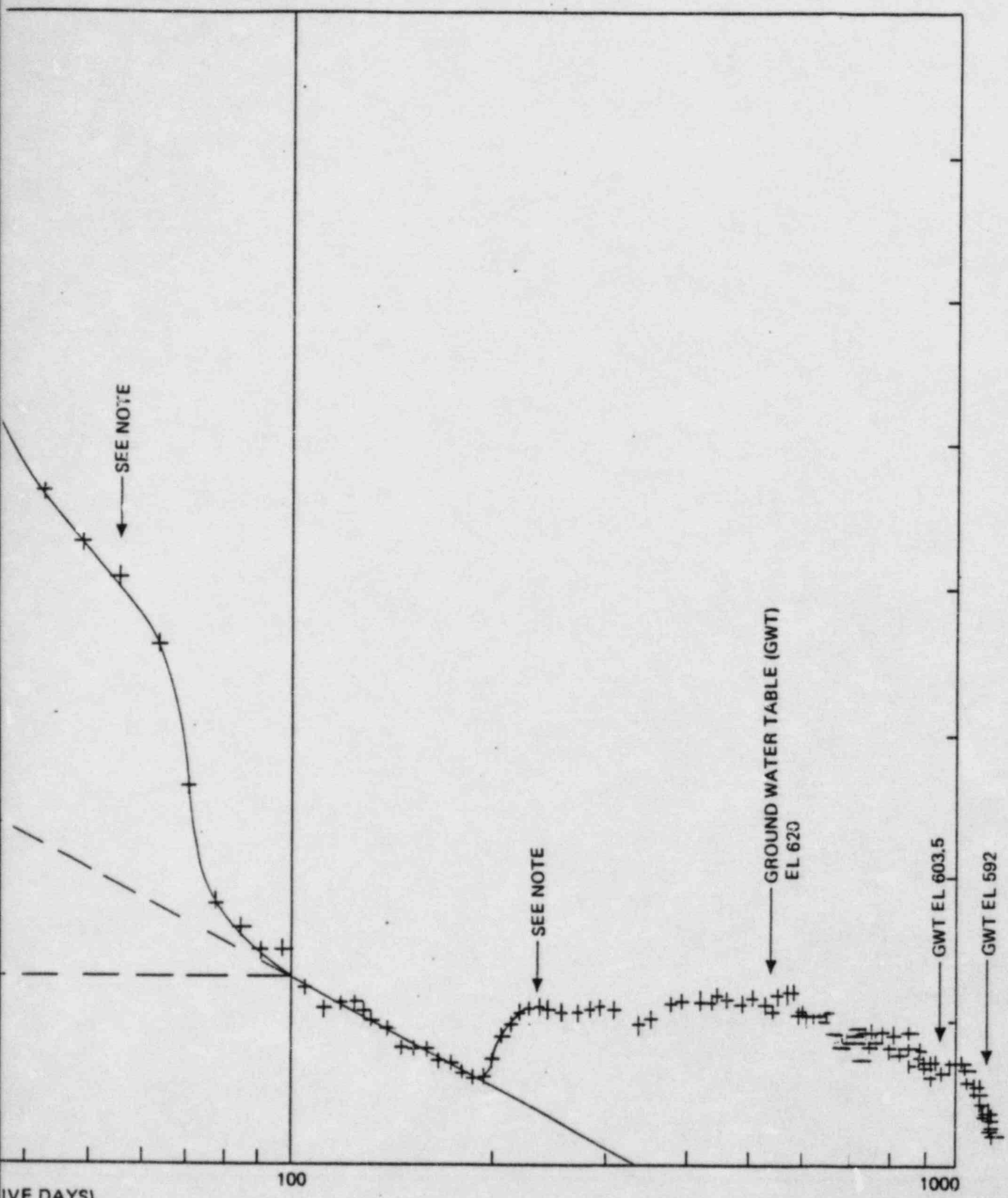
DIESEL GENERATOR BUILDING TYPICAL SETTLEMENT, COOLING POND LEVEL, PIEZOMETER LEVEL, AND SURCHARGE LOAD HISTORY

FIGURE I-2

G-1986-36



.50
0.0
0.50
1.00
1.50
2.00
2.50
3.00
3.50
SETTLEMENT (INCHES)
SETTLEMENT PER LOG CYCLE OF TIME C_α = 1.25"
1-26-79
1
10
TIME (CUMULAT

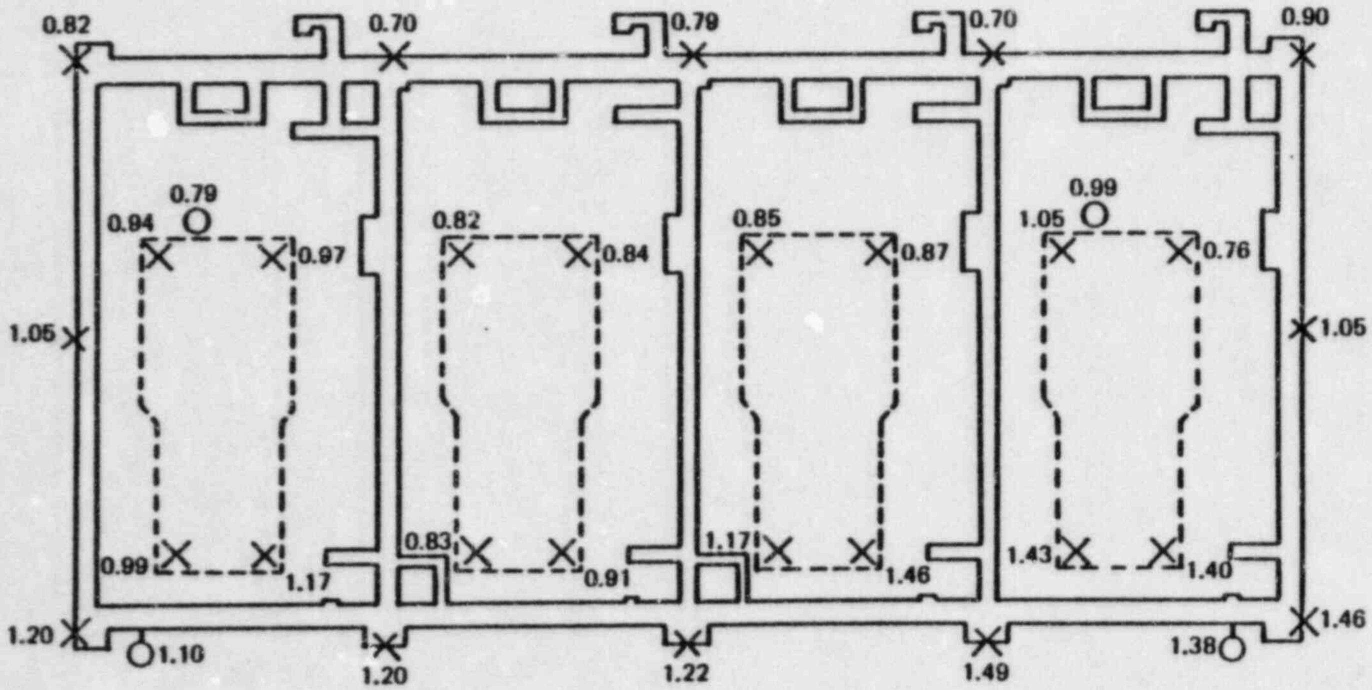


NOTE

The permanent marker could not be monitored from 3-22-79 to 9-14-79 due to surcharge. Temporary markers at elevation 664'-0" ± were used during this period to estimate the settlement of the permanent markers. On 9-14-79 the settlement was again based directly upon the permanent markers.

CONSUMERS POWER COMPANY MIDLAIND UNITS 1 AND 2
TYPICAL SETTLEMENT VS LOGARITHIM OF TIME DURING AND AFTER SURCHARGE (MARKER DG-3)
FIGURE I-3

DIESEL GENERATOR BUILDING



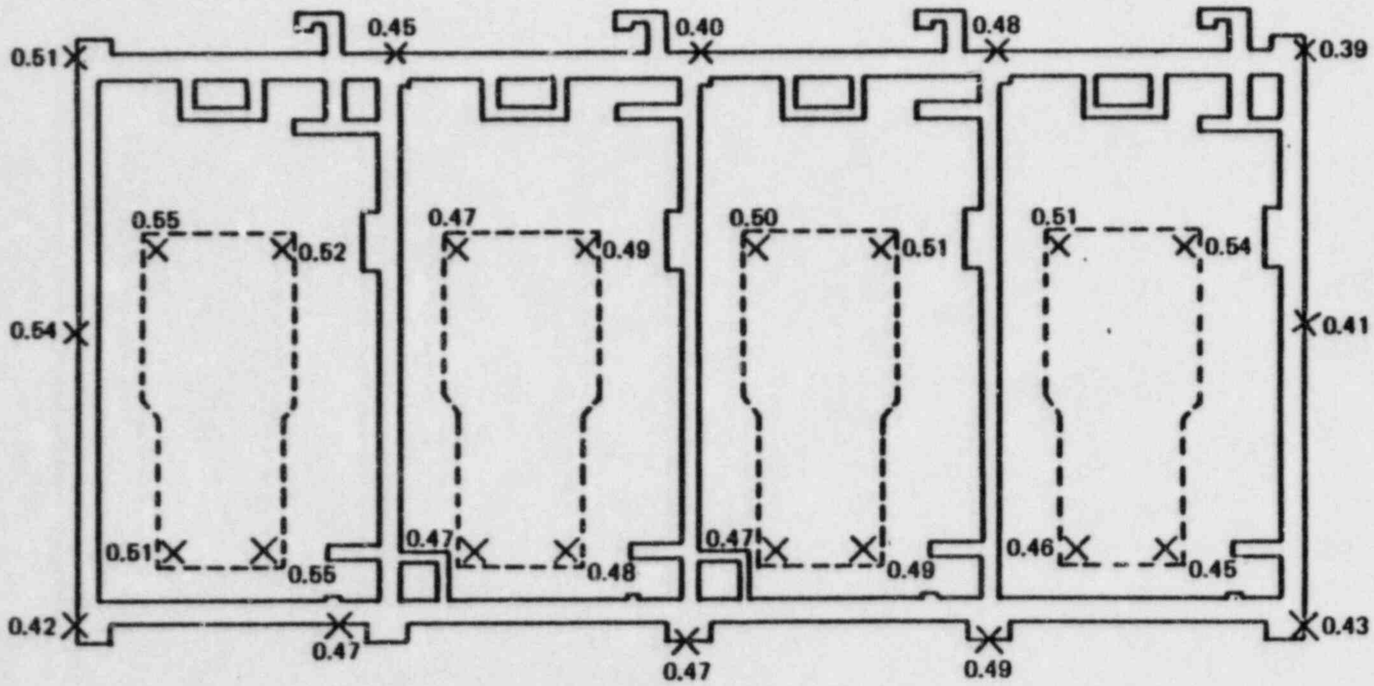
LEGEND

○ —— DEEP BORROS ANCHOR
X —— BUILDING / PEDESTAL SETTLEMENT MARKER
1.20 —— SETTLEMENT IN INCHES

CONSUMERS POWER COMPANY MIDLAND UNITS 1 AND 2
ESTIMATED SECONDARY COMPRESSION SETTLEMENTS FROM 12/31/81 TO 12/31/2025 ASSUMING SURCHARGE REMAINS
FIGURE I-4

G-1986-39

DIESEL GENERATOR BUILDING



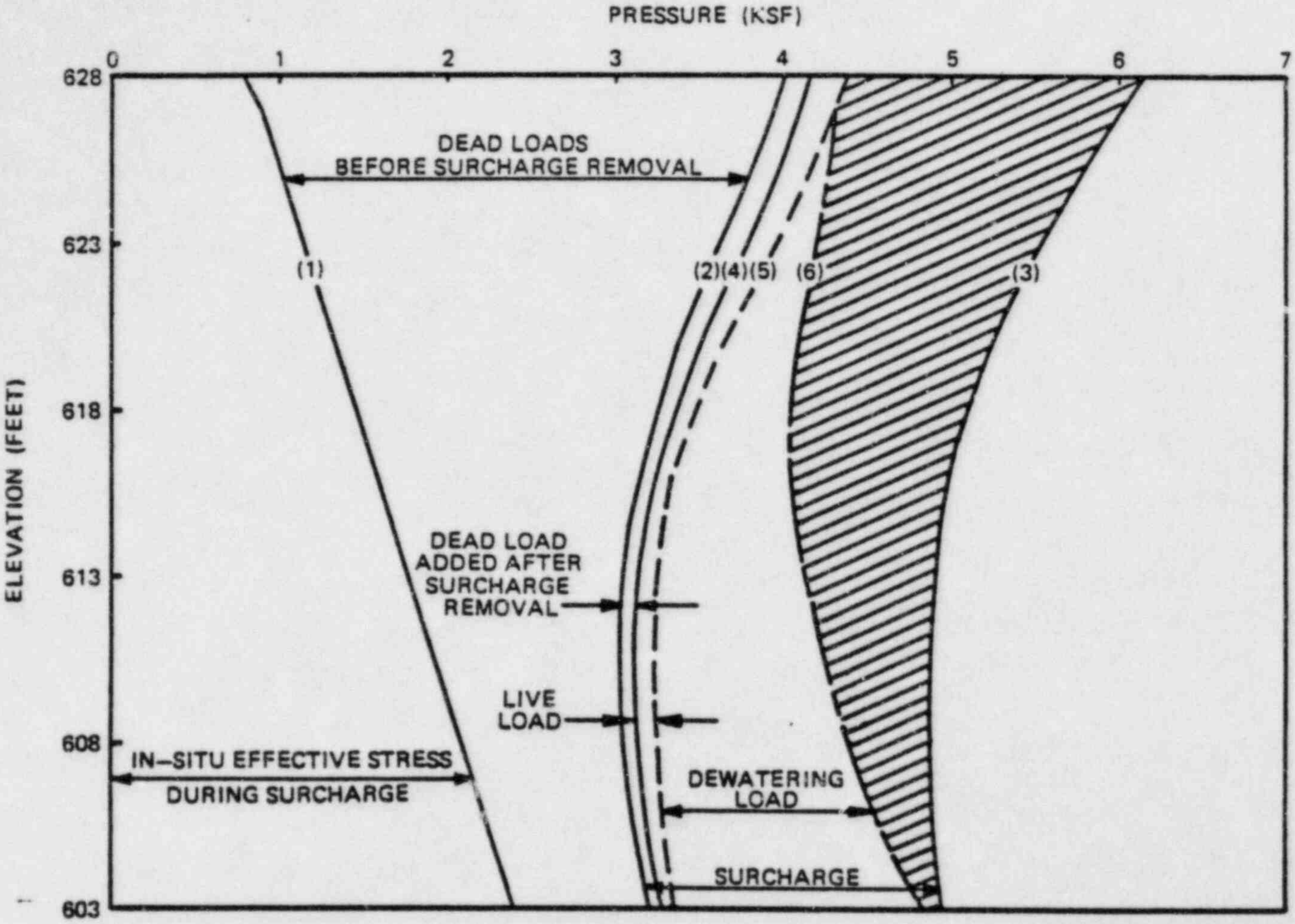
LEGEND

X ——— BUILDING / PEDESTAL SETTLEMENT MARKER

0.42 ——— MEASURED SETTLEMENT BETWEEN 9/14/79 AND 12/31/82.

CONSUMERS POWER COMPANY MIDLAND UNITS 1 AND 2
MEASURED SETTLEMENT S_2, FROM 9/14/79 TO 12/31/81
FIGURE I-5

G-1986-38



EXPLANATIONS

(1) In–situ effective overburden pressure (GWT at 627).

(2, . otal effective pressure before surcharge removal due to In–situ effective overburden pressure and structural dead loads present during surcharge.

(3) Total effective pressure at the end of surcharge due to In–situ effective overburden pressure, structural dead loads, and surcharge loads.

(4) Total effective pressure due to In–situ effective overburden pressure and total structural dead loads (loads present during surcharge plus dead loads added after surcharge removal).

(5) Total eff :tive pressure due to In–situ effective overburden pressurc, .otal structural dead loads, and expected live loads.

(6) Total effective pressure during the life of plant operation due to In–situ effective overburden pressure, structural dead loads, dewatering loads, and expected live loads.

CONSUMERS POWER COMPANY MIDLAND UNITS 1 AND 2
COMPARISON OF EFFECTIVE STRESS BEFORE AND AFTER SURCHARGE SOUTHWEST CORNER DIESEL GENERATOR BUILDING
FIGURE I-6

G-1986-37

PART II: AUXILIARY BUILDING AND FEEDWATER ISOLATION VALVE PIT

1.0 INTRODUCTION

The 1978 investigation of the plant fill revealed inadequately compacted fill under some areas of the auxiliary building and feedwater isolation valve pits (FIVPs).

The auxiliary building houses a number of safety-related systems, including control and fuel handling. The general arrangement and layout of this building is shown in Figures II-1 and II-2. The auxiliary building is constructed of reinforced concrete.

Parts of the auxiliary building foundations rest on plant area fill; namely, the railroad bay on the north side, the electrical penetration areas for Units 1 and 2, and the control tower on the south side. The rest of the auxiliary building is founded on natural material.

The FIVPs are symmetrically located at the sides of each containment building and are adjacent to the auxiliary building, electrical penetration areas, turbine building, and the buttress access shaft. Each pit is C-shaped with the open end in contact with, but structurally separate from, the containment building. Primarily, the pits enclose the Seismic Category I feedwater pipe isolation valves. The FIVPs for both Units 1 and 2 are founded on plant fill. Exhibit II-1 is a photograph of a scale model of the auxiliary building and shows subsurface conditions under the electrical penetration areas and control tower.

The inadequately compacted fill under the electrical penetration area of the auxiliary building and the FIVPs led to the need for remedial actions for these structures.

2.0 DESIGN CONCEPTS

As agreed upon with the NRC staff, remedial actions consist of the following (see Figures II-3 and II-4):

a. Installing a system of concrete walls below the existing foundations of the electrical penetrations areas and the control tower

b. Installing new concrete foundations for the FIVPs which rest on new compacted granular fill.

The new foundation system provides permanent underpinning that will transfer the load of the affected structure from the existing fill to undisturbed material.

2.1 IMPLEMENTATION OF PLAN

The structure, including the underpinning, has been analyzed for the loads from the building, the effects of the 40-year settlement of soil, and environmental effects such as earthquakes and tornados. The dimensiors and major details of the underpinning have been finalized based on a design which used the results of the analyses. The existing structure has been found to be adequate, based on these structural analyses and design. The supporting, undisturbed material also has been found to be adequate.

Before construction of the permanent underpinning can be started, temporary support for the electrical penetration areas and the control tower and lateral earth support are needed. This is the temporary underpinning system. It allows equipment to be used for mass excavation under the areas to be permanently supported.

The temporary underpinning consists of constructing concrete piers under the turbine buildings and installing temporary beams under the electrical penetration areas. The piers provide vertical support for turbine building column loads and support the south end of the temporary beams. They also retain earth during construction. Support for the north end of the temporary beams is provided by steel columns resting on the ledge of the reactor building foundation. The control tower is suppored by piers under the south wall and building columns. The piers are constructed by hand digging pits and filling these with concrete. After the pits are completed, the load is transferred by jacking.

To construct the temporary underpinning, which is below the existing foundations, access is needed from the present grade. Vertical access will be provided by two access shafts. Horizontal access, which is required for pit construction, will be provided by drifts (horizontal tunnels).

The construction of temporary piers and permanent underpinning must be done in a dry condition. Because the present dewatering system is not adequate to lower the groundwater to the bottom of the underpinning, an additional construction dewatering system is needed. This will be accomplished by constructing a freeze curtain dam around the area supplemented by additional dewatering inside the dam.

The freeze curtain dam is constructed by installing a network of vertical pipes in the ground connected to a common supply and return system. Chilled coolant is circulated throughout the system to freeze the ground in the area of the pipes.

After completing the temporary underpinning, mass excavation under the electrical penetration area, the control tower, and the FIVPs is accomplished. During this excavation, the temporary piers are tied by bracing to existing structures.

Completion of mass excavation provides the necessary access to construct the permanent underpinning. After the permanent, reinforced concrete underpinning is complete, the load is transferred from the temporary to the permanent underpinning. The underpinning is connected to the structure with dowels (see Figures II-5 and II-6). The excavations are backfilled with fill material and concrete. At this stage, the permanent foundation rests on undisturbed natural material and the underpinning operation is complete.

During the underpinning operations, extreme care must be taken to protect the existing structure. This is accomplished by removing only small portions of supporting soil during temporary underpinning installation, and replacing it with a temporary system with greater load bearing capacity. In addition, the structure is monitored frequently for movements to ensure that these movements are below predetermined limits.

2.2 LICENSING STATUS

The design concept for the auxiliary building underpinning has been presented and discussed with the NRC staff using several methods: technical reports, testimony for the Atomic Safety and Licensing Board (ASLB) soils hearings, design audits by the NRC staff, and technical meetings.

A technical report describing the underpinning was submitted on September 30, 1981. This was supplemented by responses to NRC staff requests for additional information on November 16, 1981, and by addendum on December 3, 1981. This provided preliminary analytical results. Specialized reports regarding the effects of cracking of concrete on the FIVP and the auxiliary building were submitted on January 25, 1982, and January 29, 1982, respectively.

Testimony presented at the ASLB soils hearings in December 1981 also provided the staff with information about the underpinning system.

Design audits were conducted in the Bechtel offices at Ann Arbor, Michigan, on three occasions: January 16 through 19, 1981; February 2 through 5, 1982; and March 16 through 19, 1982. During these audits, the staff reviewed in detail the design concepts and calculations for the temporary underpinning.

Meetings between the staff and the Applicant were held on October 1, 1981; November 4, 1981; and February 26, 1982; to discuss both the concept and details of the design. In addition, meetings were held December 10, 1981; and January 11, 1982; to specifically discuss effects of concrete cracking.

The design concept has received NRC staff concurrence.

3.0 STRUCTURAL ANALYSIS AND DESIGN

Structural analysis of the auxiliary building and its underpinning is performed in two parts:

a. A seismic analysis using a mathematical model to analyze the structure for the dynamic conditions during a seismic event

b. A static analysis, where the static loads imposed on the structure, such as dead load, live load, wind load, etc, are analyzed.

The loads from these two analyses are combined in accordance with applicable load combinations. Load combinations presented in Final Safety Analysis Report (FSAR) Subsection 3.8.6 and supplemented by the Responses to NRC Requests Regarding Plant Fill, Question 15, (Revision 3, September 1979) are used for the structure and the underpinning and its connections to the structure. Additional loading combinations based on American Concrete Institute (ACI) Code 349-76 and supplemented by NRC Regulatory Guide 1.142 are used for the underpinning and its connections to the structure.

3.1 SEISMIC ANALYSIS

A seismic model is developed to evaluate overall building response to seismic loadings as well as to generate in-structure response spectra for equipment design. The responses from this model provide input to other static analyses. The building is represented by a three-dimensional, lumped-mass stick model with plate elements used to represent the stiffness of the shear walls and underpinning in the electrical penetration area and control tower.

By NRC staff direction, the underpinning is designed to withstand the effects of the site-specific response spectra (SSRS) ground motion. The existing structure is evaluated for the effects of the plant's original design basis as stated in the FSAR ground motion description. In order to proceed with the underpinning design while NRC concurrence with the proposed SSRS was being obtained, the structural forces resulting from the FSAR safe shutdown earthquake (SSE) ground motion were multiplied by a factor of 1.5 for design of the underpinning. The response from a 1.5 times FSAR SSE envelops the final SSRS response.

The seismic analysis of the underpinned structure has been completed and the results are being used for the static analysis of the underpinning and reevaluation of auxiliary building equipment for seismic loadings.

3.2 STATIC ANALYSIS

3.2.1 Finite-Element Models

The superstructure and underpinning of the auxiliary building are analyzed by a finite-element method. The structure is analyzed for four conditions with four different finite-element models. Each model is briefly described below. The modeled conditions are:

a. Construction sequence of the proposed underpinning

b. Long-term loading without connecting the underpinning to the building

c. Long-term loading with full connection between the underpinning and building

d. Short-term loading with full connection between the underpinning and building

The models consist primarily of plate elements. Beam elements are used to represent columns, minor concrete elements, and major steel components of the structure. The nodal mesh is intensified in the areas significantly affected by underpinning. The soil subbase is represented by boundary springs placed under the foundation areas. The spring constants are based on appropriate soil response predictions as dictated by the load duration.

The underpinning is modeled as a continuation of the main shear walls in the control tower and the auxiliary building electrical penetration areas and extends the full length under these areas.

3.2.2 Construction Model

A construction sequence model reflects loadings on the structure during various stages of temporary underpinning. This model is used to investigate the construction sequence as the existing soil support of the structure is sequentially replaced by jacking loads.

Several variations of this model are utilized, modeling differences in the total number of boundary springs which are replaced by jacking loads. The temporary underpinning is reflected as a jacking load in this model. The spring constants for the boundary springs reflect the soil properties prior to underpinning. The load cases applied to the model include dead load, live load, jacking loads, external hydropressures, soil pressures, and wind loads.

3.2.3 Models for Long-Term Loads

3.2.3.1 Underpinning and Structures Disconnected

This model is used to investigate the effects of long-term loads with the underpinning disconnected from the superstructure. This model represents the construction stage when the superstructure and underpinning are separated by a series of hydraulic jacks and shims with the jacks and shims totally supporting the underpinned areas. Structural interaction is produced by placing upward jacking loads on the superstructure and placing equal and opposite loads on the underpinning.

The boundary springs have spring constants based on the predicted soil response to long-term loads. The load cases applied to the model are dead load, live load, external hydropressures, soil pressures, jacking loads, and wind loads.

3.2.3.2 Underpinning and Structures Connected

This model is used to investigate the effects of long-term loads with the underpinning fully connected to the superstructure. The load cases applied to the model include dead load, live load, soil and water pressures, and differential settlement loads. The differential settlement is considered in the model by calculating appropriate spring constants based on settlements.

Based on the properties of the natural materials, over the 40-year life of the underpinning, the settlement after construction is predicted to be 0.3 inch at the control tower and 0.2 inch in the electrical penetration area. The main portion of the auxiliary building is predicted to settle in the range of 0.1 inch to 0.5 inch. These predicted settlements are based on an investigation conducted by Woodward-Clyde Consultants (WCC), who performed soil borings and laboratory testing of the undisturbed natural materials. These tests show the preconsolidation pressure of the natural materials to be between 30 to 40 tons/sq ft.

3.2.4 Model for Short-Term Loads

This model is used to investigate the effects of short-term loads with the underpinning fully attached to the superstructure. The spring constants for the boundary springs are based on the predicted soil response to short-term loads. The load cases applied to the model are east-west earthquake, north-south earthquake, vertical earthquake, tornado, wind, and pipe rupture loads.

3.3 DESIGN

The results of the structural analyses are factored and added in specific combinations to evaluate the structural adequacy of the structure and underpinning. This verification ensures that computed stresses and loads will be lower than or equal to the allowable stresses and capacities.

3.3.1 Temporary Underpinning

Salient design features of the temporary support system include (see Figure II-8):

a. Steel frames, as shown in Figure II-9, supporting the FIVPs.

b. Thirty-six concrete piers at the north end of the turbine building - These piers support the turbine building column load on Column Lines K and K_C and also retain soil under the turbine building basemat. These piers are permanently left in place. The piers are braced with struts and tie rods to transmit lateral loads to the containment wall.

c. Three frame supports under each electrical penetration area - Each frame support consists of a concrete pier, needle beams, and steel columns supported on the reactor building foundation slab or on another concrete pier (see Figure II-10). These frames also support part of the turbine building load.

d. Ten concrete piers under the south side wall of the control tower - These piers are a part of the underpinning wall for the control tower. Struts are provided to transmit lateral loads from the soil under the turbine building to the auxiliary building.

e. Additional concrete piers under each of the three existing steel columns inside the control tower - These piers are part of the permanent underpinning.

f. Two concrete piers below each buttress access shaft - These support the reaction load from the temporary steel frames which support the FIVPs and retain soil under the buttress access shaft. These piers are permanently left in place.

g. Tunnels under the turbine building and access drift tunnels - These tunnels and drifts are constructed by the usual construction methods utilizing lagging and steel frames.

h. Temporary post-tensioning - The temporary dewatering system removes the buoyancy force normally provided by groundwater under the electrical penetration areas. To compensate for this effect during construction, a temporary system of post-tensioning ties is installed to apply a compressive force to the upper part of the east-west walls of the electrical penetration areas. The post-tensioning ties are removed when the temporary supports are installed and jacking loads are applied under the electrical penetration areas.

The temporary support system is designed to resist the calculated imposed loads using ACI and American Institute of Steel Construction (AISC) codes.

3.3.2 Permanent Underpinning

Design features of the electrical penetration and control tower areas are (see Figures II-3, 4, 5, 6, and 7):

a. The proposed underpinning for the Unit 1 and 2 penetration areas are a 6-foot thick, reinforced concrete wall 38 feet high belled out to 10 feet thick at the bottom. The belling limits bearing pressures to the allowable values. The underpinning walls under the control tower are 6 feet thick, 41 to 47 feet high, and are belled out to 14 feet thick. The walls are constructed to act as a continuous member under the perimeter of the structures. Individual piers are provided to underpin interior columns of the building. The entire wall and pier system is founded on undisturbed natural material.

b. Allowable bearing pressures for the undisturbed natural material is based on a safety factor of 2 for dynamic loading and 3 for static loading. The ultimate bearing capacity for the natural material is based on the undrained triaxial tests performed on the WCC boring samples. These yielded a median shear strength of 7.6 ksf.

c. A design jacking force is applied to the existing structure to provide adequate load transfer from the structure to the permanent underpinning. These jacking forces transmit the structural loads through the permanent underpinning wall to the bearing stratum.

d. Dowels connect the underpinning walls and the existing structure at the vertical and horizontal interfaces. The dowels are designed to transfer shear and tension forces between the structure and the underpinning wall.

These dowels are connected after the permanent load transfer is accomplished.

3.4 LICENSING STATUS

The structural analysis for the underpinning was presented in technical reports, ASLB hearing testimony, design audits, and meetings as previously indicated in Section 2.2.

The seismic analysis was covered in detail during testimony by Dr. R.P. Kennedy of Structural Mechanics Associates (SMA) and Dr. P. Hadala of the U.S. Army Corps of Engineers (representating the NRC staff) during the ASLB soils hearings of December 14, 1981.

As indicated in Section 2.2, three design audits have been performed by the NRC staff. During these audits, structural design calculations for the temporary underpinning and the resulting structural stresses have been reviewed in detail.

Preliminary analysis of the permanent underpinning has been completed and the results presented to the staff. Analysis of the temporary underpinning also has been completed and audited by the staff. Analysis of the final underpinning is being completed and when finished will be presented to the NRC staff.

Design of the temporary underpinning is complete and has been presented in technical reports, meetings, and design audits. Drawings are being issued for construction. Start of construction is currently awaiting NRC concurrence and is scheduled for May 1982.

As directed by the NRC, the Applicant is performing a parametric analysis by varying the subgrade reaction modulus for the till under the auxiliary building to a value of 70 kcf. The Applicant also will perform, at the NRC's direction, an analysis of the electrical penetration area for the effects on existing soil support caused by the adjacent access tunnel under the turbine building. A confirmatory load test on the bearing stratum will be performed.

4.0 CONSTRUCTION SUPPORT PROGRAMS

4.1 GROUNDWATER CONTROL

At the start of underpinning work it is anticipated that the groundwater level will be at about el 600'. Because this work will extend at least 29 feet below that level, the control of groundwater level will be an important prerequisite for successful completion.

The underpinning work is in a location with limited access, bounded by the two containment buildings, the main auxiliary building, and the turbine building. In the immediate construction area, groundwater will be removed by pumping from dewatering wells.

To reduce recharge of groundwater into this narrow area, an underground freeze curtain dam will be constructed. The proposed layout of the dam is shown in Figure II-11. The dam will be formed by drilling a line of boreholes at approximately 4-1/2-foot spacing and circulating glycol coolant at low temperatures through pipes in the boreholes. The coolant will freeze the soil in a narrow strip along the line from el 610' down to the undisturbed glacial till. The frozen soil will act as a dam and reduce subsequent seepage of groundwater from the pond side toward the underpinning construction area. The freeze curtain dam will be formed in permeable sandy soil that exists above the glacial till and below el 610'. The actual extent of these sandy soils will be determined by the initial borehole drilling.

The existing clay cutoff dike along the western edge of the power block will form a part of the underground dam. The effectiveness of the dewatering system will be monitored by measurements of the groundwater levels using piezometers located in the work area.

Design of the groundwater control system is complete and has been presented to the NRC staff in a technical report, meetings, and audit discussions. NRC concurrence has been received for installation and activation of the groundwater control system. Installation is approximately 75% complete. The safety-related utilities crossing the freeze curtain dam will be isolated by excavating so that they are unaffected by any potential heave of the ground due to freezing operations.

4.2 ACCESS SHAFT

Immediately east and west of the two FIVPs and adjacent to the turbine building, shafts are being constructed to provide access for workers and equipment for the underpinning work. The location of the west access shaft is shown in Figure II-12. The east access shaft will be symmetrically located. Each shaft will be about 16 feet by 26 feet in clear plan dimensions.

The shafts will be excavated in three phases. Initially, they will be excavated to el 609' to permit installation of the initial underpinning piers beneath the adjacent turbine building basemat. These piers will constitute permanent underpinning for the turbine building. When the initial turbine building underpinning is completed, the access shafts will be lowered to el 600' to provide access for excavation beneath the FIVPs.

After all temporary underpinning is completed for the FIVPs and electrical penetration areas, the two access shafts will be gradually lowered from el 600' to el 571'. At that time, a level working surface extending into the shafts will be constructed for the general excavation and removal of soil down to el 571' beneath the FIVPs, electrical penetration areas, and control tower.

The shafts will be constructed using standard methods and utilizing soldier piles, wales, and lagging.

The access shaft design is complete and has been presented in a technical report, meetings, and the audit of January 18 through 20, 1982. NRC concurrence has been received for installation to el 609' and this installation is complete. El 609' is the foundation level of the FIVP, auxiliary building, and turbine building.

5.0 CONSTRUCTION PROGRAMS

5.1 TEMPORARY UNDERPINNING

In order to construct the permanent underpinning, it is necessary first to install a temporary underpinning system to support the FIVPs and portions of the turbine building, electrical penetration areas and the control tower. The temporary underpinning system is shown in Exhibit II-2, which is a photograph of a scale model.

The following is a summary of the construction sequence of the temporary underpinning on the east side. The sequence for the west side is similar. The layout and the identification numbers of the underpinning system are shown in Figures II-8 and II-9.

The initial effort for the temporary underpinning was to construct access shafts to el 609'. This is the bottom of the turbine building and electrical penetration area foundations. It is also necessary to support the FIVP with steel framing. The purpose of these activities is to obtain access to the initial turbine building supports. Construction of both of these activities has been completed.

The next step will be to provide support to the turbine building near the electrical penetration area by constructing Piers E-9 and E-12. Before constructing these piers, the freeze curtain dam, which is near completion, will be activated to control groundwater. The completion of these turbine building piers is necessary to construct the tunnel/drift under the turbine building and to access the first support, Pier E-8, for the electrical penetration area.

Pier E-8 will be completed next and the first excavation under the electrical penetration area will be begun to install the needle beams needed to provide the first support for the electrical penetration area (see Figure II-10). The completion of Pier E-8 and the needle beams is very important to the temporary underpinning operation because after their completion, the entire weight of the electrical penetration area can be supported and any loss of soil support under the electrical penetration area is no longer critical. With Pier E-8 and the needle beams in place, the tunnel under the turbine building can be extended to access the first corner Pier E-1 of the control tower. While extending the tunnel, additional piers on Column Line K_c, to support the turbine building columns, are constructed.

The corner Pier E-1 of the control tower will be completed and jacked next. The completion of the control tower corner piers is crucial because after this the remaining control tower and electrical penetration area temporary underpinning piers can be simultaneously constructed.

With completion of the temporary underpinning piers, the weight of the electrical penetration area and control tower can be completely supported and the mass excavation under the electrical penetration area and control tower can begin. For performing the mass excavation, the access shaft will be extended to el 571'.

With completion of the mass excavation, the permanent underpinning can be started.

5.2 PERMANENT UNDERPINNING

A continuous underpinning wall resting on undisturbed natural material will be provided under the control tower and the electrical penetration area exterior walls. Also, a new concrete foundation resting on new concrete, which, in turn, is set on new compact granular fill, will be provided for the FIVPs. This underpinning provides the necessary vertical and horizontal support to the affected part of the structure. The details of the permanent underpinning are shown in Figures II-3, 4, 5, 6, and 7.

A summary of the construction sequence for the permanent underpinning follows.

After the completion of mass excavation, the permanent wall under the electrical penetration areas and the permanent section of the wall in the control tower area can be constructed. At this stage, compacted backfill will be placed below the FIVP area and a new slab will be poured at el 600'.

After completion, jacks will be placed on the wall. Jacking forces will be transferred from the temporary to permanent walls in stages. Adjustments will be made until all the load is transferred from the temporary to the permanent underpinning and the wall has reached the final design jacking load. The slab under the FIVP foundation also will be jacked against the FIVP to transfer the load from the temporary steel support to the new slab.

Jacking loads will be held on the permanent underpinning and the settlements monitored. When the settlement rate has reached a predetermined value, the jacking load will be locked off. The permanent underpinning walls will be connected to the existing structure by grouting and the gaps filled with grout. For the FIVP, the area between the new slab and the FIVP existing foundation slab will be filled with lean concrete. At this stage, the excavation will be backfilled with fill or lean concrete and the permanent underpinning will be complete.

The design of the underpinning is complete to the preliminary safety analysis report (PSAR) level and has been presented in the technical report and in meetings. NRC concurrence to proceed with construction has not been received.

There are no unresolved issues regarding the permanent underpinning and an operating license level design audit will be conducted by the NRC staff.

5.3 BUILDING MODIFICATIONS

Preliminary analysis indicates that strengthening may be required for one area of an existing slab at el 659' for certain loading combinations, including seismic loads. This area is between the control tower and spent fuel pool at the operating floor level. Detailed analysis is being performed to resolve this concern.

Because this strengthening, if required, is needed only to resist loads during a seismic event, it is not required prior to or during underpinning but will need to be installed prior to fuel load. The present plan is to finalize the design for this strengthening, if required, after the final analysis of the building and underpinning is completed.

6.0 MONITORING PROGRAM

To ensure that installation of the underpinning system is proceeding within acceptable limits, a monitoring program will be implemented during construction. This program has three parts: building movement and strain, cracking, and underpinning.

6.1 BUILDING MOVEMENT AND STRAIN MEASUREMENT

The underpinning methods to be used require that the soil be removed in small, discrete units and that these units be replaced with load bearing units of greater capacity than the unit that was removed. Discrete units are removed and replaced progressively, according to a predetermined plan, in a manner that will maintain the stresses in the structure below allowable limits.

Two systems will be used for detecting vertical and horizontal movements of the auxiliary building. The first system is for detecting movement of the reactor containment, auxiliary building, and turbine building with respect to a fixed datum. The second system is for detecting relative movement of the auxiliary building to the other structures.

The first system consists of seven deep-seated benchmarks to serve as reference points for measuring movement of the free ends of the electrical penetration areas, the east and west ends of the control tower, and the main auxiliary building. Movement will be measured with dial gages and electronic linear variable differential transducers (LVDTs). The precision of this instrumentation is ±0.001 inch and the accuracy is ±0.005 inch.

The second system will measure relative vertical movement between the structures described above by means of dial gages and LVDTs. Those relative readings will have an accuracy of ±0.005 inch. In addition, movements of the FIVPs will be monitored using LVDTs and one deep-seated benchmark in each pit.

Because of direct reading and high precision, the benefit of the movement measurement system is that data is readily produced for sensing differential movements and developing trends.

Relative horizontal movement will be measured at vertical measurement locations with relative movement dial gages and LVDTs. In addition, relative horizontal movement between the turbine building and auxiliary building will be measured at the roof level of these two structures.

Strains will be monitored in critical areas, which include the slab at el 654', the walls at el 614', and the connection of the electrical penetration area and control tower roof. Additionally, selected steel beams at el 659' will be provided with strain gages.

6.2 CRACKS

6.2.1 Existing Crack Evaluation

The existing cracks in the control tower, electrical penetration area, and FIVPs have been monitored. The size and location of existing cracks have been recorded on crack map drawings. The Applicant's consultant, Portland Cement Association (PCA), evaluated the structural significance of these cracks based on its site visit and review of the crack maps. The consultant concluded that all cracks are attributable to restrained volume changes that occur during curing and drying of concrete. PCA also did not observe any structural distress during the visit.

The consultant's evaluations and conclusions are contained in reports submitted to the NRC staff on January 25 and 29, 1982.

6.2.2 Crack Monitoring During Underpinning

Existing cracks will be monitored for changes in length and width during various phases of construction. The areas containing cracks will be inspected for new cracks that, if present, will be similarly mapped and monitored.

Because of the sequence of construction procedures, it is not anticipated that existing cracks will significantly widen or that significant new cracks will appear. However, any new structural cracks exceeding 0.01 inch in width or any crack exceeding 0.03 inch in width will be evaluated by PCA to determine whether underpinning operations should stop or continue. If development of yield strain is inferred from any observed crack, underpinning will be stopped and an evaluation made by PCA before continuing underpinning operations.

6.2.3 Repair of Cracks

A report on a crack repair program by PCA for all cracks in all structures will be submitted to the NRC staff in the near future.

6.3 UNDERPINNING

During underpinning installation, each temporary pier will be instrumented to monitor deflection of the pier tops and bottoms. Pier top movement will be monitored with readings taken between the underside of the foundation slab and the pier top. Monitoring will begin after pier concrete is placed and will include measurements during and after initial jacking. In addition, the underpinning wall movements will be similarly monitored.

Pier and wall bottom movement will be monitored by a rod attached to a plate at the base of the underpinning. The rod will be greased and enclosed in a small diameter pipe sleeve. The rod and sleeve will extend to the top of the pier before the pier concrete is placed. Rod movements will be recorded by dial gage extensometers simultaneously monitoring the movement of the pier or wall top. These instruments produce measurements relative to the position of the base slab. Absolute top and bottom movement values can be obtained by adding the measurements of movement, if any, of the base slab obtained from the deep benchmark monitoring.

The instrument readings for the movement of the pier base and top will be compared to anticipated values for creep and shrinkage of concrete and for the soil settlement. Actual values will be compared to expected values to determine when the final jacking loads can be locked off.

Carlson gages will be used to measure loads in selected temporary piers.

6.4 LICENSING STATUS

The design of the monitoring program is complete and was presented in a technical report, the meeting of February 26, 1982, and design audits. NRC concurrence has been received for installation and operation. Installation is currently in progress.

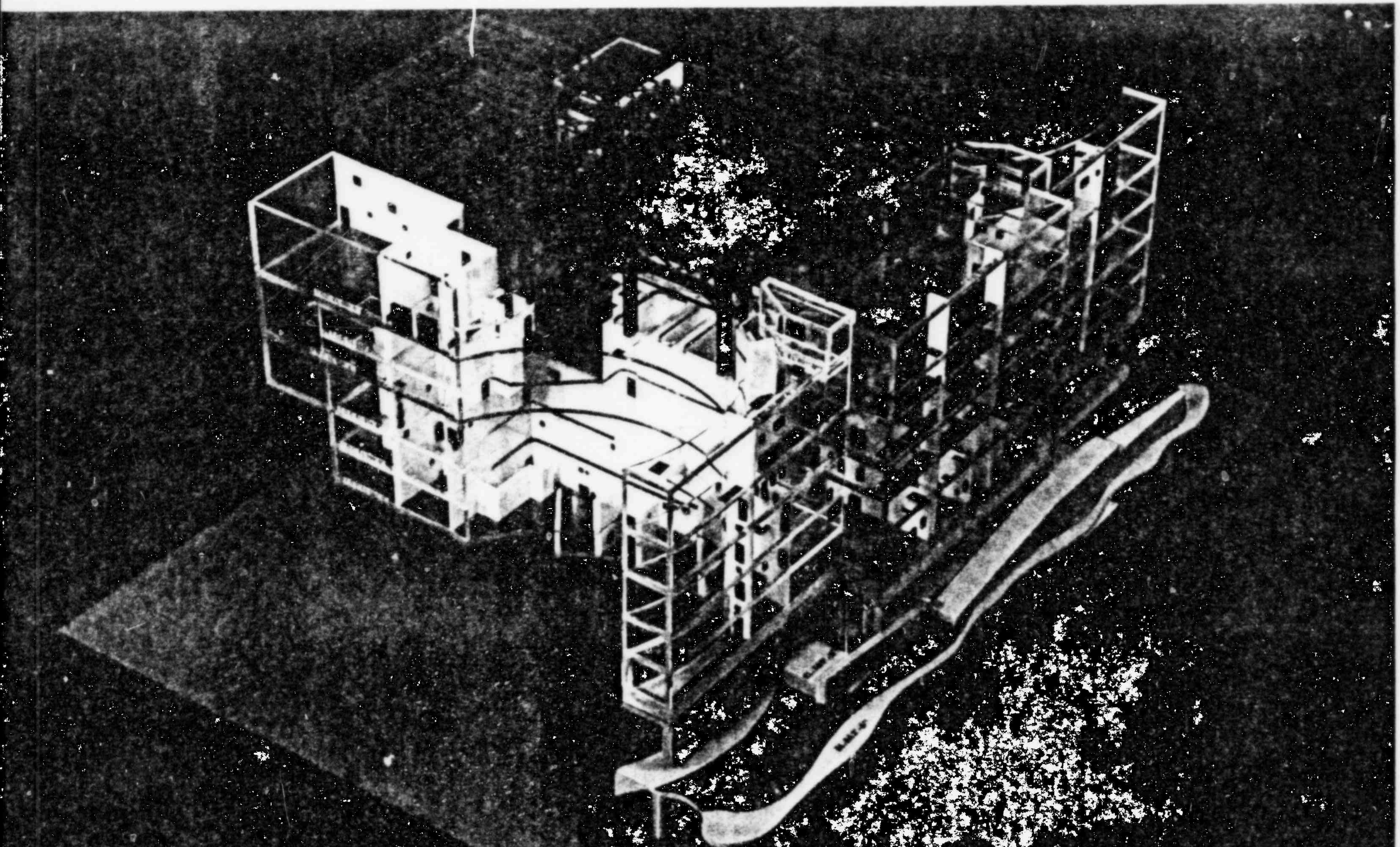


EXHIBIT II-1 -
MODEL OF EXISTING STRUCTURE

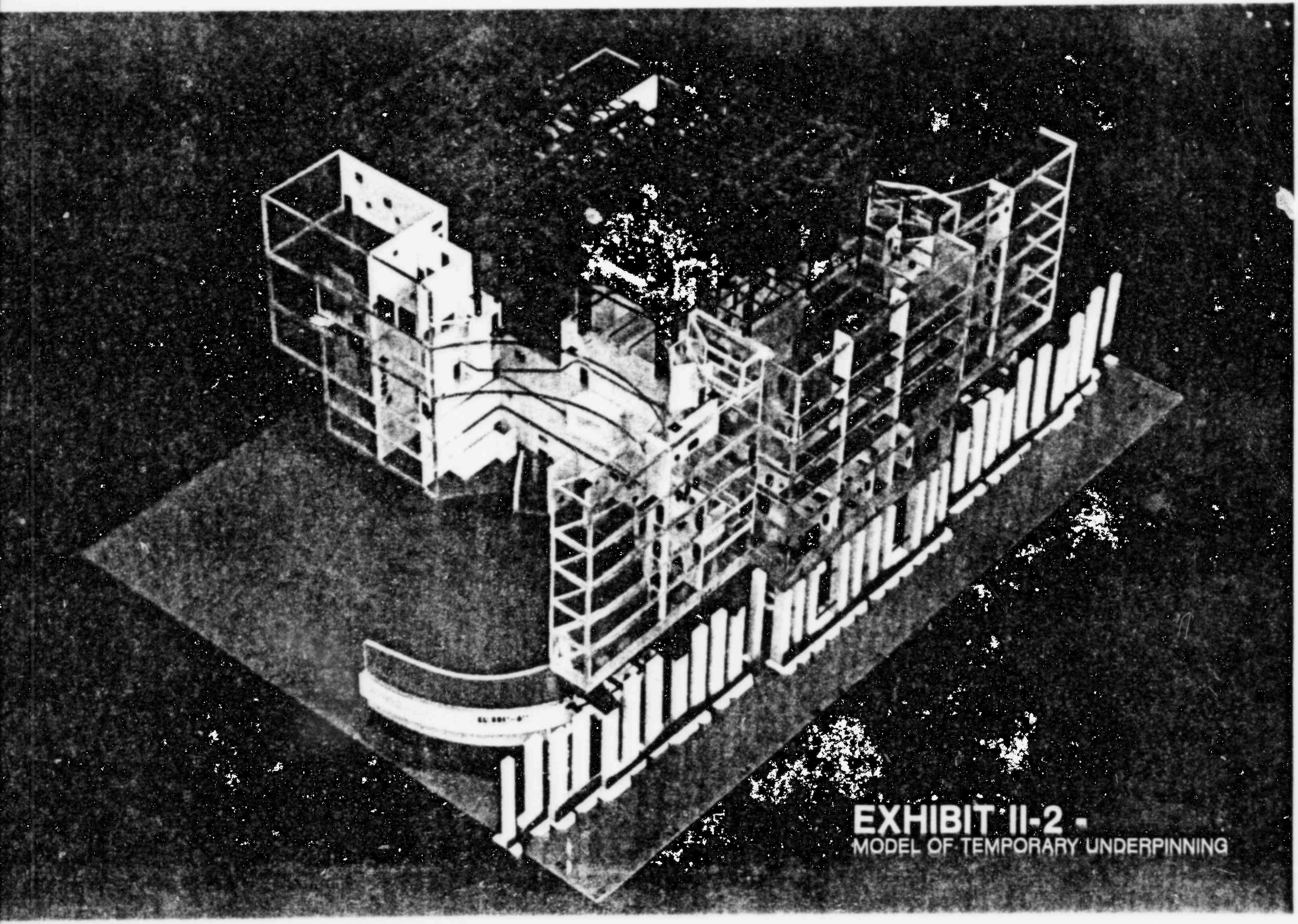


EXHIBIT II-2 -
MODEL OF TEMPORARY UNDERPINNING

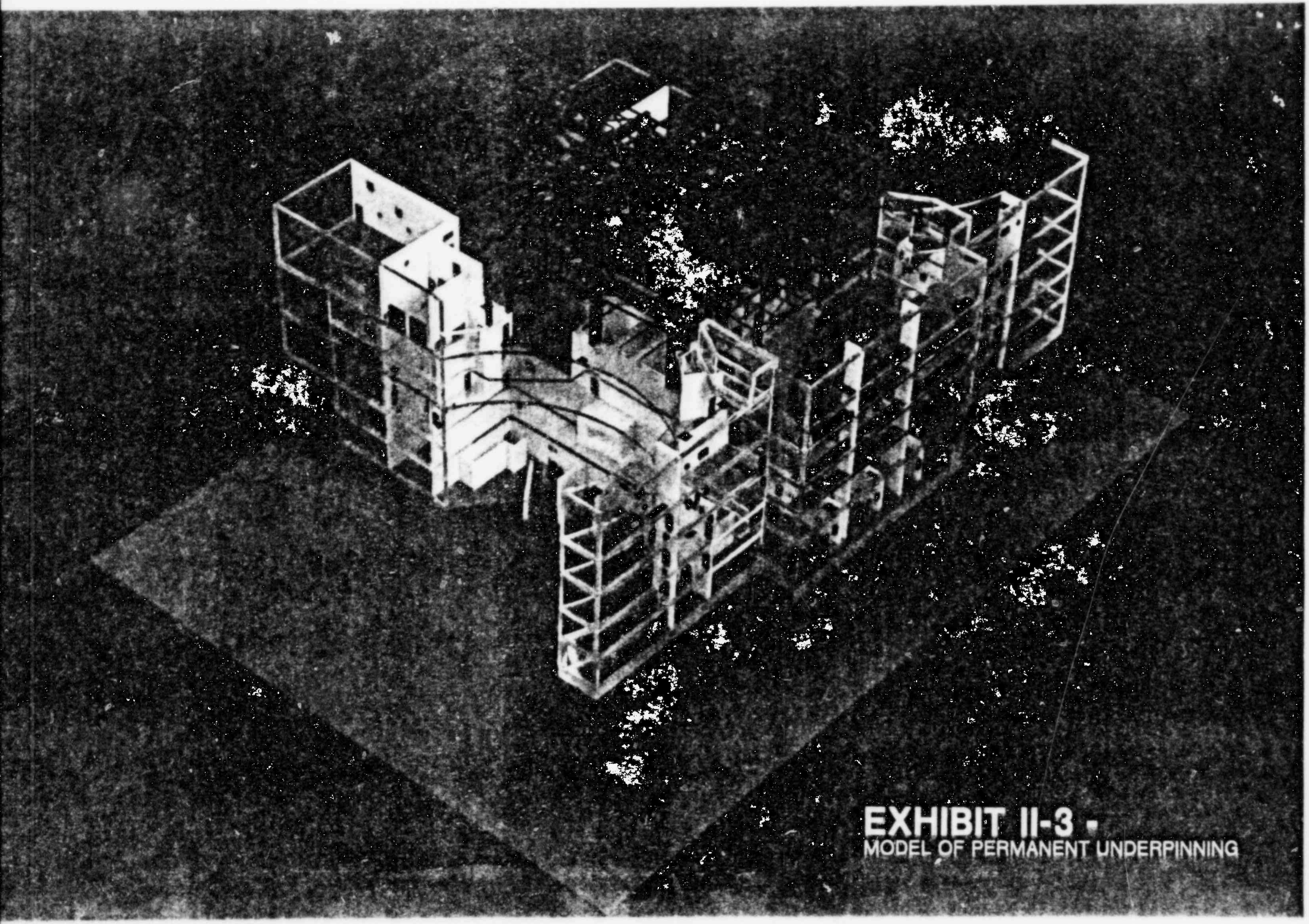
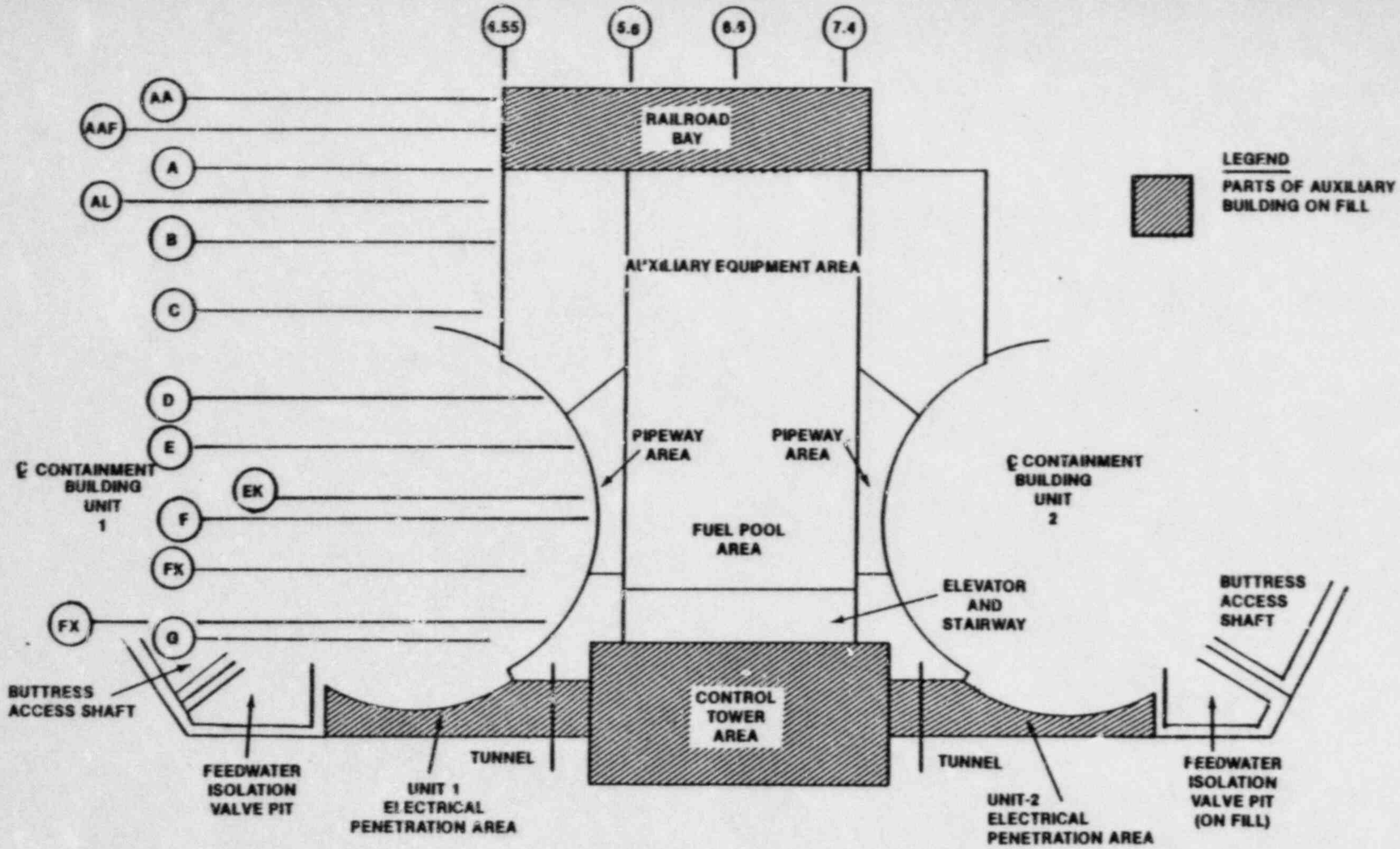


EXHIBIT II-3 -
MODEL OF PERMANENT UNDERPINNING

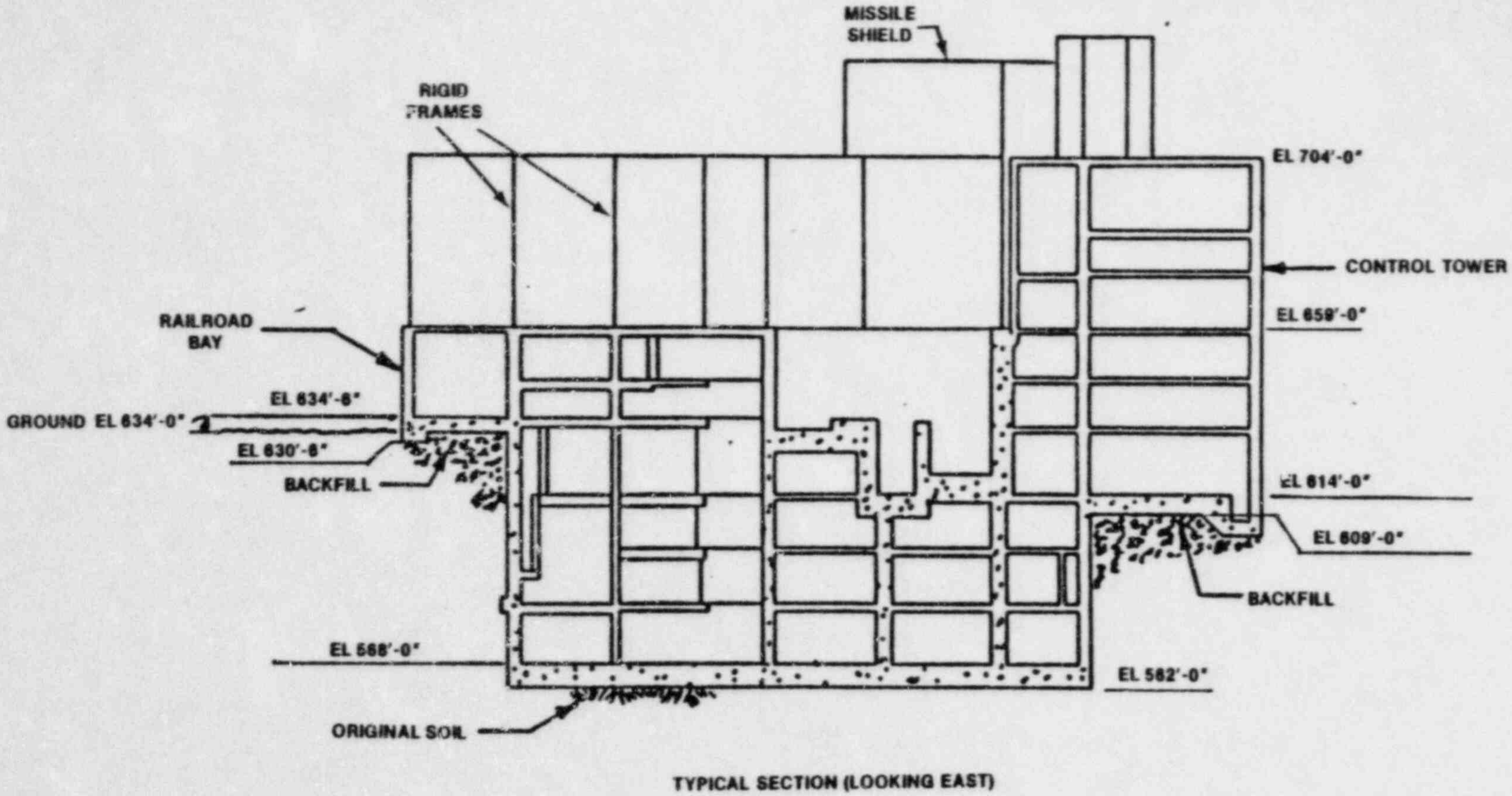


CONSUMERS POWER COMPANY
MIDLAND UNITS 1 AND 2

AUXILIARY BUILDING PLAN

FIGURE II-1

G-1986-12

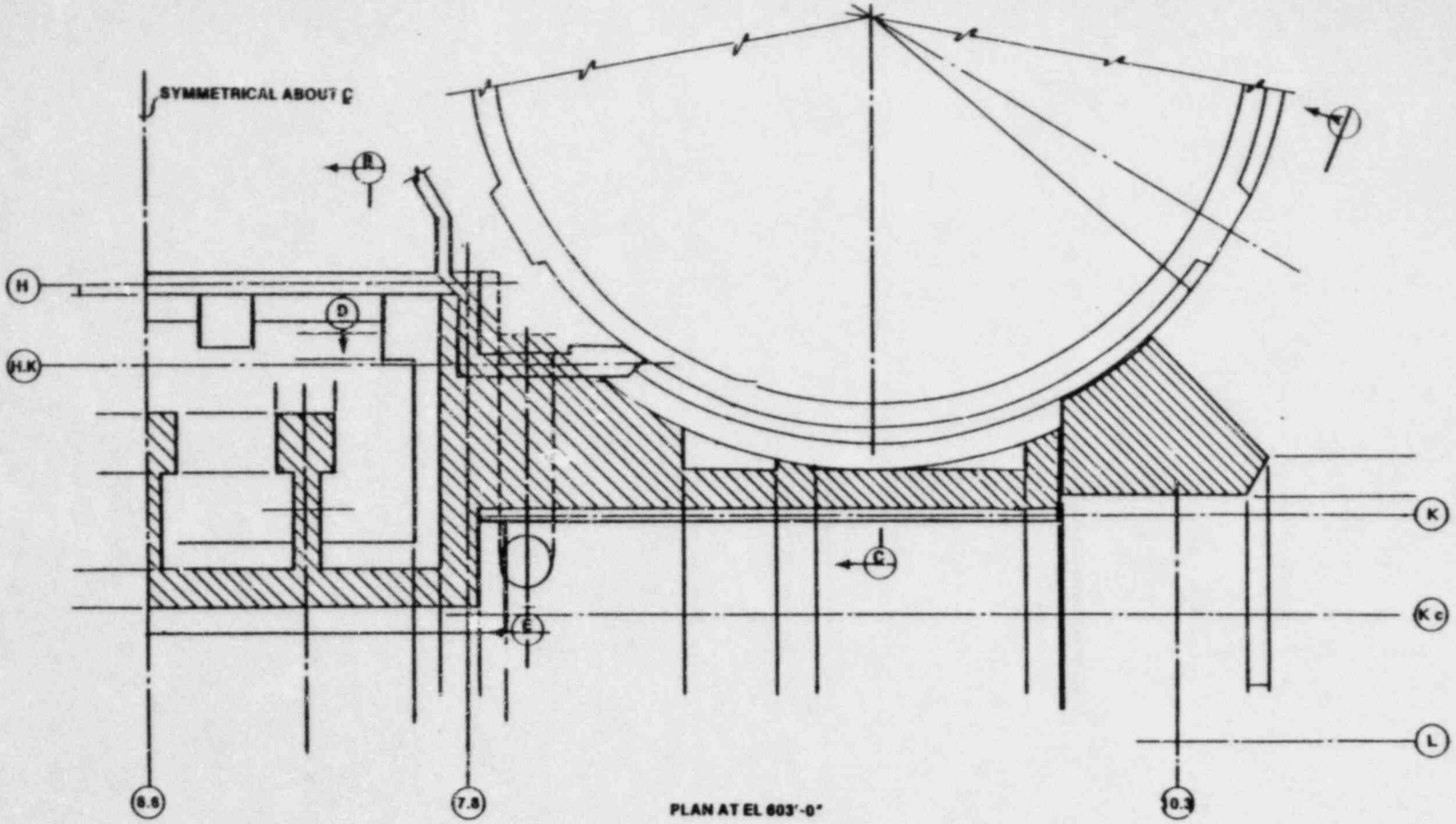


CONSUMERS POWER COMPANY
MIDLAND UNITS 1 AND 2

AUXILIARY BUILDING SECTION

FIGURE II-2

G-1986-10



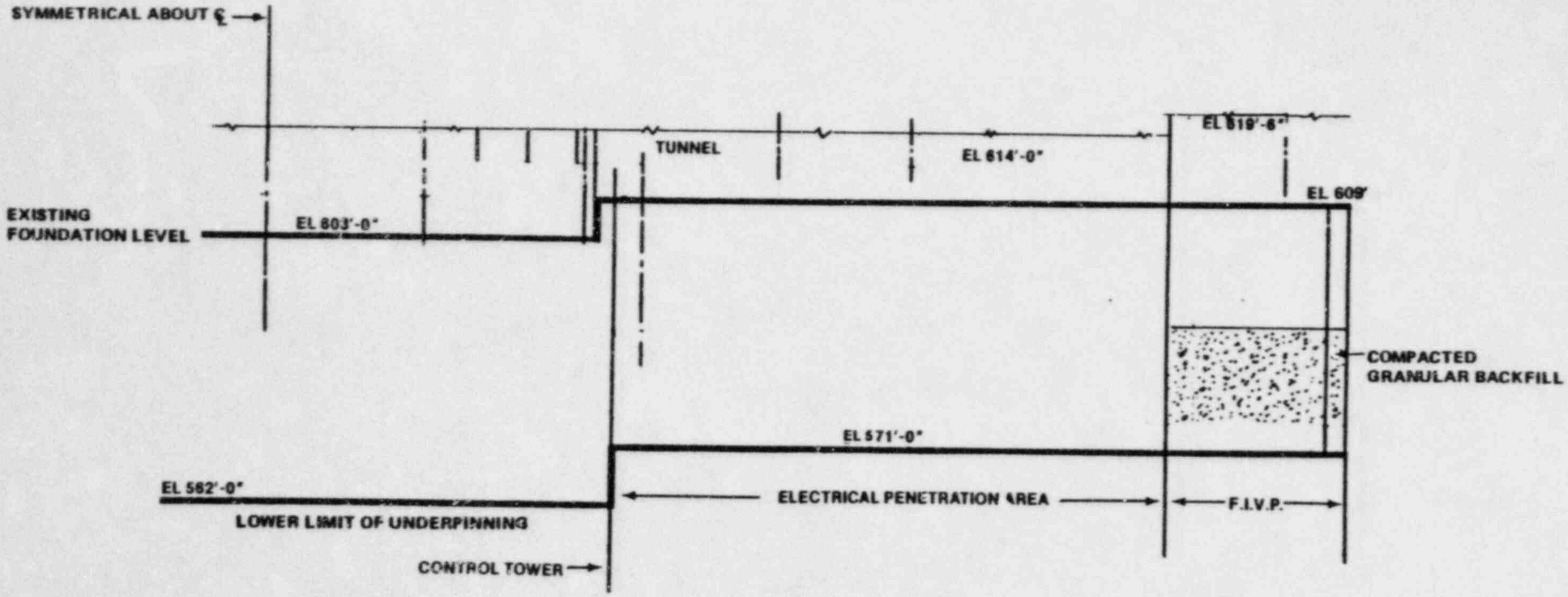
G-1986-08

CONSUMERS POWER COMPANY
MIDLAND UNITS 1 AND 2

UNDERPINNING PLAN AT
ELEVATION 603'

FIGURE II-3

G-1986-08

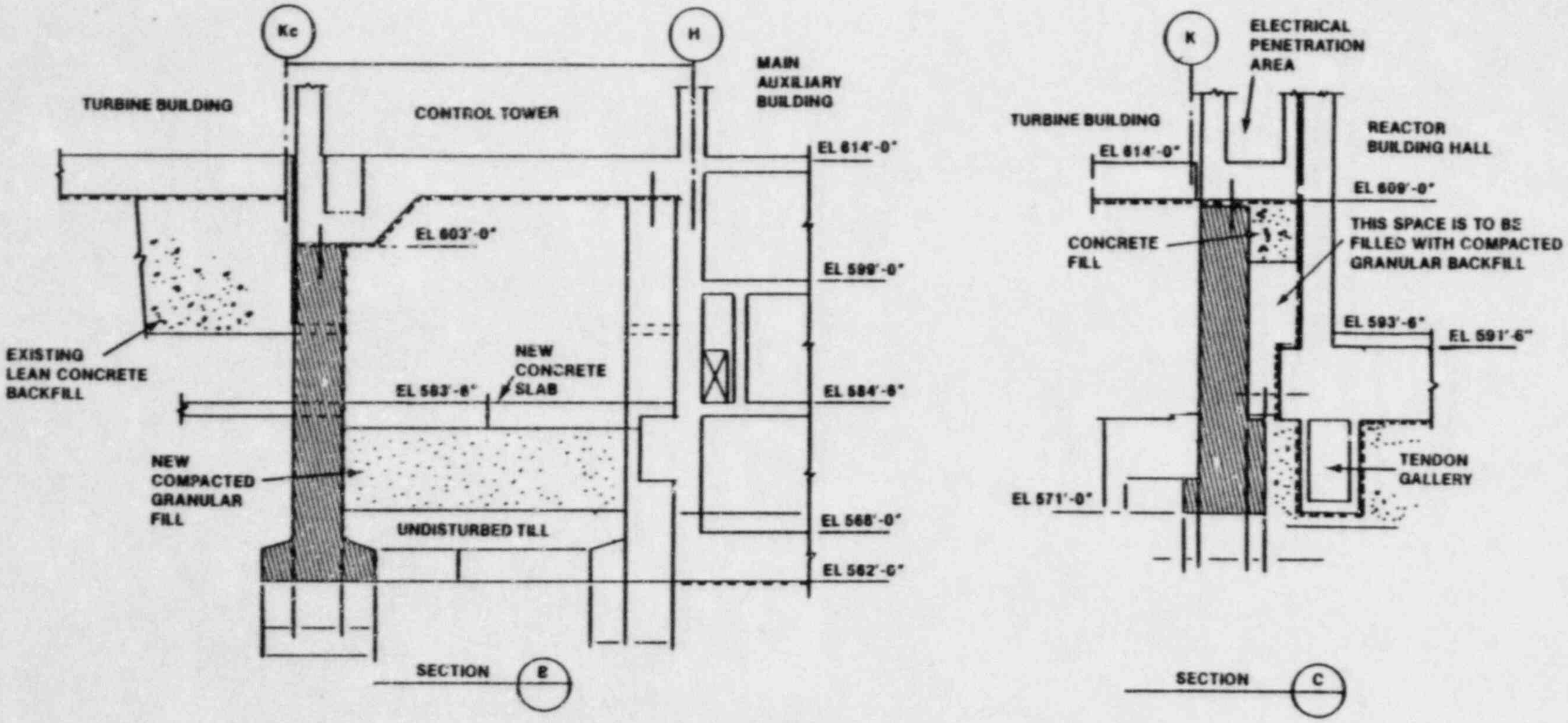


CONSUMERS POWER COMPANY
MIDLAND UNITS 1 AND 2

ELEVATION OF
UNDERPINNING WALL

FIGURE II-4

G-1986-07

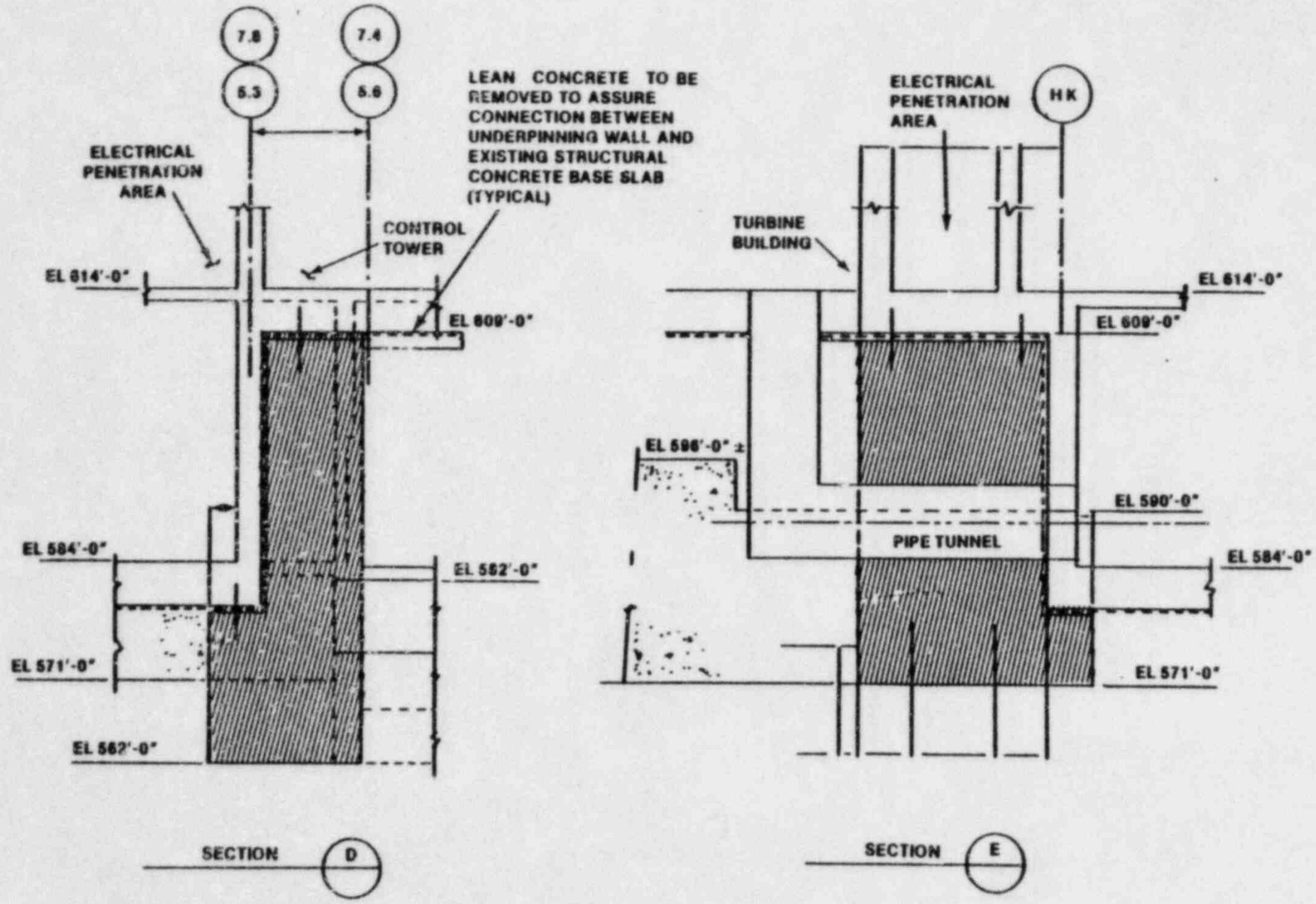


CONSUMERS POWER COMPANY
MIDLAND UNITS 1 AND 2

UNDERPINNING WALL SECTIONS

FIGURE II-5

G-1986-05

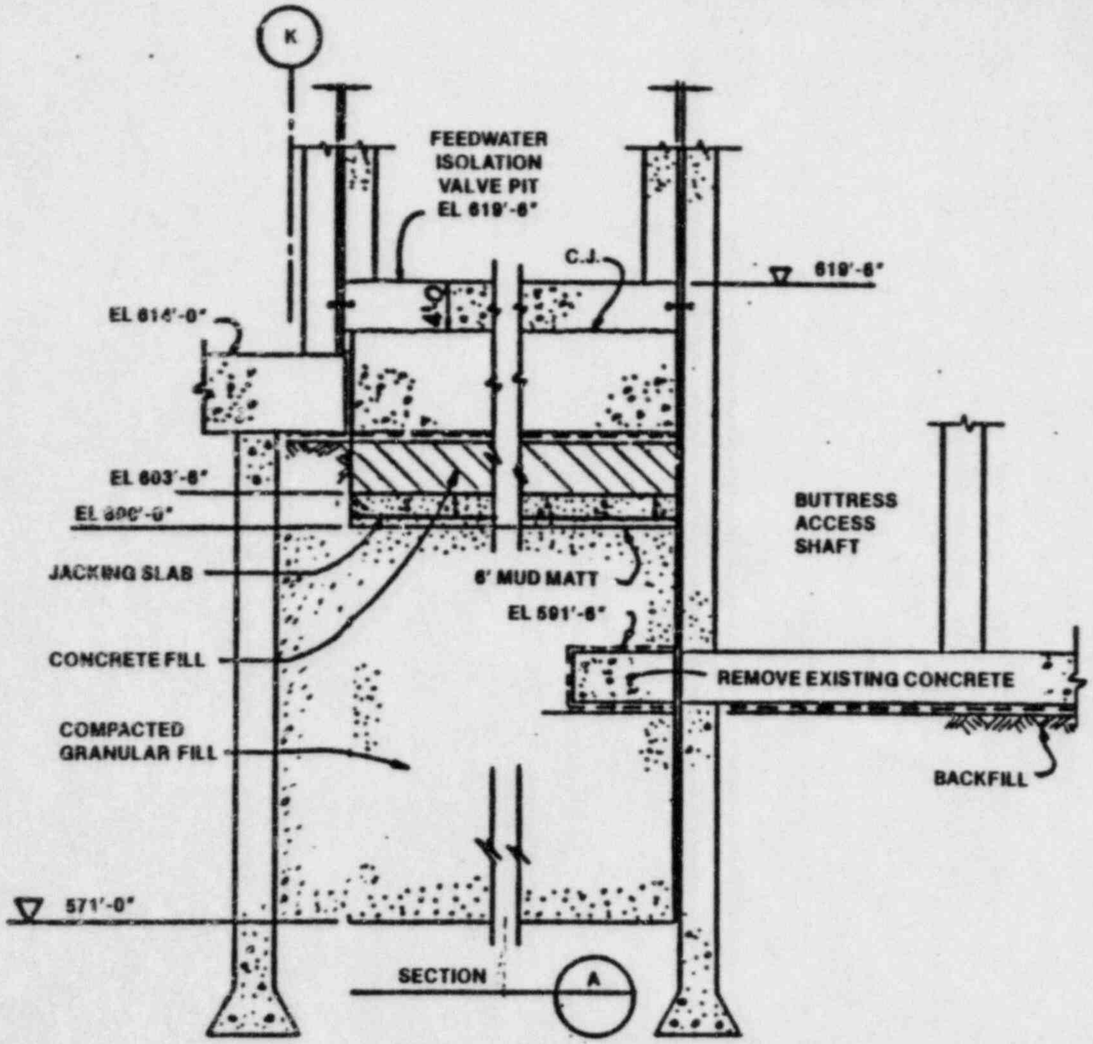


CONSUMERS POWER COMPANY
MIDLAND UNITS 1 AND 2

UNDERPINNING WALL SECTIONS

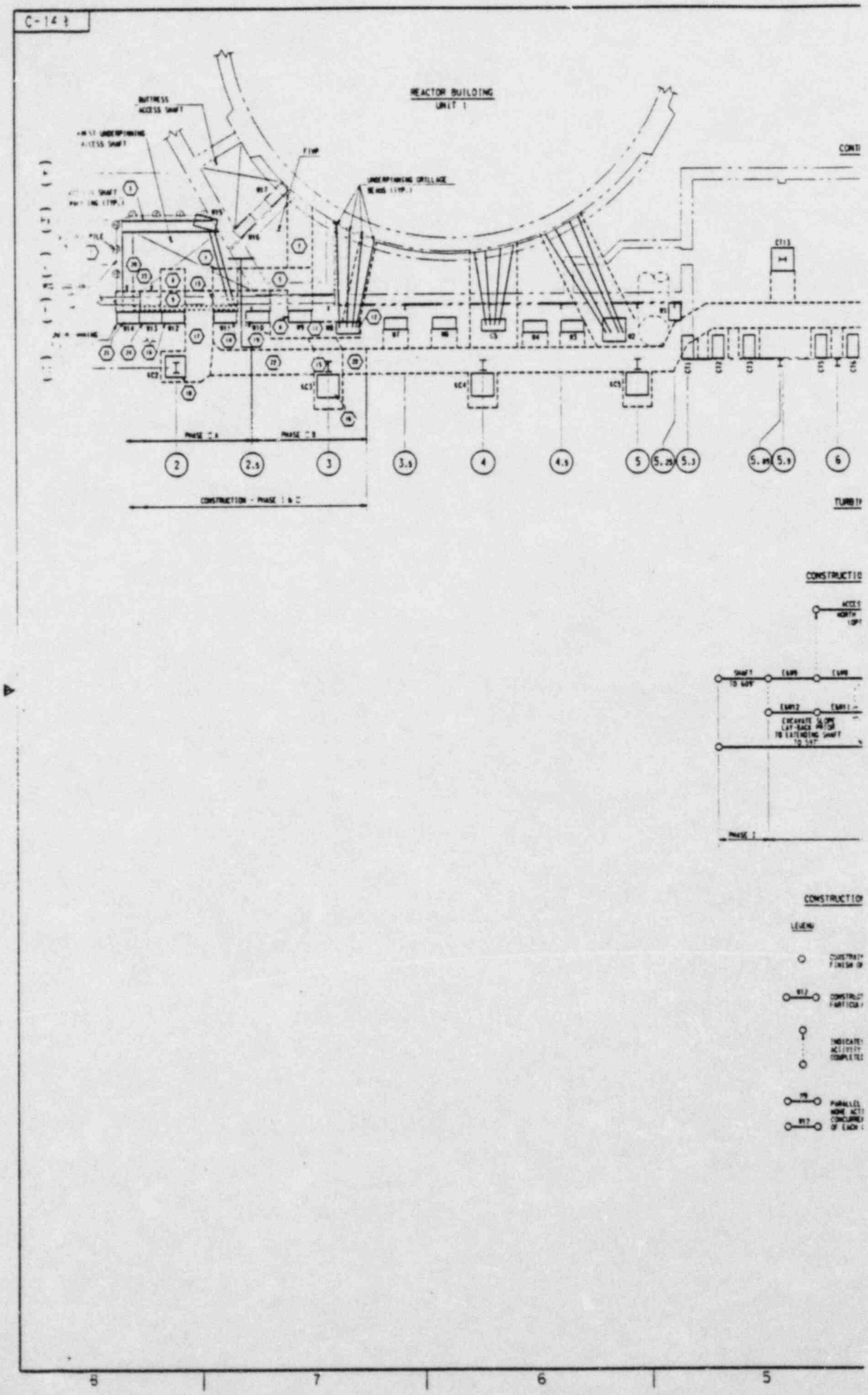
FIGURE II-6

G-1986-06

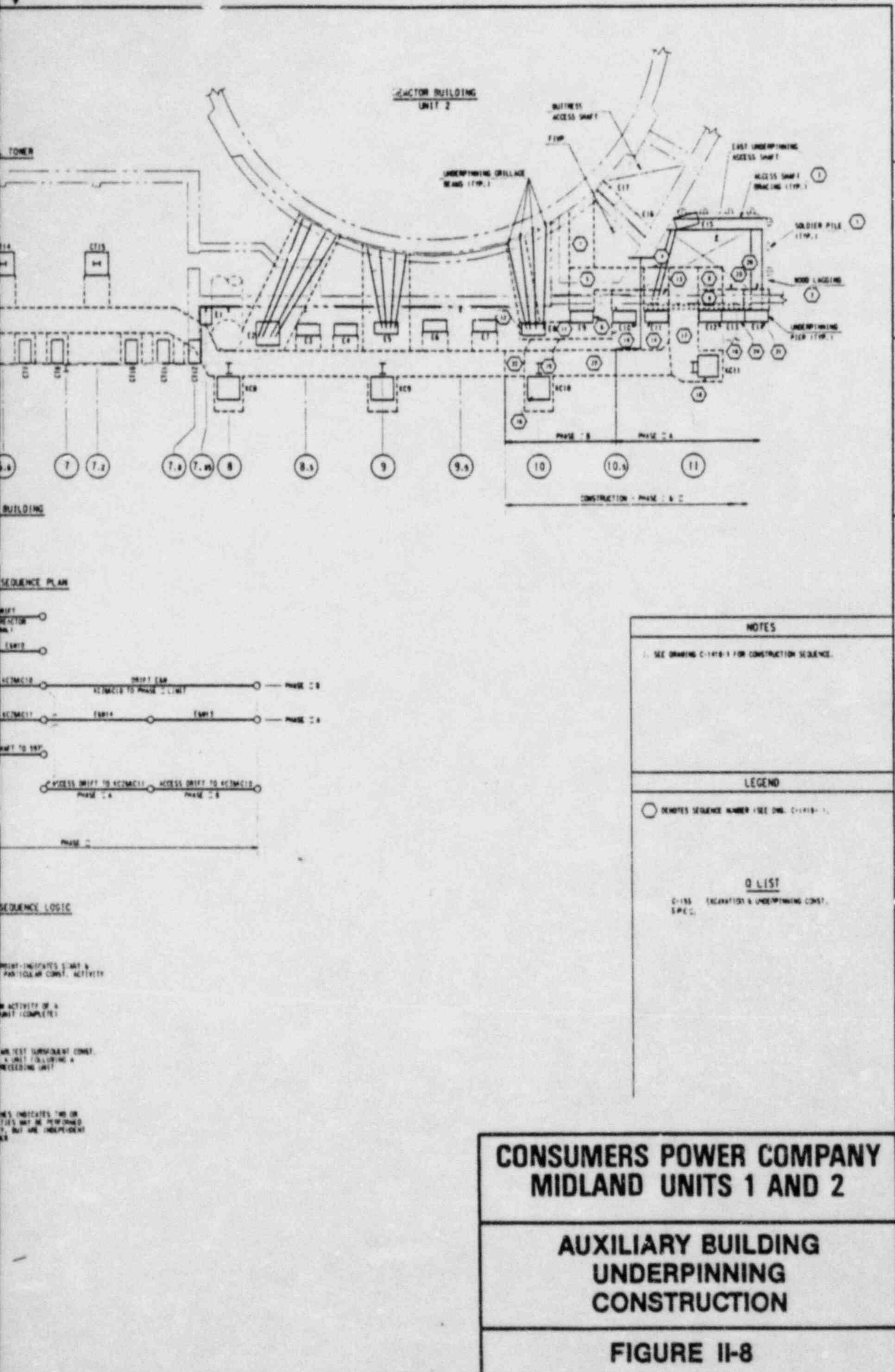


CONSUMERS POWER COMPANY MIDLAND UNITS 1 AND 2
SECTION AT FIVP
FIGURE II-7

G-1986-11


C-14 3
REACTOR BUILDING
UNIT 1
BUTTRESS
ACCESS SHAFT
UNDERPINNING GRILLAGE
BEAMS (TYP.)
PHASE II B
2
2.5
3
3.5
4
4.5
5
5.3
6
CONSTRUCTION - PHASE I & II
LEGEND
8
7
6
5

SIZE E

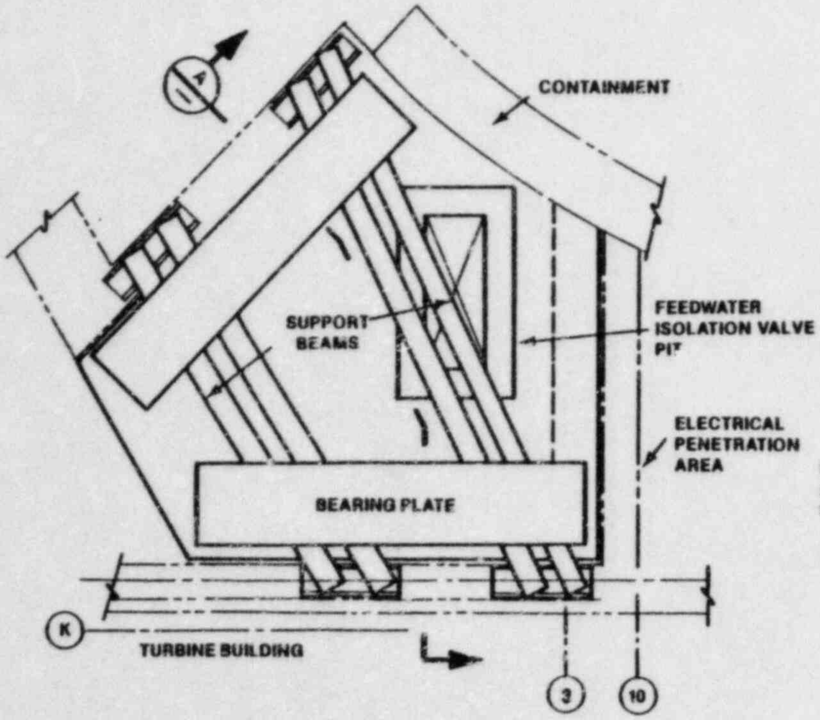


CONSUMERS POWER COMPANY
MIDLAND UNITS 1 AND 2

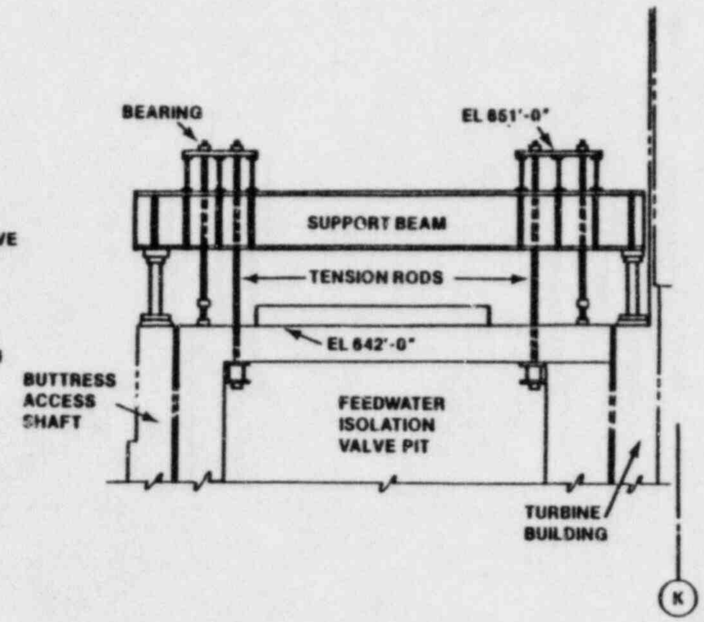
AUXILIARY BUILDING
UNDERPINNING
CONSTRUCTION

FIGURE II-8

G-1986-13



PLAN AT EL 651'-0"



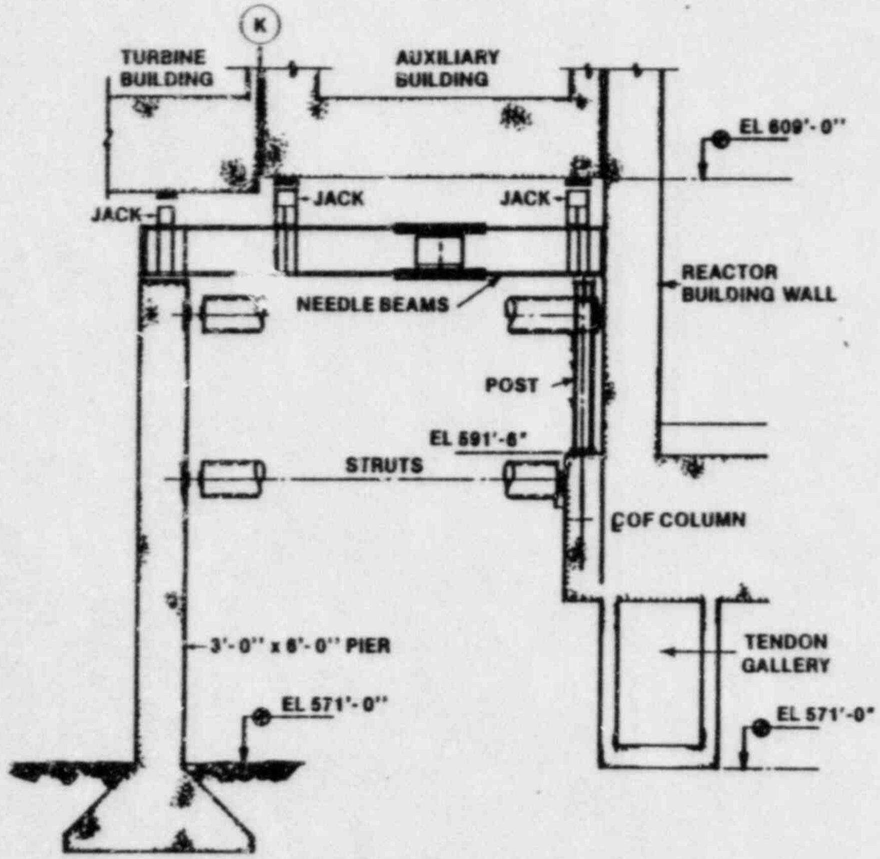
SECTION A

CONSUMERS POWER COMPANY
MIDLAND UNITS 1 AND 2

TEMPORARY SUPPORT FOR FEEDWATER ISOLATION VALVE PIT

FIGURE II-9

G-1986-02

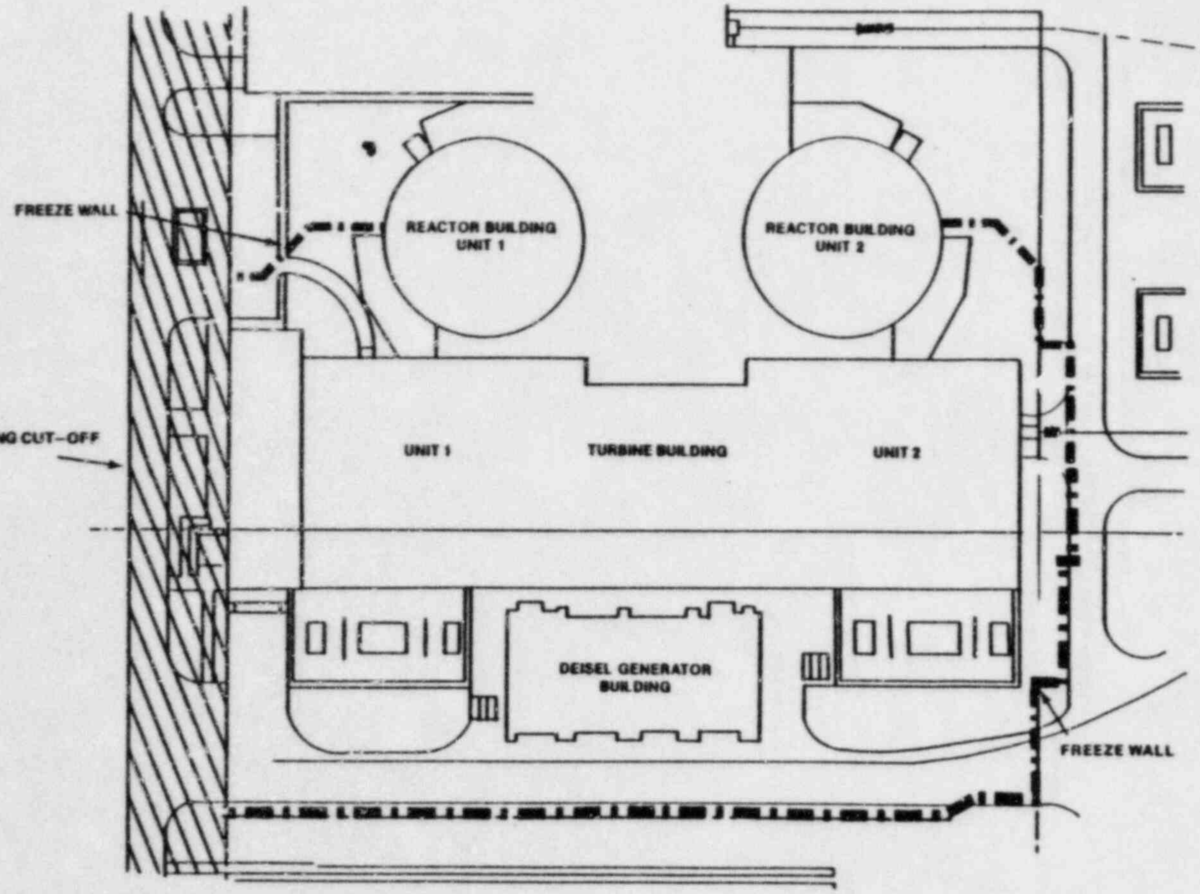


CONSUMERS POWER COMPANY
MIDLAND UNITS 1 AND 2

UNDERPINNING SECTION AT ELECTRICAL PENETRATION AREA

FIGURE II-10

G-1986-04

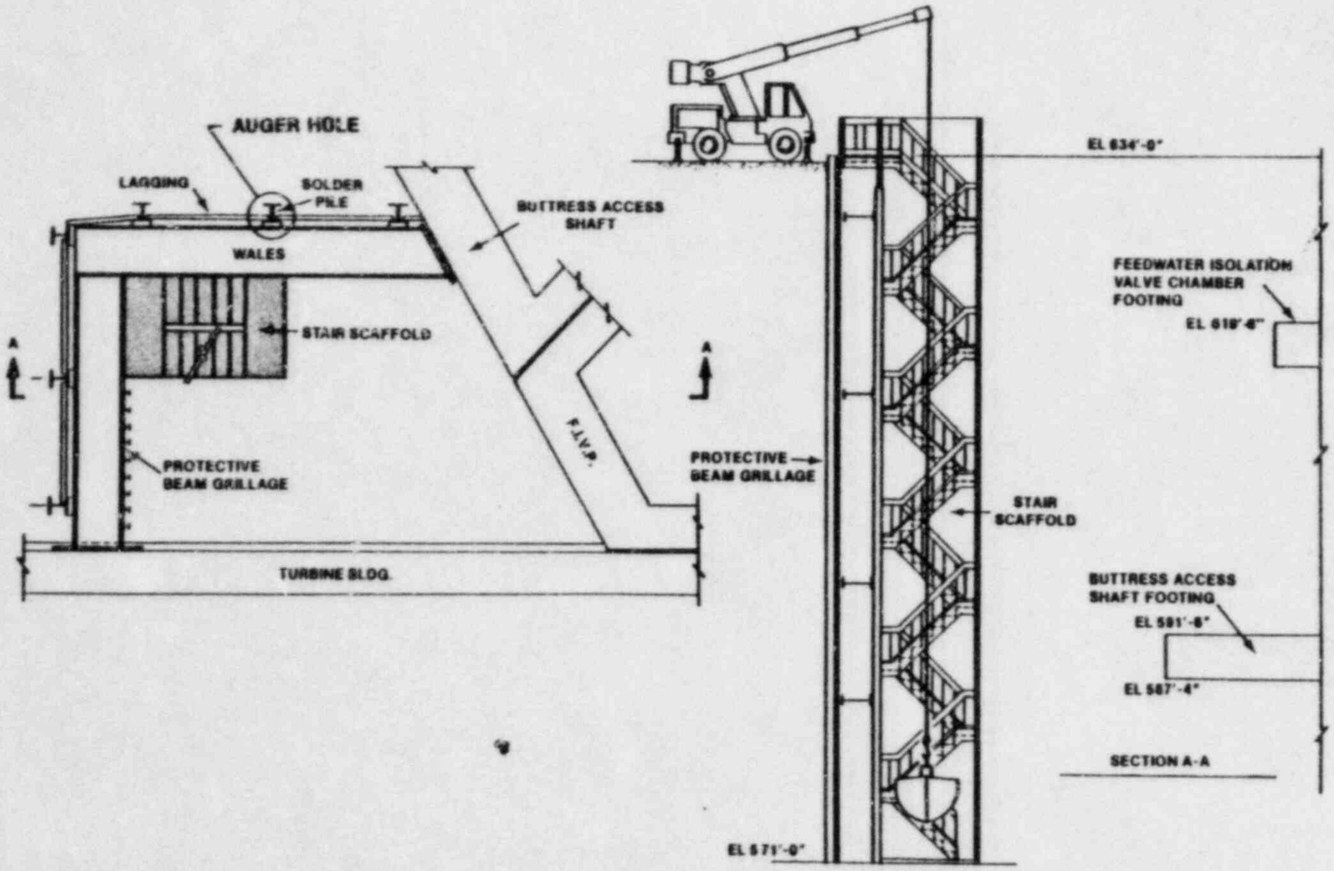


CONSUMERS POWER COMPANY
MIDLAND UNITS 1 AND 2

FREEZE CURTAIN DAM

FIGURE II-11

G-1986-03



CONSUMERS POWER COMPANY
MIDLAND UNITS 1 AND 2

ACCESS SHAFT

FIGURE II-12

G-1986-01

PART III: SERVICE WATER PUMP STRUCTURE

1.0 INTRODUCTION

The 1978 settlement of the diesel generator building (DGB) and subsequent plant soil investigation revealed inadequately compacted fill under a portion of the service water pump structure (SWPS).

The SWPS is a two-level, rectangular, reinforced concrete structure. Figure III-1 shows a general arrangement of this building. The foundation slab for the lower part of the building rests on undisturbed natural material. The foundation slab for the upper part of the building rests on plant fill.

The inadequately compacted fill resulted in the need for remedial action for the overhang portion of the structure (the portion founded on fill material). The remedial action is described below.

2.0 DESIGN CONCEPT

The remedial action agreed upon with the NRC staff consists of installing a permanent, continuous underpinning wall under the foundation of the overhang portion of the structure (see Figure III-1). The wall transfers the loads of this part of the structure from the fill to undisturbed natural material. The wall is connected to the existing structure.

2.1 IMPLEMENTATION OF PLAN

The structure, including the underpinning, has been analyzed for the loads from the building, the effects of the 40-year settlement of soil, and environmental effects such as earthquakes and tornados. The dimensions and major details of the underpinning have been finalized, based on a design which used the results of the analyses. The existing structure has been found to be adequate based on these structural analyses and design. The supporting undisturbed material has also been found to be adequate.

The underpinning wall is constructed in small sections (piers) which are tied together to form a continuous wall. The piers are constructed by hand digging pits and filling them with concrete. After a pier is completed, the load from the structure is transferred by jacking to a predetermined value known as initial jacking load.

To construct the underpinning piers, which are below the existing foundations, access is needed from the grade elevation. This access is provided from the outside of the building by open excavation for the piers on the north and the east walls. The access for the piers on the west wall is provided by an access shaft from the grade and a tunnel under the base slab for the overhang portion.

The underpinning is to be constructed in a dry condition. Because the present site dewatering is not adequate to lower the groundwater to the bottom of the underpinning, additional dewatering is accomplished by installing dewatering wells around the areas to be excavated.

The first piers to be constructed are three corner piers at the two corners of the underpinning walls. The completion of these piers is very important to the underpinning operation, because at this stage the entire weight of the overhang can be supported without depending on the fill. Therefore, the loss of fill support is not critical after this stage.

After the corner piers are completed, the remaining piers, except four sections on the east and west walls, are completed based on a predetermined sequence. At this stage the building is supported by initial jacking loads.

The jacking is now adjusted to the final design jacking loads. The settlements are monitored and after the rate of settlements has reached a predetermined value, the jacking load is locked off.

The underpinning is now connected to the structure by anchor bolts and dowels and by constructing the remaining sections on the east and west walls. Also, the gaps between the underpinning and the existing structure are filled with grout. All the excavations are backfilled with fill or concrete. At this stage, the underpinning wall rests on undisturbed material and the underpinning operation is complete.

During the underpinning operation, extreme care must be taken to protect the existing structure. This will be accomplished by removing only small portions of supporting soil and replacing these with piers of greater load-bearing capacity. In addition, the structure will be monitored frequently for strains to ensure that these remain below predetermined limits.

2.2 LICENSING STATUS

The design concept for the SWPS underpinning has been presented and discussed with the NRC staff using several methods: technical reports, meetings, and design audits by the staff.

A technical report describing the underpinning was submitted on August 26, 1981. This was supplemented by responses to NRC staff requests for additional information on November 16, 1981, and by an appendix, dated February 23, 1982, to the August report.

A meeting between the staff and the Applicant was held on September 17, 1981, to discuss both the concept and details of the design. Additional meetings were held on February 23, through 26, 1982, to discuss the finite-element model, construction aspects, and geotechnical issues.

A design audit was conducted in the Bechtel offices at Ann Arbor, Michigan, on March 16 through 19, 1982. During the audit, the staff reviewed the design calculations for the SWPS underpinning.

The design concept of the SWPS underpinning has concurrence from the NRC staff.

3.0 STRUCTURAL ANALYSIS AND DESIGN

The structural analysis of the SWPS and its underpinning is performed in two parts:

a. Seismic analysis using a mathematical model to analyze the structure for the dynamic conditions during a seismic event

b. A static analysis using different models to analyze the structure for the static loads, such as dead, live, and wind loads, etc imposed on the structure.

The results of these two analyses are combined in accordance with applicable load combinations. Load combinations presented in Final Safety Analysis Report (FSAR) Subsection 3.8.6 and supplemented by the Responses to NRC Requests Regarding Plant Fill, Question 15, (Revision 3, September 1979) are used for the structure and the underpinning and its connections to the structure. Additional loading combinations based on American Concrete Institute (ACI) Code 349-76 and supplemented by NRC Regulatory Guide 1.142 are used for the underpinning and its connections to the structure.

3.1 SEISMIC ANALYSIS

A seismic model is developed to evaluate overall building response to seismic forces as well as to generate in-structure response spectra for equipment design. The seismic forces are determined using a lumped-mass model with the response spectrum modal superposition technique. The computed seismic response accelerations are multiplied by the structural element masses to provide the seismic forces for the seismic structural analysis.

The underpinning is designed by staff direction to withstand the effects of the site-specific response spectra (SRSS) ground motion, while the existing structure is evaluated and found acceptable for the effects of the FSAR ground motion description. In order to proceed with the underpinning design while NRC concurrence with the proposed SRSS was being obtained, the structural forces resulting from the FSAR SSE ground motion were multiplied by a factor of 1.5 for design of the underpinning. The response from 1.5 times the FSAR SSE envelops the final SSRS response.

The seismic analysis of the underpinned structure has been completed and the results have been used for the static analysis of the underpinning.

3.2 STATIC ANALYSIS

The static structural analysis uses a finite-element analytical model capable of representing the structure behavior. The interface between the existing structure and the underpinning wall is modeled to transfer loads. The soil media are represented by springs of appropriate stiffness at the base of the structure.

The analysis uses different analytical systems requiring two different models and appropriate springs. The two analytical models that have been developed are used in the following manner.

3.2.1 Disconnected Model

A disconnected model, in which the underpinning wall is not connected to the structure, is used to investigate various construction stages. This model is also utilized in combination with the connected model to determine preload effects on the existing structure due to jacking.

3.2.2 Connected Model

A model in which the underpinning wall is connected to the structure is used to investigate the effects of long-term loading such as differential settlement and short-term loading such as seismic forces. The differential settlement is considered in the model by calculating appropriate spring constants based on settlements of the underpinning and the existing structure.

Based on the properties of the natural materials, it is estimated that the settlement of the underpinned structure after construction is completed will range from 0.1 inch to 0.2 inch for the 40-year life of the structure. The settlement of the main SWPS will range from 0.2 inch to 0.3 inch for the 40-year life of the structure. These predicted settlements are based on an investigation conducted by Woodward-Clyde Consultants (WCC), who performed soil borings and laboratory testing of the undisturbed natural materials. These tests show the preconsolidation pressure of the natural materials to be 48 tons/sq ft.

3.3 DESIGN OF UNDERPINNING

The results of these structural analyses are then factored and added in specific combinations. The results are used to evaluate the structural adequacy of the structure and the underpinning. The computed stresses or loads are ensured to be lower than the allowable stresses or capacities.

The underpinning walls and their connections are designed to meet the requirements set forth in FSAR Subsection 3.8.6 as supplemented by the Responses to NRC Requests Regarding Plant Fill, Question 15, and ACI 349-76 as supplemented by NRC Regulatory Guide 1.142. The capacity of the existing structure is reviewed in accordance with FSAR Subsection 3.8.6 requirements and Question 15 of the Responses to NRC Requests Regarding Plant Fill.

3.3.1 Underpinning

The design features of the underpinning are described below.

The proposed underpinning, as shown in Figure III-1, is a 4-foot thick, reinforced concrete wall that is 30 feet high and is constructed to act as a continuous member under the perimeter of the structure overhang. The entire wall is founded on undisturbed natural material. The base of the north underpinning wall is belled out to a 6-foot thickness to limit bearing pressures to the allowaable values, whereas the bases of the east and west side walls are 4 feet wide.

The allowable bearing pressures for the undisturbed natural material are based on a safety factor of 2 for dynamic loading and 3 for static loading. The ultimate bearing capacity for the natural material is based on the undrained triaxial tests performed on the WCC boring samples. These yielded a median shear strength of 18 ksf.

A jacking force is applied to the overhang perimeter to provide adequate load transfer from the structure to the underpinning. These jacking forces transmit the structural loads through the permanent underpinning wall to the bearing stratum.

Dowels and anchor bolts connect the underpinning walls and the existing structure at the vertical and horizontal interfaces.

The dowels and anchor bolts are designed to transfer shear and tension forces between the structure and the underpinning wall.

3.3.2 Temporary Post-Tensioning

A temporary post-tensioning system is designed to apply a compressive force to the upper part of the building along the north-south exterior walls. This post-tensioning is required to compensate for the loss of buoyancy, which results in additional forces on the overhang, when the construction site dewatering is installed.

The post-tensioning and the access shaft design are based on the ACI 318 and American Institute of Steel Construction (AISC) codes.

3.4 LICENSING STATUS

Structural analysis and design for the underpinning was presented in the technical reports, meetings, and a design audit by the staff, which have been previously identified in Section 2.2.

The seismic analysis was covered in detail during testimony by Dr. R.P. Kennedy of Structural Mechanics Associates and Dr. P. Halada of the U.S. Army Corp of Engineers (representing the NRC staff) during the Atomic Safety and Licensing Board soils hearings of December 14, 1981.

As indicated in Section 2.2, a design audit has been performed by the NRC staff. During this audit, structural analysis and design calculations for the underpinning, access shaft, and post-tensioning were reviewed. The NRC audit resulted in a list of confirmatory issues which the Applicant is addressing and will be prepared to discuss with the NRC staff in the near future.

4.0 CONSTRUCTION SUPPORT PROGRAM

4.1 GROUNDWATER CONTROL

At the start of the underpinning work, it is anticipated that the groundwater level will be about el 600'. Because this underpinning will extend at least 15 feet below this level, the control of groundwater is an important prerequisite for successful completion.

The groundwater level will be lowered below el 585' by using temporary dewatering wells. As part of the temporary dewatering procedure, piezometers will be installed to monitor the groundwater level. These wells will be sealed after the underpinning wall is completed.

The design of the temporary dewatering well system is complete and is shown in Figure III-2.

4.2 CONSTRUCTION ACCESS

The Applicant has recently decided to employ an improved method of access for installation of the hand-dug pits. The method utilizes external access from the outside in lieu of the tunnels shown in Figure III-3. The advantage of this proposal is that it can be better coordinated with the proposed replacing/rebedding of the service water piping north of the SWPS. An access shaft and tunnel will still be installed along the west wall of the overhang because the circulating water intake structure (CWIS) is adjacent to the SWPS on the west side. The underpinning installation sequence will not be altered by adoption of the improved access method.

4.3 LICENSING STATUS

The groundwater control and the improved access method have been discussed with the NRC staff during its recent audit mentioned in Section 2.2. Permission has been received for the installation and activation of the dewatering system.

5.0 CONSTRUCTION PROGRAM

5.1 BUILDING POST-TENSIONING

A temporary post-tensioning system has been installed at the upper part of the building along the north-south exterior walls. The post-tensioning system will be removed after the initial jacking loads are applied.

5.2 UNDERPINNING

This section describes the construction sequence of the underpinning wall. The layout and the sequence are shown in Figure III-3. The underpinning wall is constructed in small sections (piers) to preserve the structural integrity of the building. The first piers to be constructed are approximately 30-foot deep, 5-foot by 4-foot hand-dug sheeted pits located at each corner of the overhang. After the subgrade for these pits is inspected and approved by a geotechnical engineer, reinforcement, subgrade settlement and stress monitoring instrumentation, and anchor bolt assemblies to tie the pier to the underside of the slab are installed. The piers are then encased with concrete. An initial jacking load is applied to the overhang from jacks placed on the pier tops. After jacking, the remaining piers are constructed in the sequence outlined in Figure III-3.

Stress monitoring instrumentation will be installed in designated piers. The piers are tied together with threaded reinforcing bar couplers and shear keys to form a continuous underpinning wall.

The final jacking loads are applied after No. 10 piers (see Figure III-3) are constructed and the underpinning wall has progressed to within 6 feet of the vertical interface with the existing structure. Settlements caused by this load are monitored. When the geotechnical engineer determines that the settlement has decreased to a predetermined rate, the load is transferred from the jacks to wedges positioned between the top of the piers and the underside of the overhang, and the jacks are removed. No. 11 piers are poured, encasing dowel bars that were previously drilled and grouted into the vertical face of the existing structure and thereby connecting the underpinning wall to the existing structure. The space between the top of the underpinning wall and the underside of the base slab is filled with nonshrink grout and previously placed anchor bolt assemblies are tightened. The underpinning wall is connected to the structure at both the vertical and horizontal interfaces. Piers 12 are then constructed, completing the underpinning wall.

5.3 LICENSING STATUS

The construction details and sequence have been discussed with the NRC staff in meetings and during an NRC audit mentioned in Section 2.2. The audit resulted in a list of confirmatory issues which the Applicant is addressing and will be prepared to discuss with the NRC staff in the near future.

6.0 MONITORING PROGRAM

To ensure that installation of the underpinning is proceeding within acceptable limits, a monitoring program will be implemented during construction. This program has four parts:

a. Building settlement

b. Building strain

c. Cracking

d. Underpinning

6.1 BUILDING SETTLEMENT

In addition to the pier settlement monitoring program, a program to closely monitor the overall structure settlement has been planned. Besides the four existing settlement markers at each corner of the building, five additional markers have been installed on the building. A settlement dial indicator has been installed at each of the two north building corners where the underpinning will be constructed. The dial indicators measure displacement between the building and permanent benchmarks founded in undisturbed soil approximately 50 feet below the bottom of the underpinning wall. The depth at which the tip of the benchmark is located ensures that the benchmark movement will be negligible. The settlement markers will be monitored before and after major construction events.

Based upon a request from the NRC during the audit on March 16, through 19, 1982, one additional deep-seated benchmark is being placed on the south side of the structure to monitor settlements.

6.2 STRAIN MONITORING

Before the actual construction of the underpinning wall begins, strain indicating devices with gage lengths of approximately 20 feet will be installed near the top and bottom of the exterior north-south walls at the location of their connection to the existing structure. The strain will be monitored to ensure that it is lower than predetermined levels.

6.3 CRACKS

6.3.1 Existing Crack Evaluation

The existing cracks in the SWPS have been monitored. The size and location of existing cracks have been recorded on crack map drawings. The Applicant's consultant, Portland Cement Association (PCA), evaluated the structural significance of these cracks based on its site visit and review of the crack maps. The consultant concluded that cracks observed in this structure are attributable to restrained volume changes that occur during curing and drying of concrete. PCA also did not observe any structural distress during its visit. Furthermore, PCA concluded that while occurence of stress-related cracking because of differential building settlement cannot be completely dismissed, it did not appear that such hypothesized settlements were a primary cause of cracks observed in this structure. PCA's evaluations and conclusions are contained in a report submitted to the NRC staff on March 3, 1982.

6.3.2 Crack Monitoring During Underpinning

Existing cracks will be monitored for changes in length and width during various phases of construction. The areas containing cracks will be inspected for new cracks that, if present, will be similarly mapped and monitored.

Because of the sequence of construction procedures, it is not anticipated that existing cracks will significantly widen or that significant new cracks will appear. However, any new structural cracks exceeding 0.01 inch in width or any crack exceeding 0.03 inch in width will be evaluated by PCA to determine whether underpinning operations should stop or continue. If development of yield strain is inferred from any observed crack, underpinning will be stopped and an evaluation made by PCA before continuing underpinning operations.

6.3.3 Repair of Cracks

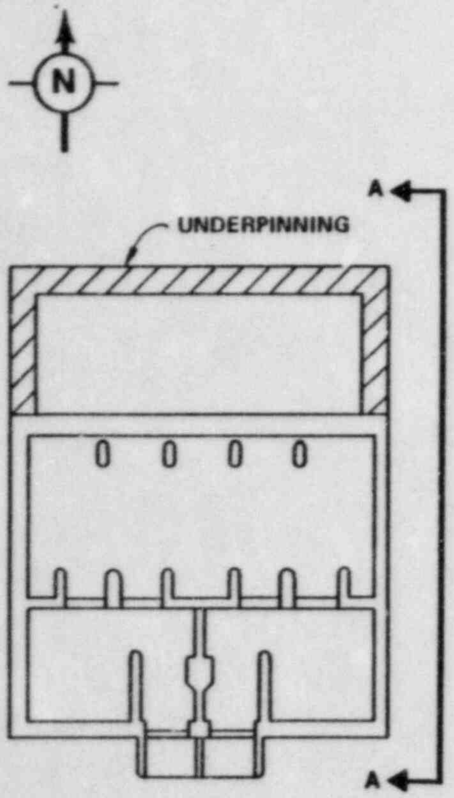
A report on a crack repair program by PCA for all cracks in all structures will be submitted to the NRC staff in the near future.

6.4 UNDERPINNING

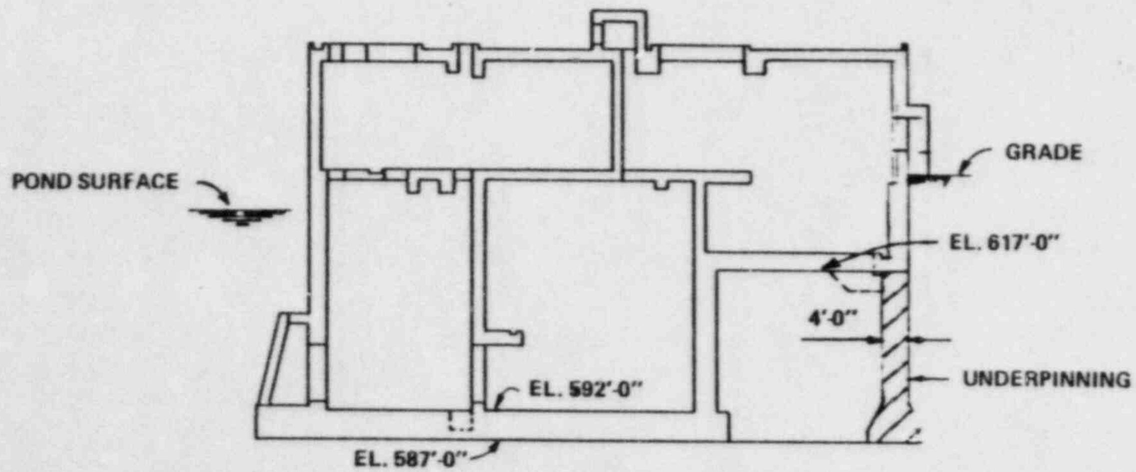
A settlement monitoring program for the top and base of each pier begins immediately after pier construction. Instruments accurate to 0.001 inch are installed before the initial jacking is applied. The information from this monitoring program is used to evaluate the time required to dissipate shrinkage and creep of the concrete and the time when settlement of the undisturbed natural material below the underpinning wall has reached a predetermined rate.

Stress meters will be cast in concrete near the top and bottom of designated piers. These instruments will monitor variations in applied loads.

3



PLAN AT EL 592'-0"

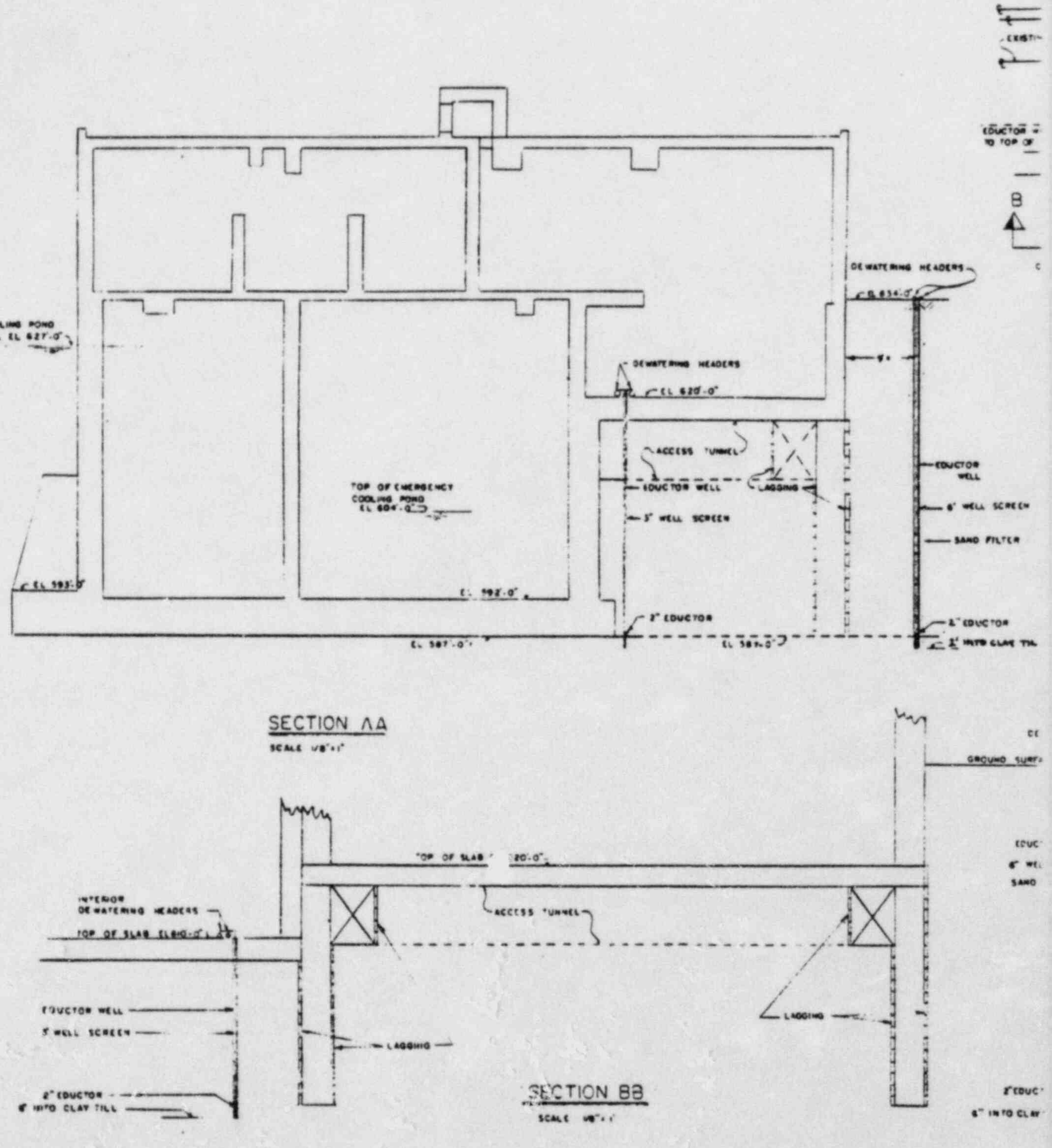


ELEVATION A-A

CONSUMERS POWER COMPANY
MIDLAND UNITS 1 AND 2

UNDERPINNING GENERAL LAYOUT

FIGURE III-1

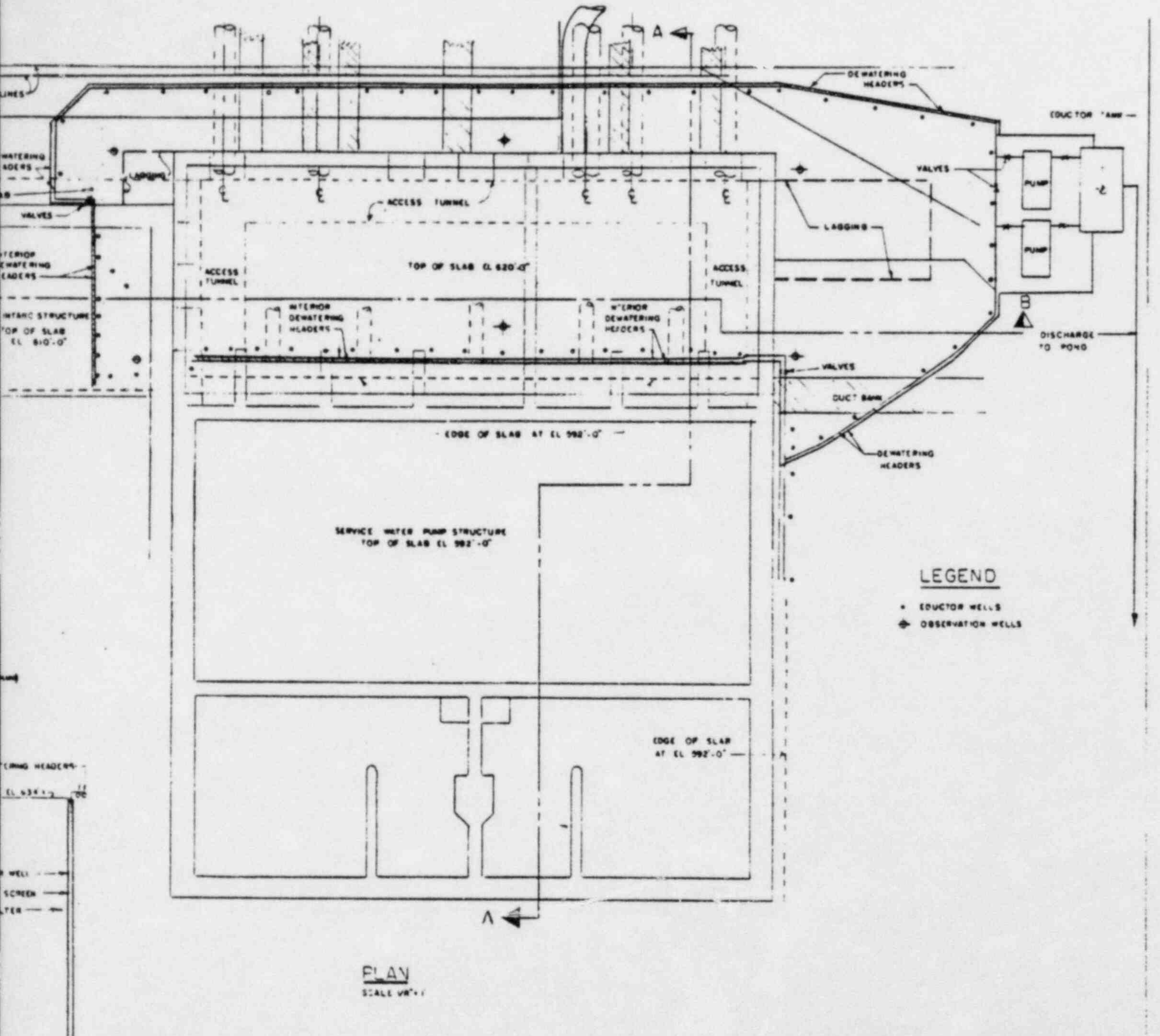


SECTION AA

SCALE 1/8"=1'

SECTION BB

SCALE 1/8"=1'

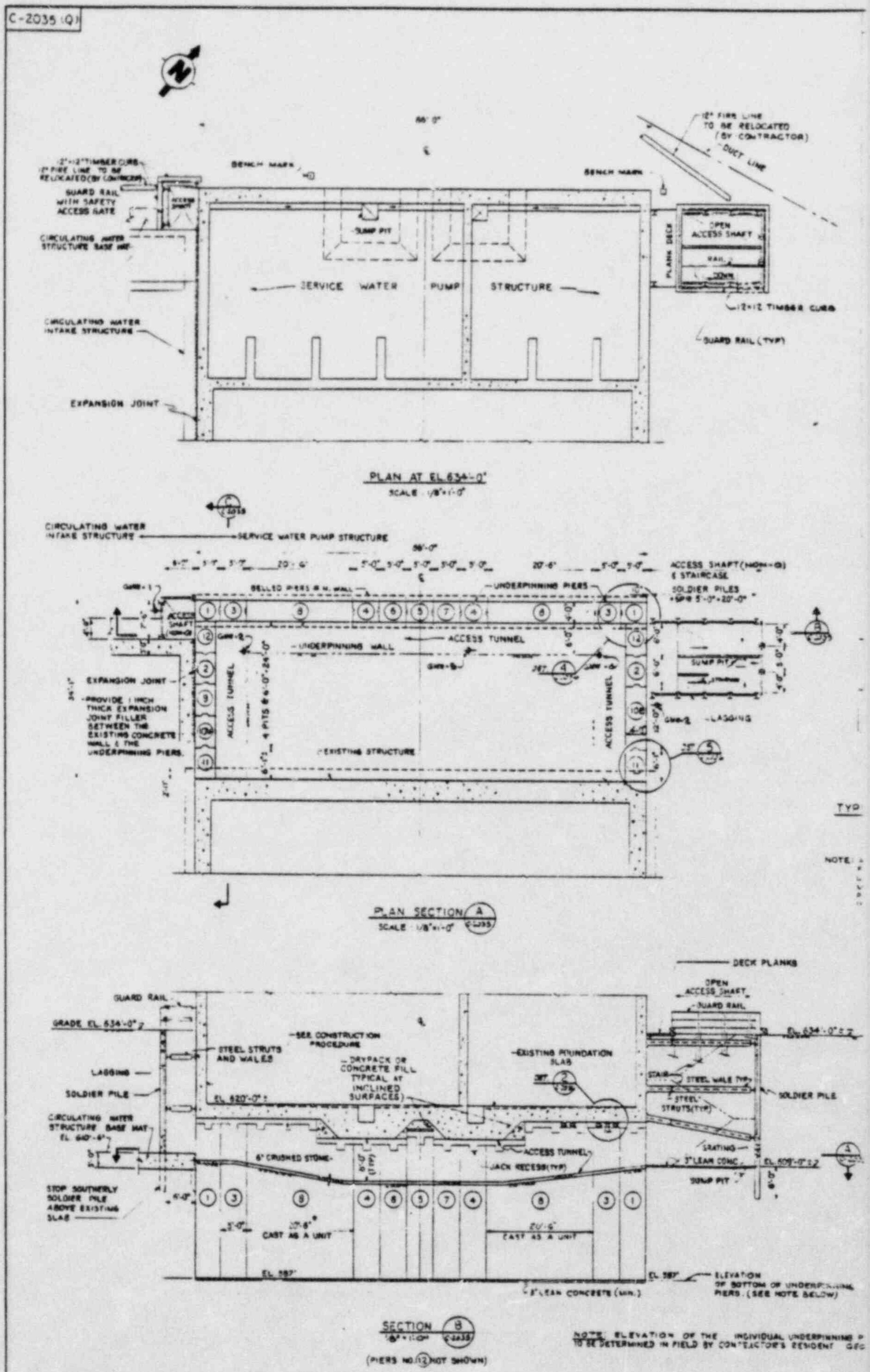


CONSUMERS POWER COMPANY
MIDLAND UNITS 1 AND 2

DEWATERING PROCEDURE
SERVICE WATER PUMP STRUCTURE

FIGURE III-2

G-1986-33


C-2035 (Q)
12" FIRE LINE TO BE RELOCATED (BY CONTRACTOR)
DUCT LINE
BENCH MARK
GUARD RAIL WITH SAFETY ACCESS GATE
CIRCULATING WATER STRUCTURE BASE MAT
SUMP PIT
SERVICE WATER PUMP STRUCTURE
OPEN ACCESS SHAFT
PLANK DECK
12x12 TIMBER CURB
GUARD RAIL (TYP)
CIRCULATING WATER INTAKE STRUCTURE
EXPANSION JOINT
PLAN AT EL. 634'-0"
SCALE 1/8"=1'-0"
CIRCULATING WATER INTAKE STRUCTURE
SERVICE WATER PUMP STRUCTURE
ACCESS SHAFT (NON-Q) & STAIRCASE
UNDERPINNING PIERS
SOLDIER PILES
UNDERPINNING WALL
ACCESS TUNNEL
PROVIDE 1 INCH THICK EXPANSION JOINT FILLER BETWEEN THE EXISTING CONCRETE WALL & THE UNDERPINNING PIERS.
EXISTING STRUCTURE
LAGGING
TYP
NOTE:
PLAN SECTION A
SCALE 1/8"=1'-0"
DECK PLANKS
OPEN ACCESS SHAFT
GUARD RAIL
GRADE EL. 634'-0"
SEE CONSTRUCTION PROCEDURE
STEEL STRUTS AND WALES
LAGGING
SOLDIER PILE
DRYPACK OR CONCRETE FILL TYPICAL AT INCLINED SURFACES)
EXISTING FOUNDATION SLAB
EL. 634'-0"
STEEL WALE TYP
STEEL STRUTS (TYP)
SOLDIER PILE
EL. 620'-0"
CIRCULATING WATER STRUCTURE BASE MAT EL. 610'-6"
6" CRUSHED STONE
ACCESS TUNNEL
JACK RECESS (TYP)
GRATING
3" LEAN CONC.
SUMP PIT
STOP SOUTHERLY SOLDIER PILE ABOVE EXISTING SLAB
CAST AS A UNIT
EL. 587'
ELEVATION OF BOTTOM OF UNDERPINNING PIERS. (SEE NOTE BELOW)
3" LEAN CONCRETE (MIN.)
SECTION B
(PIERS NO. 12 NOT SHOWN)
NOTE: ELEVATION OF THE INDIVIDUAL UNDERPINNING P
TO BE DETERMINED IN FIELD BY CONTRACTOR'S RESIDENT GEO

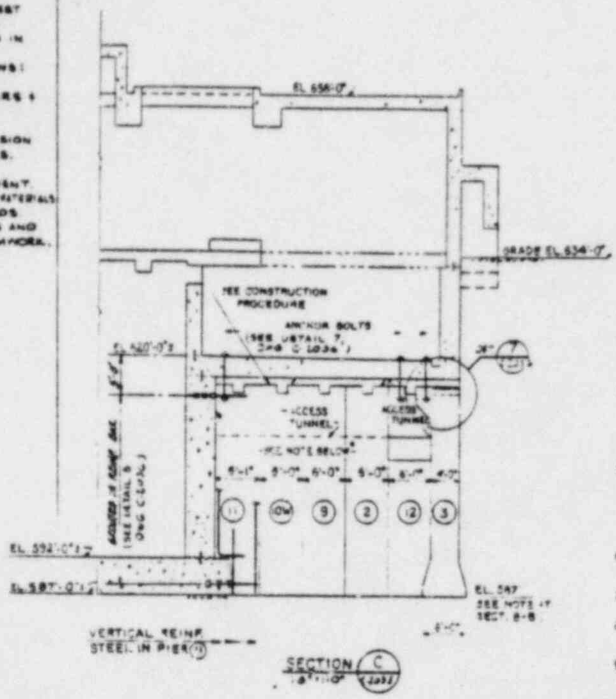
NOTES

THE NON-Q MATERIALS FURNISHED BY THE SUBCONTRACTOR SHALL INCLUDE BUT NOT NECESSARILY BE LIMITED TO THE FOLLOWING:

a) ALL MATERIAL REQUIRED TO CONSTRUCT SHEETED & BRACED TEMPORARY ACCESS SHAFTS AND TUNNEL VENTILATION AND LIGHTING.

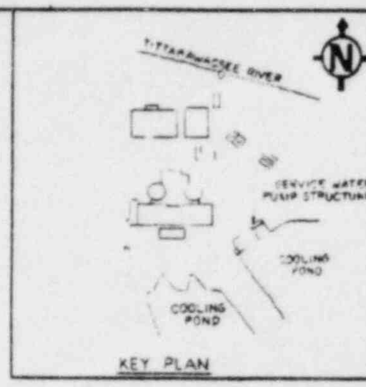
b) ALL MATERIAL REQUIRED TO MEET THE TEMPORARY DEWATERING REQUIREMENTS AS SET FORTH IN SPEC. 7220-C-194(Q)

c) MISC. MATERIALS AS FOLLOWS:
1.) ETHAFOAM OR EQUAL
2.) CHAIRS, BOLSTERS, SPACERS & TIE WIRE REQUIRED FOR REBAR PLACEMENT.
3.) STEEL ANGLES AND EXPANSION BOLTS FOR FORMING PIERS.
4.) CRUSHED STONE.
5.) JACKS & JACKING EQUIPMENT.
6.) GROUND WATER MONITORING WELL MATERIALS
7.) PRECAST CONCRETE BOARDS
8.) PAINT FOR ANCHOR BOLTS AND BOND BREAKER FOR FORMWORK.



SECTION C

NOTE: SHEAR KEYS BETWEEN PIERS NOT SHOWN.
* PREPARE SURFACES IN CONFORMANCE WITH SPEC 166 USING A LIGHT CHIPPING GUN



KEY PLAN

CONSTRUCTION PROCEDURE

1. ESTABLISH SETTLEMENT READING [illegible] ON THE SERVICE WATER STRUCTURE AND MAKE PERIODIC READINGS AS SPECIFIED. (BY CONTRACTOR)
2. FURNISH, INSTALL AND OPERATE DEWATERING SYSTEM TO PERMIT UNDERPINNING WORK IN A DRAINED CONDITION IN ACCORDANCE WITH SPECIFICATIONS. [illegible]
3. CONSTRUCT SHEETED AND BRACED ACCESS SHAFTS AS [illegible] ON THE DRAWINGS. IN THE EAST SHAFT [illegible] FRESH AIR PIPE AND CONCRETE PUMPING PIPE [illegible]
4. [illegible] FROM THE TWO ACCESS SHAFTS, [illegible] SUFFICIENT LENGTH OF ACCESS TUNNEL TO PERMIT PIER WORK TO PROGRESS. EXCAVATE AND SHEET PIERS [illegible] STRUCTURE HAS BEEN INSPECTED AND APPROVED PLACE REINFORCEMENT, INSTRUMENTATION AND LOAD [illegible] INTO HOLES DRILLED THROUGH THE SERVICE WATER STRUCTURE SLAB ABOVE; PUMP CONCRETE INTO PIERS.
5. WHEN PIERS NO. 1 CONCRETE HAS CURED FOR 24 HOURS MINIMUM, PLACE HYDRAULIC JACKS INTO JACKING SLOT, SET BEARING PLATES AND WEDGES. DRYPACK BEARING PLATES AT JACKS. AFTER DRYPACK HAS CURED 24 HOURS MINIMUM, JACK 145 KIPS LOAD INTO THE PIER [illegible] OBSERVE AND RECORD PIER DEFLECTIONS [illegible] THROUGHOUT THE [illegible] RE-JACK AS NEEDED [illegible] LOAD; ADJUST WEDGES AS NEEDED.
6. SIMULTANEOUSLY WITH STEP 5 EXTEND ACCESS TUNNEL TO PIER NO. 2 AND COMPLETE IN A SIMILAR [illegible] TO STEPS 3, 4 AND 5 ABOVE. JACKING TO 175 KIPS LOAD.
7. COMPLETE REMAINDER OF PIERS NOS. 3 THROUGH 7 WITH PROCEDURE SIMILAR TO STEPS 3, 4, 5 & 17.
8. AT PIER NO. 8 EXCAVATE AND BRACE PIT; PLACE REINFORCEMENT, INSTRUMENTATION, LOOSE [illegible] AND PUMP CONCRETE. COMPLETE AS PER NOTE 7 & LOAD [illegible]
9. EXTEND ACCESS TUNNEL TO PIERS 9 AND 10E. EXCAVATE & BRACE PITS AND COMPLETE THESE PIERS FOLLOWING PROCEDURE OUTLINED IN STEP 5. SEE TABLE FOR THE STAGE 1 JACKING LOADS.
10. PROCEED TO AND COMPLETE PIER 10W, JACKING TO 70 KIPS
11. JACK FINAL STAGE LOADS INTO PIERS 1 THROUGH 13 SIMULTANEOUSLY IN ACCORDANCE WITH THE TABLE OF JACKING LOADS AFTER ALL COMPLETED PIER CONCRETE HAS CURED FOR 30 DAYS OR AS DIRECTED BY CONTRACTOR. [illegible] TAKE DAILY OBSERVATIONS AND RECORD ALL SETTLEMENT READINGS AT ALL INSTRUMENTED PIERS FOR A PERIOD [illegible] UNTIL THE RATE OF SETTLEMENTS FALLS WITHIN THE [illegible] LIMITS. ADJUST AND REMOVE JACKS. [illegible] TOPS OF PIERS AND UNDERSIDE OF EXISTING STRUCTURE WITH CONCRETE.
12. EXCAVATE & BRACE PIT [illegible] & INSTALL [illegible] PREDRILLED HOLES IN THE EXISTING WALL. GROUT [illegible] PIERS, JACK, WEDGE & DRYPACK
13. FILL TUNNEL WITH [illegible] CONCRETE.
14. COMPLETE PIER NO. 13 FROM THE OUTSIDE, FOLLOWING PROCEDURE SIMILAR TO STEPS 3, 4 AND 5.
15. PRELOAD, [illegible] ANCHOR BOLTS [illegible]
16. REMOVE [illegible] CLEANUP SITE.
17. EXTEND ACCESS TUNNEL FROM EITHER SHAFT TO INSTALL PIERS 4 & 5. WHEN THESE TWO PIERS ARE LOAD BEARING, AS DESCRIBED IN STEPS 3, 4 & 5, EXTEND THE OTHER ACCESS TUNNEL AND COMPLETE PIERS 4, 6 & 7.
18. FOR GENERAL NOTES SEE DWG. [illegible]-2034

-0" MAX UNSUPPORTED LENGTH

3-#3 SPACER BARS

2-#4 BARS FOR PLANKS UP TO 5' LONG
3-#3 BARS FOR PLANKS LONGER THAN 5'

PRECAST CONCRETE BOARD DETAIL

N.T.S.

PRECAST CONCRETE PLANKS SHALL BE [illegible]CATED WITH 4000 PSI CONCRETE (NON-Q) [illegible]TWEIGHT CONCRETE OF EQUAL STRENGTH [illegible] BE SUBSTITUTED. [illegible] LONGER THAN 6'-0" SHALL BE [illegible]MITTENTLY BRACED AT 6'-0" MAX O.C.

JACKING LOAD TABULATION

SEQUENCE NUMBER	LENGTH (L)	JACK LOAD (STAGE 1)	JACK LOAD (FINAL STAGE)	NUMBER OF JACK SETS PER PIER
1	5'-0"	145 KIPS	204 KIPS	1
2	6'-0"	175 KIPS	150 KIPS	1
3	5'-0"	145 KIPS*	204 KIPS	1
4	5'-0"	145 KIPS	204 KIPS	1
5	5'-0"	145 KIPS*	204 KIPS	1
6	5'-0"	145 KIPS	204 KIPS	1
7	5'-0"	145 KIPS	204 KIPS	1
8	20'-6"	596 KIPS*	834 KIPS	4
9	6'-0"	70 KIPS	150 KIPS	1
10E	12'-0"	140 KIPS	300 KIPS	2
10W	6'-0"	70 KIPS	150 KIPS	1
11	8'-1"	-	WEDGE ONLY	
13	6'-0"	-	80 KIPS	1

* SEE CONSTRUCTION PROCEDURE
* APPROVAL OF THE CONTRACTOR IS REQUIRED BEFORE STAGE 1 JACK LOAD IS APPLIED TO PIERS 3, 5 AND 8.

LEGEND

(5) UNDERPINNING PIERS CONSTRUCTION SEQUENCE NUMBER

NEW UNDERPINNING PIERS

EXISTING STRUCTURE

EXCAVATION
[illegible]ICAL ENGINEER

CONSUMERS POWER COMPANY
MIDLAND UNITS 1 AND 2

SERVICE WATER PUMP STRUCTURE UNDERPINNING PLAN AND SECTIONS

FIGURE III-3

G-1986-35

PART IV: BORATED WATER STORAGE TANKS

1.0 INTRODUCTION

Each unit of the Midland plant has a 500,000 gallon stainless steel borated water storage tank (BWST) located in the tank farm north of the auxiliary building. The tanks are 32 feet high and 52 feet in diameter and sit on a concrete foundation (see Figure IV-1).

A soils investigation program of the fill in the tank farm area, consisting of 40 borings, two test pits, and two plate load tests, was conducted. This program revealed that the fill in the area of the Unit 1 and 2 BWSTs varied from medium to very stiff clay backfill with occasional medium to very dense sand layers over dense to very dense natural sand. This fill was determined adequate to provide support for the postulated loadings from the tanks.

To develop a conservative, long-term settlement prediction, a load test was performed. This test consisted of filling the completed tanks with water. Several weeks after initiation of the test for the Unit 1 tank, a discrepancy was noted between measurements of settlement and the computed displacements derived from the structural analysis used at that time. As a result, the analysis was modified to include a finite-element model of the soil subgrade. A number of analyses were performed using various values for the modulus of elasticity (E) of the soil until the calculated foundation curvature became more severe than observed. The results of the analyses predicted that greater than allowable moments existed at several locations in the foundation (see Figure IV-2).

The foundation at these locations was examined to verify whether visible signs of high reinforcement strain existed. Cracks were found in the structure at those locations indicated by the analysis as having greater than allowable moments. The largest crack measured 0.063 inch. Subsequently, the Unit 2 tank foundation was also examined; similar cracks were found, and the largest crack measured 0.035 inch.

Additional engineering analysis determined that the valve pit, which was lightly loaded, acted as a partial end support and resulted in nonuniform loading of the foundation. This loading condition created differential settlement and localized areas of overstress.

2.0 DESIGN CONCEPT

2.1 CONCRETE FOUNDATION

A two-stage corrective action plan has been adopted for the concrete foundation repair for each tank.

2.1.1 Surcharge Program

Figure IV-3 shows the outline of surcharges that were applied to each valve pit for 4 months. The surcharge consolidated the fill beneath the valve pits, thereby reducing the amount of residual differential settlement of the foundation structure over the 40-year life of the plant. It also provided the added benefit of reducing the ring wall distortion.

2.1.2 Additional Ring Beam

Figures IV-4 and 5 show details of a reinforced concrete ring beam which will be constructed around each existing ring beam. The new ring beam is sized to resist all imposed loading from the tank, including additional future bending induced by the 40-year predicted residual differential settlement between the ring wall and the valve pit. (The predicted value, which was determined from the more severe extrapolated Unit 1 data before applying the surcharge, has not been reduced to account for the beneficial effects of the surcharge and, therefore, is conservative.) All cracks in the existing ring wall that exceed 10 mils will be repaired by pressure grouting. Shear connectors will be installed to transfer the force from the existing ring wall to the new ring beam. One end of the shear connectors will be installed in the existing ring wall by drilling and grouting. The other end will be cast in the new ring beam.

2.2 TANK

The Unit 1 tank (BWST 1T-60) will be releveled after new ring beam construction is complete. The Unit 2 tank (BWST 2T-60) need not be releveled because stresses associated with present plus future predicted differential settlement effects remain within Code-allowable values. Details of the analysis for BWST 1T-60 are provided in Section 3.3.

A detailed procedure has been developed to define a plan of action to relevel BWST 1T-60. This procedure is supported by an analysis that demonstrates that the tank will not be overstressed during this operation. Strain gaging of the tank will be used as a backup to this analysis. This procedure is to be submitted to the NRC staff for review and concurrence prior to performing the work. A brief summary of the procedure is provided below.

a. Vent and drain the tank

b. Mount strain gages

c. Attach electromechanical jacks to the anchor bolt chairs

d. Lift the vessel approximately 3 feet. (All jacks will be controlled from a central control panel and will lift at the same rate and time.)

e. Support tank with cribbing

f. Install Celotex cofferdam around the inner diameter of the ring wall to contain grout placed in Steps l and m below

g. Add and contour oil-impregnated sand

h. Clean the top surface of the ring wall

i. Place stainless steel shims on the original concrete ring wall. Level to a common datum plane above the ring wall. Set shims to the following standard:

   1) 1/8 inch within any 30 feet of circumference

   2) 1/4 inch over total circumference

j. Place Celotex in the areas between the shims

k. Remove cribbing and lower the tank

l. Add nonshrink grout under the tank bottom and allow grout to set

m. Remove the shims, install Celotex, and grout the remaining gaps

3.0 STRUCTURAL ANALYSIS AND DESIGN

3.1 SEISMIC

The preliminary seismic analyses for the BWST foundations are described in Appendix A of the design report submitted by the Applicant to the NRC on November 13, 1981. The final seismic analyses were explained in a November 24, 1981, addendum to the design report. The final seismic analyses are also discussed in testimony by Dr. R.P. Kennedy during the ASLB hearing on December 14, 1981.

The preliminary analyses conservatively determined the seismic shear and overturning moment on the BWST ring foundation from a horizontal final safety analysis report (FSAR) safe shutdown earthquake (SSE). The model included the sloshing and impulsive behaviors of fluid in the tank along with the soil-structure interaction effects. The tank shell was assumed to be rigid. A similar model was used to conservatively determine the seismic forces on the foundation from a vertical FSAR SSE, except there is no sloshing of fluid involved in this case.

Final seismic analyses were performed by Dr. R.P. Kennedy of Structural Mechanics Associates (SMA). The models were the same as those used for the preliminary analyses, except the tank shell was modeled in greater detail for the horizontal seismic load case.

The preliminary and final analyses were also performed to determine the seismic forces on the BWST foundation from earthquakes corresponding to site-specific response spectra (SSRS). The results showed that the forces from the SSRS are smaller than those from 1.5 times FSAR SSE. Also, the preliminary analyses gave consistently higher forces than those from the final analyses. The forces from preliminary analyses for 1.5 times FSAR SSE were used for the BWST ring foundation modification; hence, the design is conservative.

3.2 CONCRETE FOUNDATION DESIGN

3.2.1 Loads, Loading Combinations, and Acceptance Criteria

The modified BWST foundations are designed in accordance with the loading requirements and acceptance criteria for Seismic Category I structures using the load combinations presented in the FSAR.

Because of the presence of differential settlement, four additional load combinations as outlined in the response to NRC Requests Regarding Plant Fill, Question 15 (Revision 3, September 1979) have also been included in the design.

The new ring beam and shear connectors have been designed to withstand the load combinations of American Concrete Institute (ACI) 349-76 as supplemented by Regulatory Guide 1.142.

3.2.2 Static Finite-Element Model

The modified BWST foundation was analyzed by the finite-element method using the Bechtel Structural Analysis Program (BSAP). Because the tank has a flexible bottom, the water and tank bottom loads above the soil are transferred directly to the soil. To account for the settlement effect of the soil from this load, the soil subgrade was modeled in the analysis. The model is divided into two parts: the foundation structure and the soil subgrade. These two parts are connected at the common nodal points at the bottom of the foundation and the outside periphery. At the locations where significant cracks were observed in the ring wall and footing, the thickness of the existing ring wall was reduced by 50% in calculating the thickness of the elements in the model. This increase in flexibility of the foundation structure simulated the effect of cracks.

3.2.3 Soils

3.2.3.1 Elastic Modulus of Soil

Short-term and long-term modulii were developed and utilized in the static finite-element analysis. The long-term modulus is used when considering the effects of settlement combined with dead and live load. The short-term modulus is used for all other loading conditions.

The predicted foundation differential settlement from the finite-element analysis using the long-term modulus is more severe than the 40-year differential settlement prediction based on the Unit 1 load test; hence, the design of the new ring beam is conservative.

3.2.3.2 Foundation Bearing Pressures

The results of the finite-element analysis indicate all the soil elements immediately beneath the foundation structure are in compression for dead load and live load conditions. This behavior indicates that the structure is not lifting off the soil or the soil is not settling down away from the structure at any point. In short, the soil and foundation are displacing in a compatible manner without separation. The maximum calculated soil pressures are within the allowable values for the static and dynamic conditions.

3.3 TANK

3.3.1 Condition Prior to Foundation Repair

A finite-element analysis was conducted on BWST 1T-60 to determine the condition of the tank. Information used in the analysis included survey measurements of the elevations, field measurements of the anchor bolt loads (determined by strain gaging the bolts), a history of the tank filling and draining, and the compressibility of the asphalt-impregnated fibreboard (located between the tank bottom plate and the ring foundation) determined by laboratory testing.

All loads were known from the experimentally determined anchor bolt loads and the weight of tank components. The nonuniform support reactions and resulting tank wall stresses were computed utilizing the finite-element model.

The normal operating stress limits of the governing design code [American Society of Mechanical Engineers (ASME) Boiler and Pressure Vessel Code, Section III, Nuclear Power Plant Components, Subsection NC, 1974, supplemented by ASME Code Case 1607-1 to establish allowable stresses for conditions other than normal operation (infrequent events)] were met with two exceptions.

One exception was that the most highly loaded bolt chair top plate did not meet normal operating stress limits, but the emergency event loading criteria for an ASME Code Class 1 plate and shell-type component support were met. A subsequent dye penetrant examination of the top plate welds verified that no cracking was present.

The other exception was local tank wall compressive stresses which did not meet normal operating stress limits. 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. A visual examination of the tanks was performed while they were under their most highly stressed conditions to verify that no buckling was present.

It is concluded that the uneven tank support which resulted from soil settlement has not resulted in any damage to the BWSTs, that their design basis has not been violated, and that their safe operating life has not been reduced.

3.3.2 Condition After Releveling

A finite-element analysis has been conducted on BWST 1T-60 to determine the tank condition over the operating life of the Midland plant after releveling. An analysis for BWST 2T-60 was not required because BWST 1T-60 had the more severe predicted future settlement pattern. Two loading cases were evaluated: 1) normal operating loads plus settlement, and 2) normal operating loads plus settlement, combined with the effects of the SSRS earthquake. The modeling technique used was that described in Section 3.3.1. The computed stresses are within Code allowables for each case.

4.0 MONITORING PROGRAM

After the new ring beam is constructed, two observation pits will be provided for each BWST foundation at the high stress locations. The new ring beams will be monitored monthly for possible cracks under service conditions for 6 months after filling the tanks. At the end of the monitoring period, a report evaluating cracks will be submitted to the NRC. If during the monitoring period any cracks are noted which are 30 mils or larger, an engineering evaluation will be conducted to determine corrective action.

BWST foundation settlement will also be monitored as part of the foundation survey. Foundations are surveyed at 60-day intervals during construction and at 90-day intervals for the first year of plant operation. Subsequent survey frequency will be established after evaluating the data taken during the first year of plant operation. As a minimum, the tank foundation would be monitored annually for the next 5 years of operation and at 5-year intervals thereafter.

The critical areas of each foundation at the transition zone between the ring wall and the valve pit will be monitored using a strain gage system. This system will be monitored at the same frequency as the foundation survey using established acceptance criteria.

5.0 CONSTRUCTION

The NRC has given its concurrence to the repair of cracks in the existing foundation. Preparation for this work is under way.

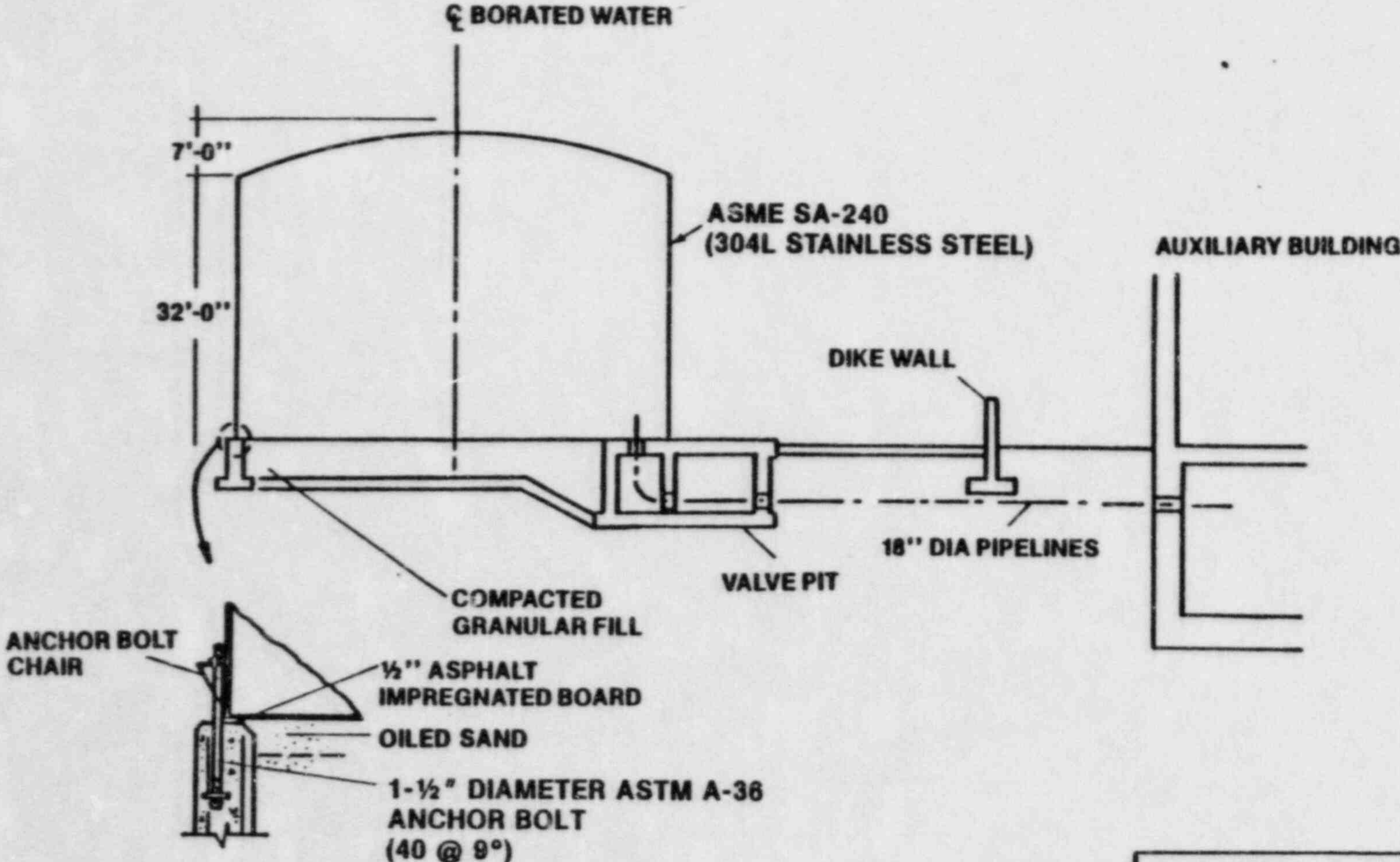
6.0 LICENSING STATUS

The remedial plan for the BWSTs has been presented in meetings and reports to the NRC and in the Atomic Safety and Licensing Board (ASLB) hearing. Meetings were held on May 7, 1981, to discuss the concept of building an additional ring; on August 3, 1981, to discuss application of the surcharge to the valve pit; on January 13, 1982, to discuss the analysis of the existing condition of the tanks; and during the January 18 through 20, 1982, audit to discuss crack repair, tank releveling, and analysis techniques.

A technical report was submitted on November 13, 1981, which described the design concept, provided details of the seismic and static analytical methods, and presented construction details. An addendum to the report was submitted on November 24, 1981, which provided the results from the final seismic analysis and verification that design acceptance criteria had been met.

In the December 14, 1981, ASLB hearing, Drs. R.P. Kennedy of SMA and P. Hadala of the U.S. Army Corps of Engineers (consultant to the NRC) testified to the adequacy of the seismic model and associated analyses. During the February 16 through 19, 1982, ASLB hearing, the Applicant and NRC staff testified to the adequacy of the proposed remedial plan and the acceptance of the tanks.

The NRC staff intends to audit the final design calculations. The NRC staff has documented its concurrence on the application and removal of the surcharge and the repair of cracks in the existing foundation.

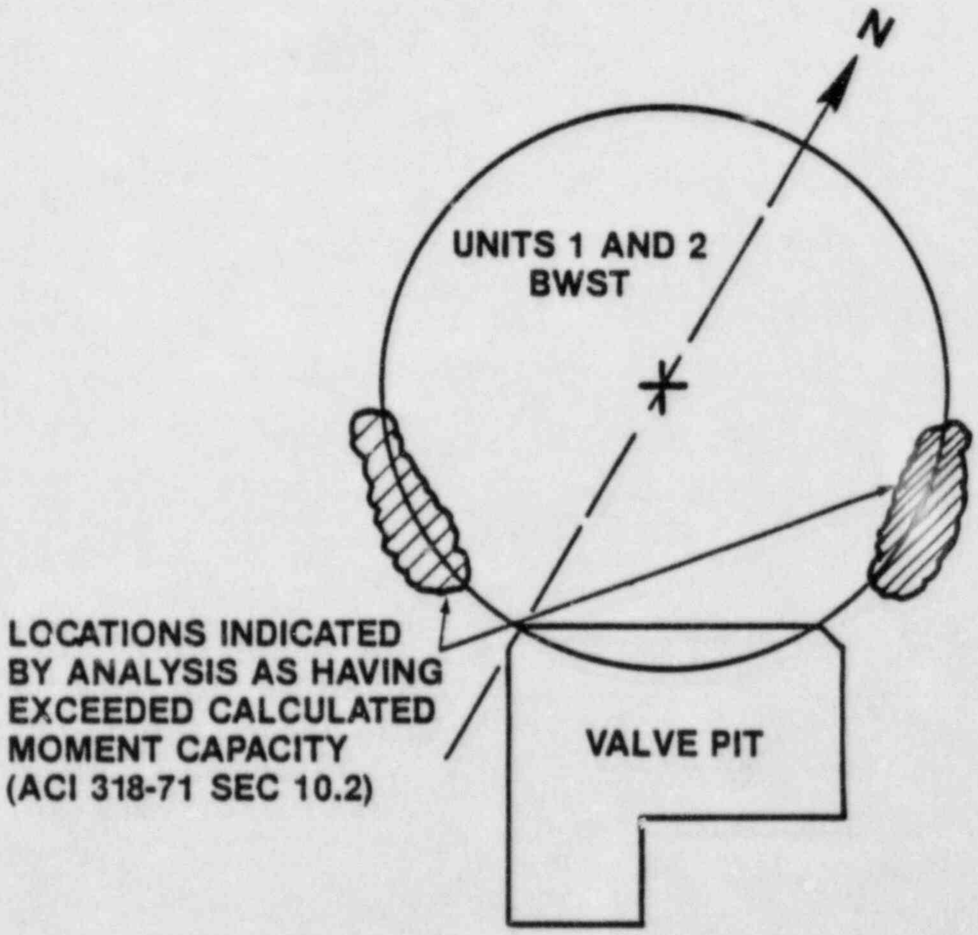


CONSUMERS POWER COMPANY
MIDLAND UNITS 1 AND 2

BORATED WATER STORAGE TANK

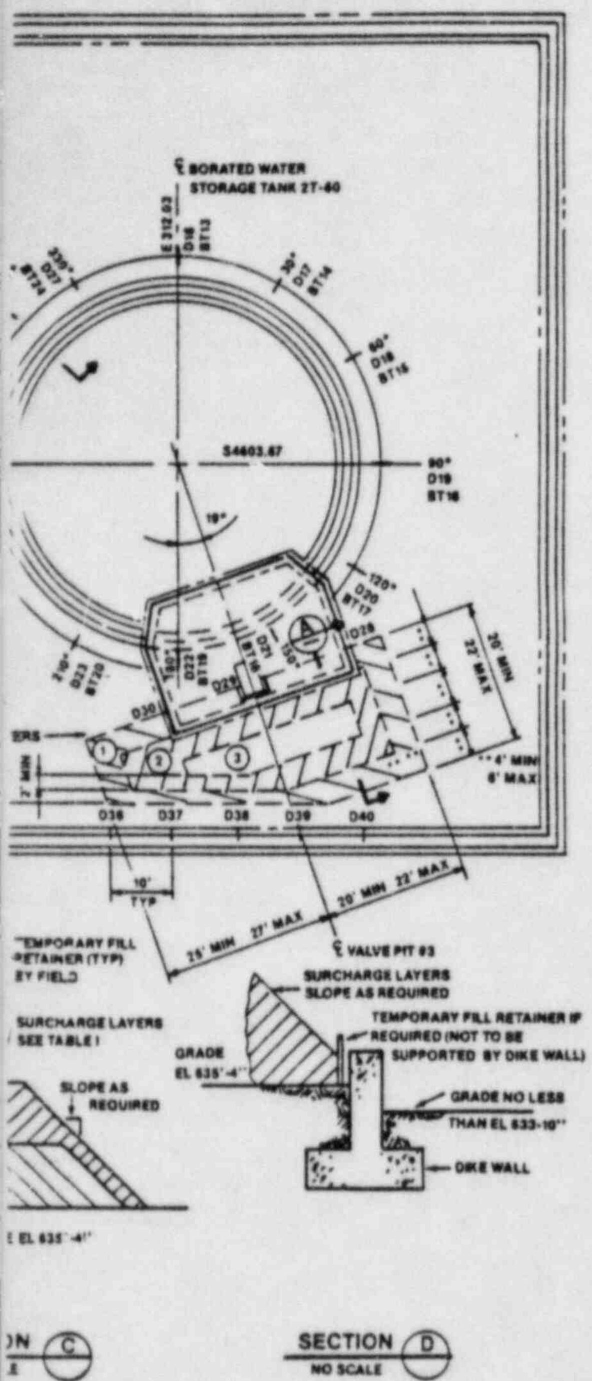
FIGURE IV-1

G-1986-27



CONSUMERS POWER COMPANY MIDLAND UNITS 1 AND 2
LOCATIONS OF EXCESSIVE MOMENT
FIGURE IV-2

G-1986-18



* Granular fill shall be placed to a height such that its weight is equivalent to the weight of the concrete blocks in the corresponding area.

TABLE II
SETTLEMENT MONITORING

DATA BASE NO.	LOCATION 1T-60	ELEVATION (approx)	DATA BASE NO.	LOCATION 2T-60	ELEVATION (approx)
D1	0°	635'-0"	D16	0°	635'-0"
D2	30°	635'-0"	D17	30°	635'-0"
D3	60°	635'-0"	D18	60°	635'-0"
D4	90°	635'-0"	D19	90°	635'-0"
D5	120°	635'-0"	D20	120°	635'-0"
D6	150°	635'-0"	D21	150°	635'-0"
D7	180°	635'-0"	D22	180°	635'-0"
D8	210°	635'-0"	D23	210°	635'-0"
D9	240°	635'-0"	D24	240°	635'-0"
D10	270°	635'-0"	D25	270°	635'-0"
D11	300°	635'-0"	D26	300°	635'-0"
D12	330°	635'-0"	D27	330°	635'-0"
D13	VALVE PIT	635'-0"	D28	VALVE PIT	635'-0"
D14	VALVE PIT	635'-0"	D29	VALVE PIT	635'-0"
D15	VALVE PIT	635'-0"	D30	VALVE PIT	635'-0"
D41	VALVE PIT	626'-4"			
D31	DIKE WALL	637'-6"	D36	DIKE WALL	637'-6"
D32	DIKE WALL	637'-6"	D37	DIKE WALL	637'-6"
D33	DIKE WALL	637'-6"	D38	DIKE WALL	637'-6"
D34	DIKE WALL	637'-6"	D39	DIKE WALL	637'-6"
D35	DIKE WALL	637'-6"	D40	DIKE WALL	637'-6"
BT1	0°	635'-6"	BT13	0°	635'-6"
BT2	30°	635'-6"	BT14	30°	635'-6"
BT3	60°	635'-6"	BT15	60°	635'-6"
BT4	90°	635'-6"	BT16	90°	635'-6"
BT5	120°	635'-6"	BT17	120°	635'-6"
BT6	150°	635'-6"	BT18	150°	635'-6"
BT7	180°	635'-6"	BT19	180°	635'-6"
BT8	210°	635'-6"	BT20	210°	635'-6"
BT9	240°	635'-6"	BT21	240°	635'-6"
BT10	270°	635'-6"	BT22	270°	635'-6"
BT11	300°	635'-6"	BT23	300°	635'-6"
BT12	330°	635'-6"	BT24	330°	635'-6"

TABLE I
SURCHARGE SEQUENCE

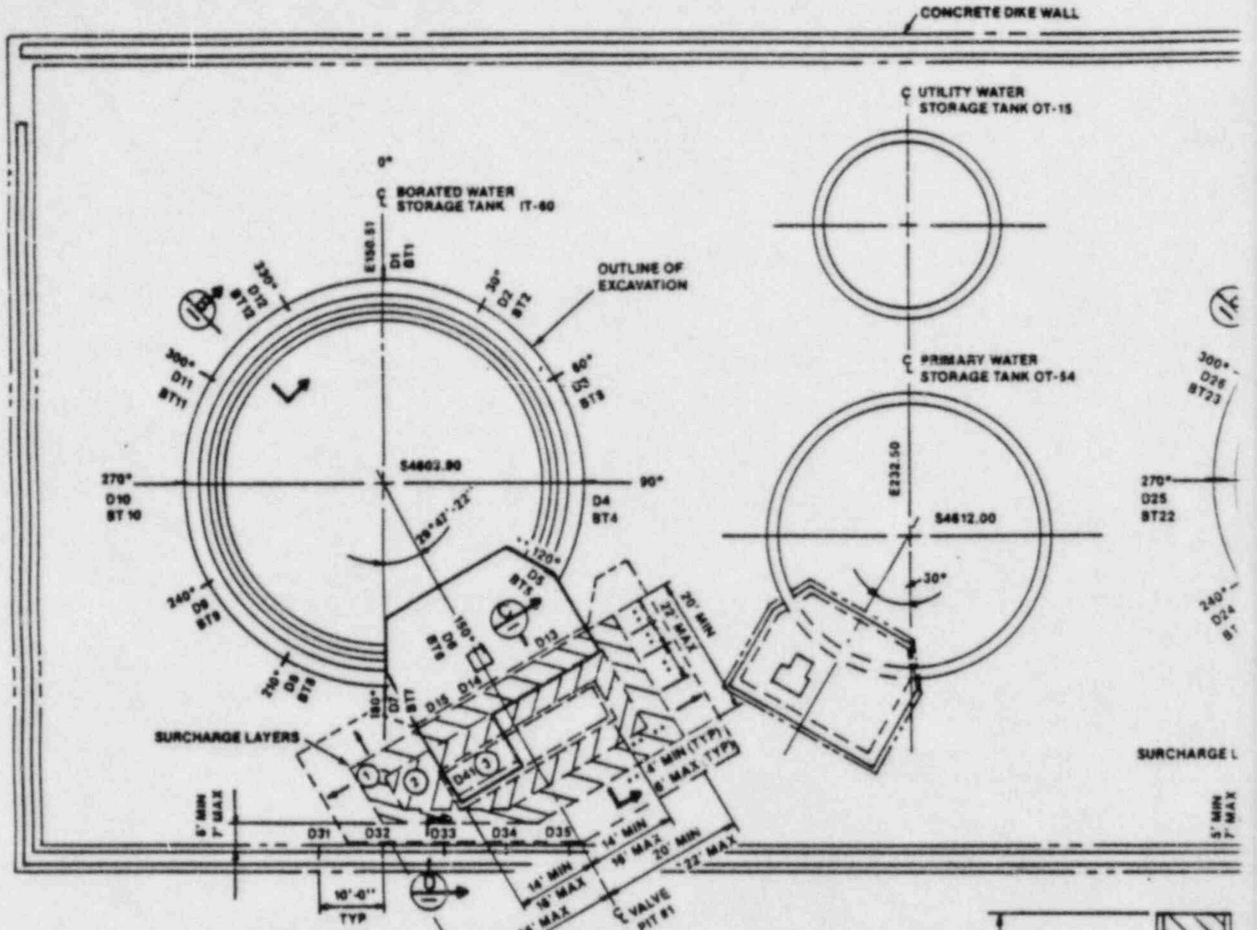
STEP	ACTIVITIES	HT OF SURCHARGE SEE NOTE 3	
		1T-60	2T-60
I	1. UTILITY MONITORING SHALL BE COMPLETED 2. CUSHION PAD SHALL BE COMPLETED 3. WATER HEIGHT IN TANK AND ANCHOR BOLT STATUS SHALL BE AS DIRECTED BY PROJECT ENGINEERING	0'-0"	0'-0"
II	1. PLACE SURCHARGE LAYER ① AND HOLD FOR TWO WEEKS	SEE SECTION C	4'-0" MIN 6'-0" MAX
III	1. PLACE SURCHARGE LAYER ② AND HOLD FOR TWO WEEKS	SEE SECTION C	8'-0" MIN 10'-0" MAX
IV	1. PLACE SURCHARGE LAYER ③ AND HOLD FOR SETTLEMENT DATA AS DIRECTED BY PROJECT ENGINEERING	SEE SECTION C	20'-0" MIN 22'-0" MAX
V	1. REMOVE SURCHARGE LAYER ③ AND HOLD FOR TWO WEEKS	SEE SECTION C	10'-0" MIN 12'-0" MAX
VI	1. REMOVE REMAINDER OF SURCHARGE LAYERS ①, ②, AND TAKE REBOUND DATA	0'-0"	0'-0"

CONSUMERS POWER COMPANY
MIDLAND UNITS 1 AND 2

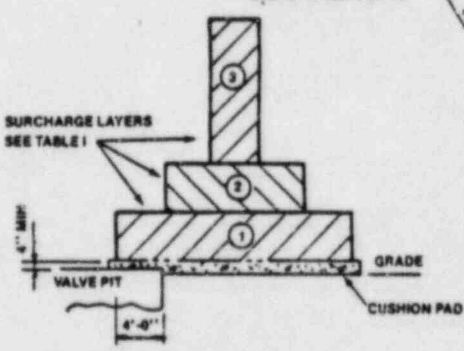
BORATED WATER STORAGE TANKS VALVE PIT SURCHARGE PROGRAM

FIGURE IV-3

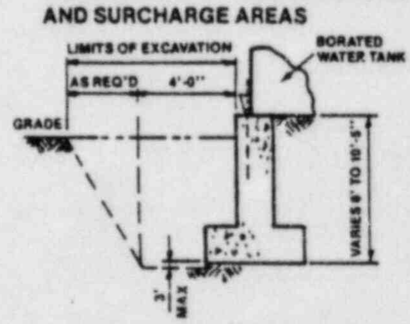
G-1986-28



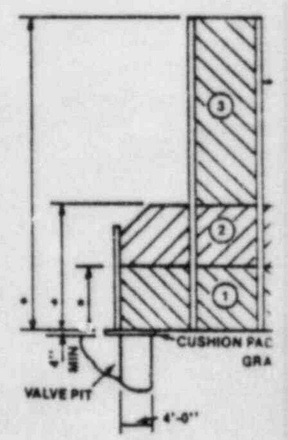
PLAN OF MONITORING LOCATIONS AND SURCHARGE AREAS



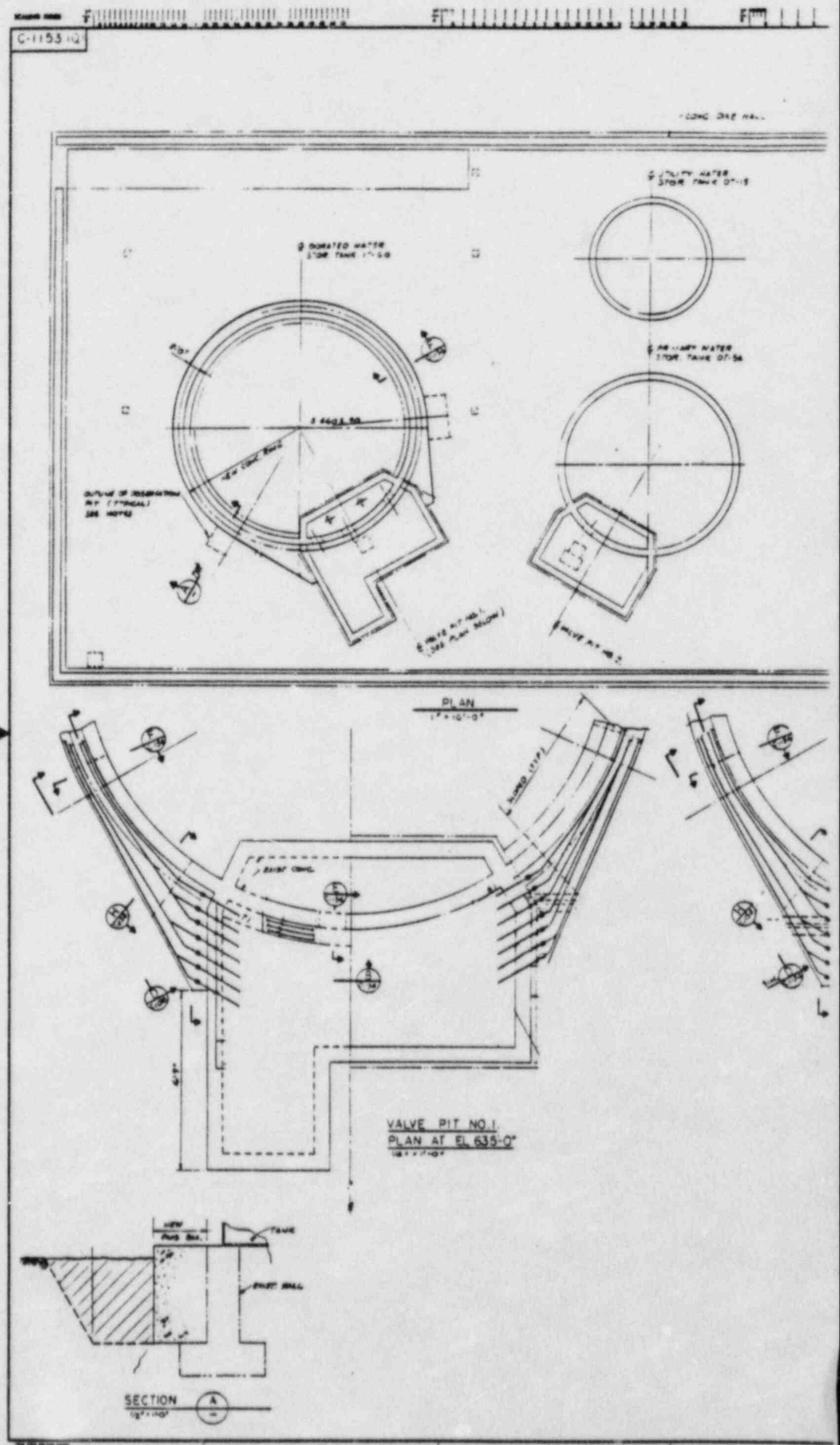
SECTION A
NO SCALE

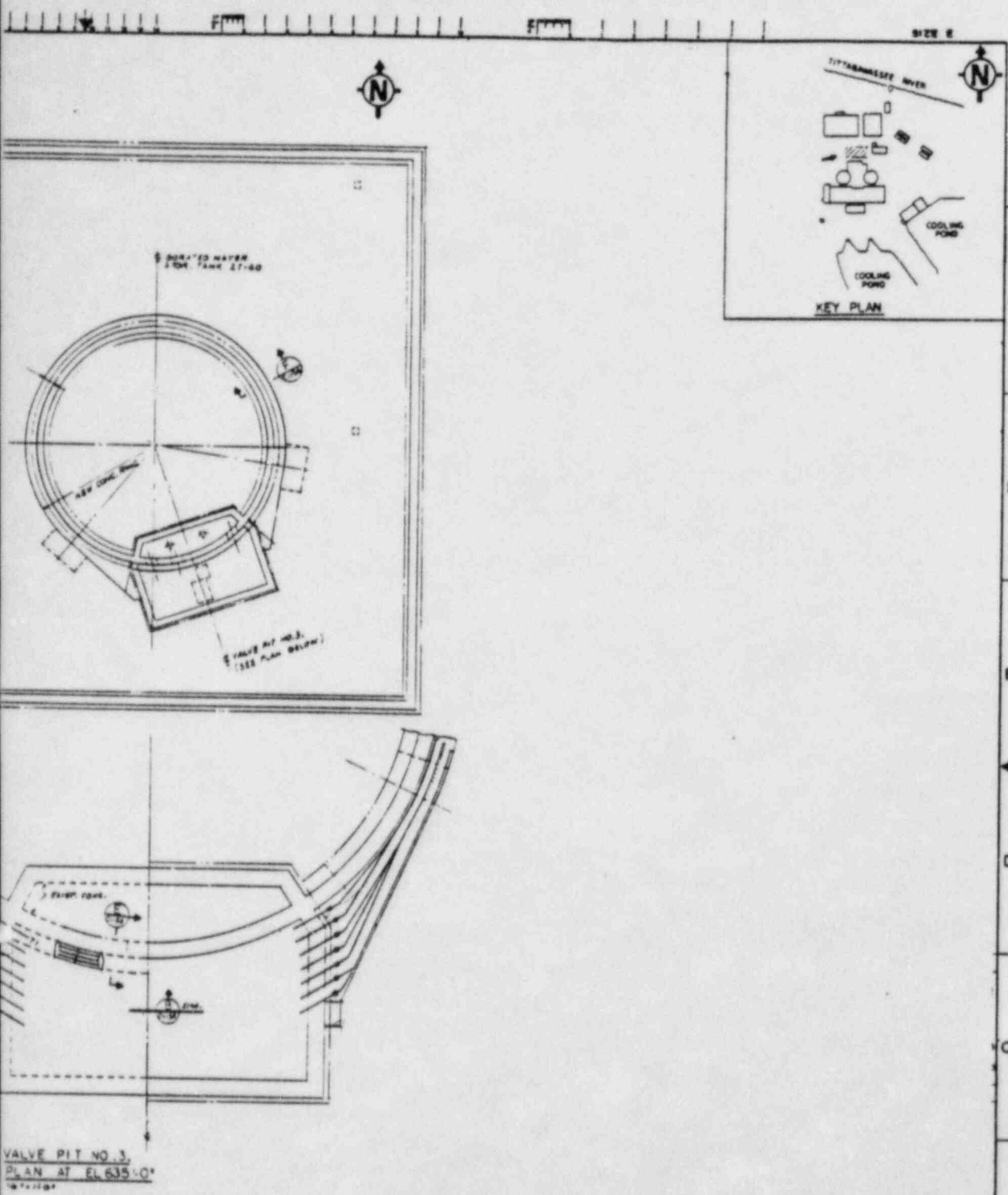


SECTION B TYP AT IT-60, 2'-60
NO SCALE



SECT
NO SC

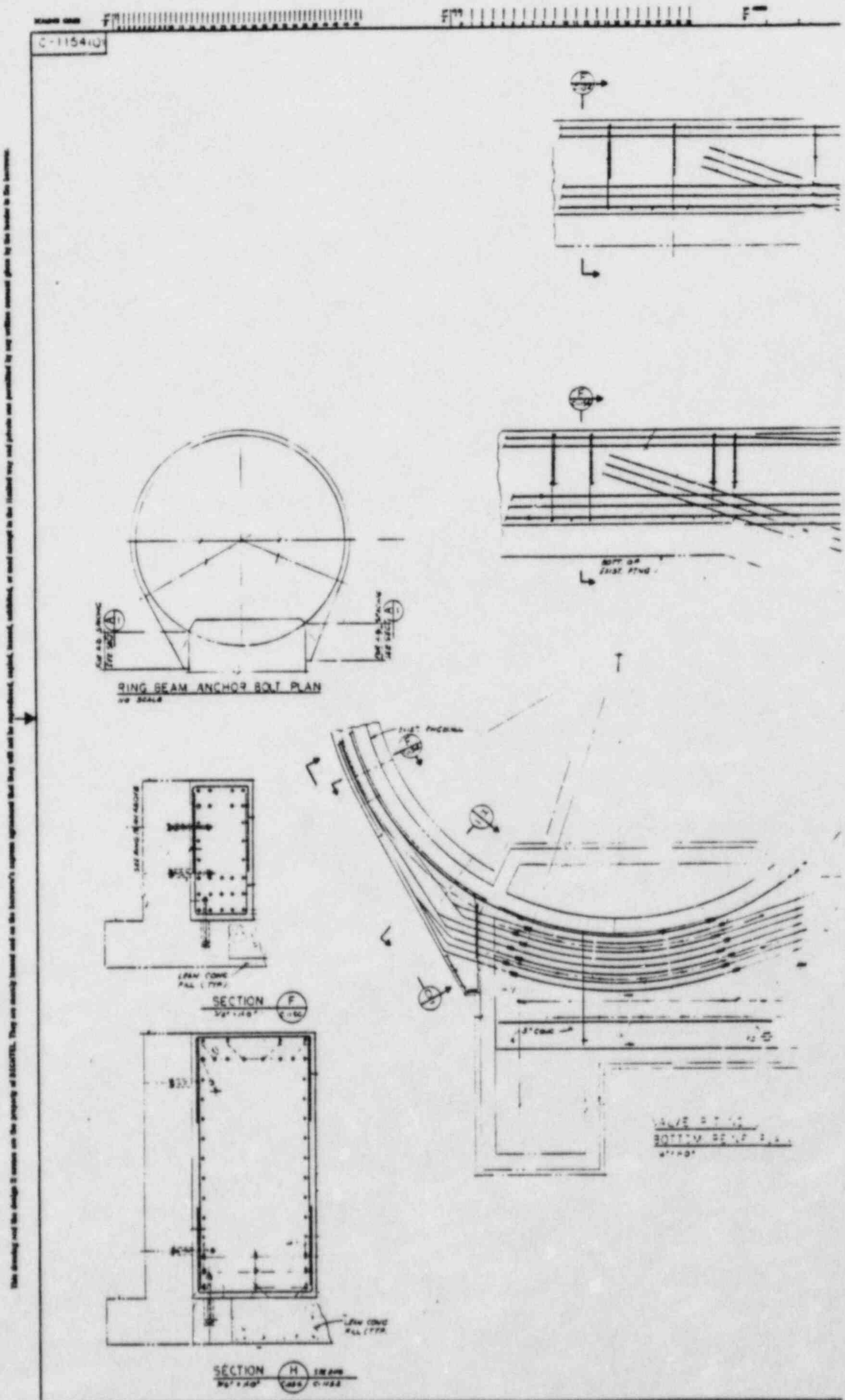

C-1153-10
BORATED WATER STOR. TANK
UTILITY WATER STOR. TANK
PRIMARY WATER STOR. TANK
OUTLINE OF EXCAVATION PIT (TYPICAL) SEE NOTES
VALVE PIT NO. 1 (SEE PLAN BELOW)
VALVE PIT NO. 2
PLAN
EXIST. CONC.
VALVE PIT NO. 1
PLAN AT EL. 635'-0"
SECTION A
TANK
EXIST. WALL
8
7
6
5



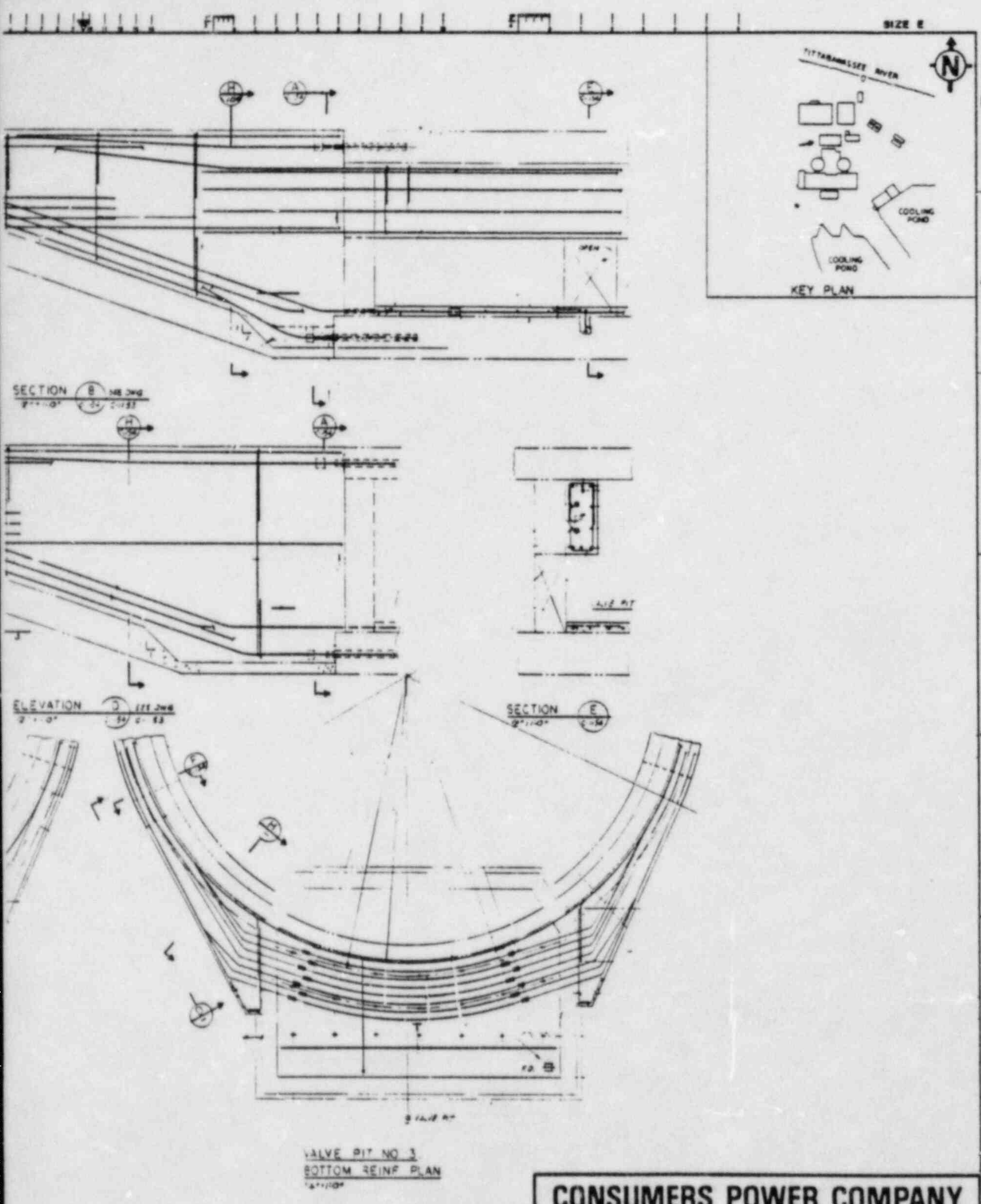
CONSUMERS POWER COMPANY
MIDLAND UNITS 1 AND 2

BORATED WATER STORAGE TANK FOUNDATION MODIFICATION SHEET 1

FIGURE IV-4



RING BEAM ANCHOR BOLT PLAN
NO SCALE
SECTION F
SECTION H
LEAN CONC FILL (TYP)
8
7
6
5



CONSUMERS POWER COMPANY
MIDLAND PLANT UNITS 1 AND 2

BORATED WATER STORAGE TANK FOUNDATION MODIFICATION SHEET 2

FIGURE IV-5

PART V: PERMANENT DEWATERING

1.0 INTRODUCTION

As a result of the site soils exploration program conducted after the discovery of the diesel generator building (DGB) settlement problem, pockets of potentially liquefiable granular backfill materials have been discovered supporting some Seismic Category I structures and buried utilities. Facilities affected include: the DGB, auxiliary building electrical penetration areas, auxiliary building railroad bay, the cantilevered section of the service water pump structure (SWPS), and a portion of the service water piping adjacent to the DGB, auxiliary building, circulating water intake structure (CWIS), and the SWPS.

Evaluation of site exploration data performed by the Applicant, the NRC staff, and its consultant (the U.S. Army Corps of Engineers) concluded that loose granular backfill supporting Seismic Category I facilities is safe against liquefaction for earthquakes that produce a peak ground surface acceleration of 0.19g or less provided the groundwater elevation in the backfill is maintained at or below el 610'.

The auxiliary building electrical penetration areas and the cantilevered portion of the SWPS will be underpinned. The service water piping adjacent to the CWIS and SWPS will be excavated to at least el 610' and rebedded to meet design requirements. These remedial steps will eliminate liquefaction as a potential problem in these areas.

In the area of the DGB and auxiliary building railroad bay, there is still a potential during the safe shutdown earthquake (SSE), which is less than 0.19g, for liquefaction in saturated backfill sands that exist above el 610'. Critical areas where the groundwater levels have to be maintained below el 610' in granular backfill supporting Seismic Category I structures and buried utilities are shown in Figure V-1.

2.0 DESIGN CONCEPT

To eliminate the potential for liquefaction during the design SSE in loose, saturated granular backfill materials for the areas designated in Figure V-1, a permanent plant dewatering system has been designed to remove the water from the backfill sands and maintain it below el 610'.

The permanent dewatering system operating level has been selected to be el 595'. This level was selected, based upon site tests, to provide time for repair or replacement of the system before groundwater levels would rise above el 610' at the critical areas.

The permanent dewatering system consists of two subsystems: interceptor wells and area dewatering wells. The design of these systems accounts for the two basic findings of the exploration and testing program: 1) The granular backfill materials are hydraulically connected to the underlying natural sands, and 2) The cooling pond, at el 627', is the main source of recharge, and seepage from the pond is occurring primarily at the CWIS and SWPS.

The dewatering system will be monitored during plant operation. This will ensure that the water level stays below el 610', that soil particles removed are below predetermined levels, and that water quality is acceptable for disposal.

The system has also been designed to ensure its operation during various accident conditions, including power outages, loss of wells, and pipe breakage.

The NRC staff has been provided information about the dewatering system design in response to 10 CFR 50.54(f), Questions 24, 47, and 49 through 53, and letters from the Applicant to H.R. Denton dated April 24, 1981; May 28, 1981; and September 16, 1981. The NRC staff have concurred with the proposed system.

3.0 ANALYSIS AND DESIGN

The permanent dewatering system design is based on an evaluation of design drawings and construction records, test boring information, field and laboratory test results, observation well and piezometer data, and pumping test results. The data obtained from these activities include type, distribution, and permeability of materials; zones of recharge and drawdown; and recharge and pumping rates. This information has been used to determine the location, spacing, size, and depth of the dewatering wells.

As stated earlier, the system consists of two subsystems: interceptor wells and area dewatering wells.

3.1 INTERCEPTOR WELLS

The first subsystem is a line of 20 interceptor wells around the CWIS and SWPS area (see Figure V-2). This line of wells was designed to prevent cooling pond water from moving through the backfill and natural sands toward the DGB and auxiliary building railroad bay areas. It will also help lower groundwater levels in the backfill and natural sands near the cooling pond so that if the dewatering wells become inoperable, the rate of groundwater level rise in the plant area will be slow enough to allow either activation of the backup dewatering system (20 backup wells corresponding to 20 interceptor wells) or effect repair or replacement of defective wells before the groundwater level reaches el 610' at either the DGB or auxiliary building railroad bay areas.

3.2 AREA DEWATERING WELLS

The second subsystem, consisting of 24 area wells distributed over the plant site area, was designed to remove the groundwater stored within the backfill and natural sands and then to maintain the groundwater level (see Figure V-2). This subsystem design utilizes the extensive natural sands underlying the backfill as a drain.

4.0 RECHARGE TIME

Analysis of data from pumping tests and from groundwater level responses to changes in cooling pond level indicates there is time available to repair or even replace the entire system before the design groundwater level would be exceeded at the critical areas. To further verify this conclusion, a full-scale test was performed between February 4 and April 5, 1982, after the groundwater levels had been lowered to el 595' or as low as practical and with the cooling pond at el 627'. The groundwater levels were lowered using only 20 permanent backup dewatering wells, existing construction dewatering wells, selected individual observation wells equipped with self-contained eductors, and temporary dewatering wells. During this test, groundwater level-versus-time curves were plotted to determine the actual recharge time at the DGB and auxiliary building railroad bay areas. The results of this test indicate that groundwater levels rise faster at the DGB than at the auxiliary building railroad bay and that there is at least 60 days' recharge time available to repair or perform maintenance on the dewatering system before groundwater levels would reach el 610' at the DGB (see Figures V-3 and V-4).

Results and progress of the recharge testing program were presented to the NRC staff in Bethesda, Maryland, on February 23 and March 3, 1982, and by telephone communication on April 5, 1982.

5.0 WELL INSTALLATION

On March 23, 1981, the Applicant sent a letter to the NRC staff requesting staff concurrence with the installation of 20 backup interceptor wells. After discussions in April, May, and part of June, the staff agreed to a slightly modified version of the proposal. Staff concurrence at that time included only 12 of the 20 wells, because the staff required additional information regarding soil conditions at the locations of the remaining eight wells. Concurrence regarding the final eight permanent wells was secured on September 2, 1981.

The 20 permanent backup dewatering wells were installed between August 17, 1981, and October 29, 1981, by a dewatering subcontractor. The architect-engineer's geologist/hydrogeologist prepared as-built drawings of each well installation, including well number, location, diameter of hole, total depth, and description of each type of casing; a log of subsurface materials encountered; and a complete compilation of field data obtained during drilling, installation, and developing of the wells including data requested by the NRC.

NRC concurrence to install the remaining permanent dewatering wells (20 interceptor, 24 area, and 6 monitoring) was given on October 22, 1981. The remaining wells are currently being installed in accordance with the same procedures, criteria, materials, methods, supervision, and inspection used for the installation of the 20 permanent backup wells. Construction of the permanent wells is about 65% complete.

6.0 MONITORING SAFEGUARDS

6.1 INITIAL OPERATING PERIOD

Groundwater quality, pumping rates, drawdown levels, and hours of operation will be monitored during the initial operating period so that an operating history of each well is established prior to plant operation. By comparing collected data, any decrease in production efficiency will be detected.

Near the end of the initial operating period, after the groundwater in storage has been removed and the groundwater levels have stabilized at or below el 595', the frequency of monitoring groundwater levels, soil particle content, and water quality will be determined for implementation during plant operation.

6.2 PLANT OPERATION

During plant operation, monitoring procedures will be performed under a quality assurance program. When it is determined by analyzing available data that a well or group of wells is no longer functioning, corrective measures will be taken. These corrective measures may include cleaning the well screens, repairing or replacing screens or any mechanical parts, or installing a new dewatering well, if necessary.

A complete set of replacement parts will be stored onsite for any repair, replacement, or installation that may be required. As a result of the proposed monitoring of the well system, any significant rise in the groundwater level will be detected in time to take remedial actions before the critical groundwater elevation (el 610') is reached at the DGB or auxiliary building railroad bay areas.

6.3 GROUNDWATER MONITORING

The dewatering system is self-verifying. This means that many design parameters and most design analyses used in the permanent dewatering system may be verified by direct observation of water levels at the Midland site. In addition, monitoring is an integral part of the system operation.

Six permanent monitoring wells are planned. Each permanent monitoring well is of the same design as a permanent well, except each permanent monitoring well will contain an ultrasonic level transmitter to continuously record the groundwater level. The locations of the permanent monitoring wells are shown in Figure V-2. These locations were selected based on their proximity to the critical areas and their position in the backfill and natural sand (two at the DGB, two at the auxiliary building railroad bay, and two north of the interceptor well system).

Currently, over 50 observation wells exist at the site to monitor various depths within the backfill and natural sands. A select number of these wells will be maintained for measurement over the life of the plant.

7.0 SYSTEM DESIGN FOR ACCIDENT CONDITIONS

The dewatering system is not a Seismic Category I system; it is not required to operate during or after an SSE. Instead, the system design is based on the conclusion that, following natural circumstances that may cause total or partial failure of the system, time exists to make necessary repairs before the potential for liquefaction develops. A worst-case assumption (the total failure of all pumping capacity in the system) would still permit time to repair or reinstall the system before the water level in liquefiable soils in the DGB and auxiliary building train bay areas reaches el 610'. This conclusion was verified by the full-scale recharge test described in Section 4.0. A summary of well failure mechanisms and repair times is presented in Table V-1. Additional discussions with the NRC staff concerning accident conditions and system response occurred at meetings with the staff on February 23 and March 3, 1982.

7.1 POWER OUTAGES

Less severe accident conditions (e.g., a partial break in the dewatering header system, line breaks outside the dewatering system, or power outages) have also been accounted for in the system design. Electrical wiring of the system will be designed such that the temporary outage of one or more wells will have no effect on the remaining wells. In addition, should any disruption in the overall power supply occur, backup diesel generator power will be available for temporary operation of the primary interceptor wells and/or backup well pumps until normal power is restored.

7.2 UNINTERRUPTED SERVCE

Assurance of uninterrupted service in the event of a partial loss of system wells is also provided by a number of redundancies built into the dewatering system. Twenty backup wells located at the CWIS and SWPS will provide standby pumping capacity for the 20 interceptor wells in this area. Another 24 area wells are available to remove any water not collected by the interceptor wells. Thus, 64 wells have been incorporated into the dewatering system design, each with a submersible pump having the capacity of at least 10 gpm. Normal operations to maintain the groundwater level at or below el 595' during the life of the plant is estimated to require only 22 of these wells.

7.3 PIPE BREAKS

The dewatering system design also accounts for pipe breaks, both at the interceptor wells and at the critical areas. Pipe breaks that would immediately impact the interceptor well system include breaks of a dewatering system header line, concrete pipe cooling pond blowdown line, concrete pipe cooling tower line, or service water discharge line. At the request of the NRC staff, the Applicant also analyzed a nonmechanistic failure of both the Unit 2 circulating water discharge pipe and the 20-inch diameter condensate water pipe near the DGB.

7.3.1 <u>Damage to the Dewatering System Header Line</u>

Damage to the dewatering system header line could result in return flow to the dewatering wells in the vicinity of the broken line. In that event, the combination of groundwater recharge and surface water inflow could exceed the capacity of the affected pump, producing a rise in groundwater level. To account for this possibility, the dewatering system will be designed to permit a flexible hose to be attached to the individual wells. If a header line breaks, a hose would be attached to each well to temporarily divert flow to the system's catch basins until the header line is repaired. In the case of an interceptor well header failure, the backup wells can be activated because they are on a separate header system. This arrangement will prevent an overload of the pumping capacity of an individual well or of a group of wells.

7.3.2 Break of Either Concrete Pipe Blowdown or Cooling Tower Lines

A break of either the concrete pipe blowdown line or the cooling tower line at the CWIS and SWPS could result in the loss of three dewatering wells. The impact of such a pipe break on the entire dewatering system, however, would be minimal. The total amount of water released by a break in either of these low-pressure lines would not produce a significant rise in the overall plant groundwater levels, even if all the released water entered the groundwater system.

Following a pipe break, the flow of the water would be shut off and the backup interceptor wells would automatically activate. The backup interceptor wells and remaining primary wells will have sufficient capacity to remove recharge from the cooling pond until the damaged wells can be replaced. Excess water introduced into the area by the pipe break would be removed by the area dewatering system.

7.3.3 Nonmechanistic Failure of the Unit 2 Circulating Water Pipe

Potential hazards from the nonmechanistic failure of the circulating water discharge pipe near the DGB were assessed by determining the time necessary for the rise in water level to activate a permanent area dewatering well. It was determined that groundwater levels would be significantly below the critical elevation when the permanent area dewatering wells would be activated.

7.3.4 Nonmechanistic Failure of the 20-Inch Condensate Pipe

A nonmechanistic failure of the 20-inch diameter condensate water pipe, which is located directly beneath the DGB, was analyzed. Using a simplified analysis, it was assumed that the entire contents of the condensate water tank (300,000 gallons) were spilled directly beneath the DGB. Further, it was conservatively assumed that all the water would be contained beneath the building. From this analysis, it was determined that the groundwater elevation would not rise above el 610'.

Because the volume of water in the condensate storage tanks is less than the volume required to fill the area beneath the DGB to el 610', a failure of the condensate water pipe would be accommodated even if no permanent area dewatering wells were operating in this area.

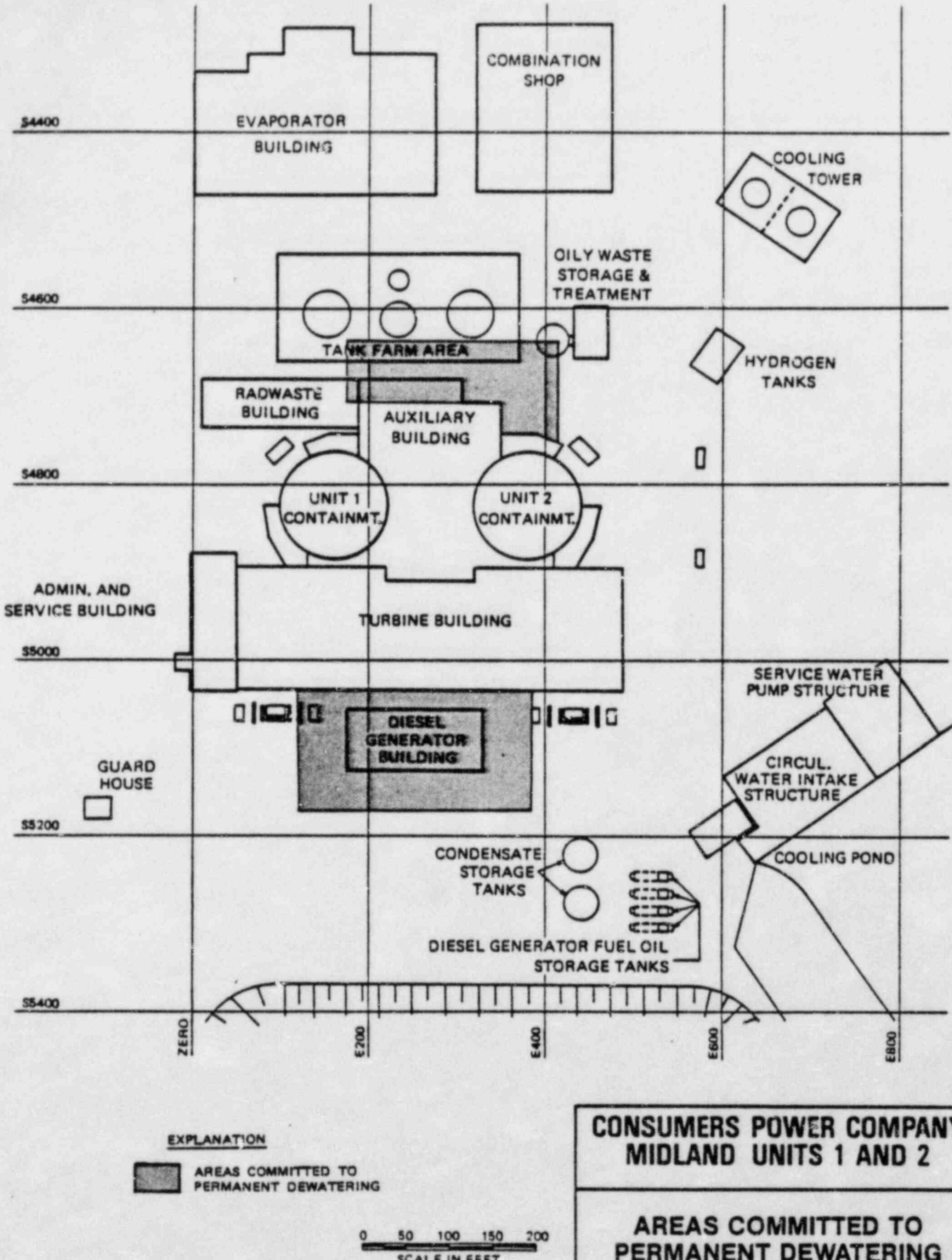
8.0 TECHNICAL SPECIFICATIONS

After the plant operator has verified that a water level measurement higher than el 595' is a correct reading and the repair measures given in Table V-1 do not affect the rise in groundwater level at the DGB or auxiliary building railroad bay, the plant will be shut down when any observation well at either critical structure exceeds el 607' (see Figure V-5). A technical specification will be prepared detailing the coordination of the shutdown.

TABLE V-1

WELL FAILURE MECHANISMS AND RESPONSES

Event	50.54(f) Reference	Repair Time
1. Electrical Failure		
a. Single well (wired in parallel)	24.a, 24.c, 47.1.b	Less than 1 day.
b. Multiple wells due to power outage	24.a, 24.c, 47.1.b	1 day to initiate operation of backup diesel power to interceptor wells. Operate until normal power can be restored. Backup interceptor wells automatically begin pumping if water levels exceed el 595'.
2. Failure of timers/pumps/check valves	24.c, 47.1.b, 47.6	Less than 1 day; replacement parts onsite.
3. Header pipe break	24.c	1 day to attach flexible hose to each well affected and pump water to storm drains. In case of interceptor well header failure, initiate backup wells (on separate header system).
4. Well screen encrustation	24.h, 47.6, 47.8	2 days to acidize well.
5. Complete loss of well	24.c, 47.1.b	4 days to replace one well using cable tool rig. 1 day if other drilling method used. If well or wells need to be replaced, there is enough redundancy and pumping capacity to prevent water levels from rising in plant fill, while the replacement wells are being installed.

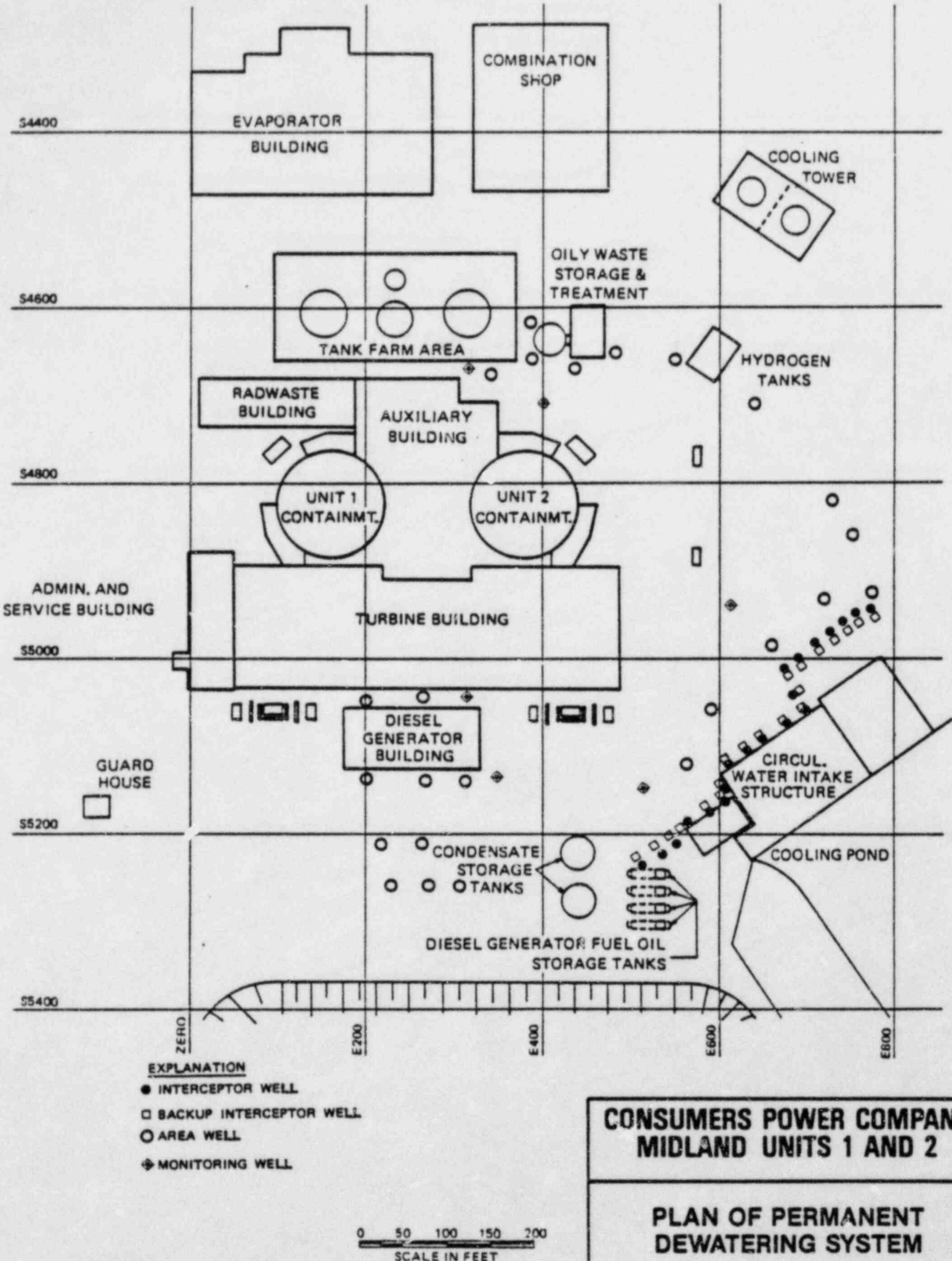


CONSUMERS POWER COMPANY
MIDLAND UNITS 1 AND 2

AREAS COMMITTED TO PERMANENT DEWATERING

FIGURE V-1

G-1986-22

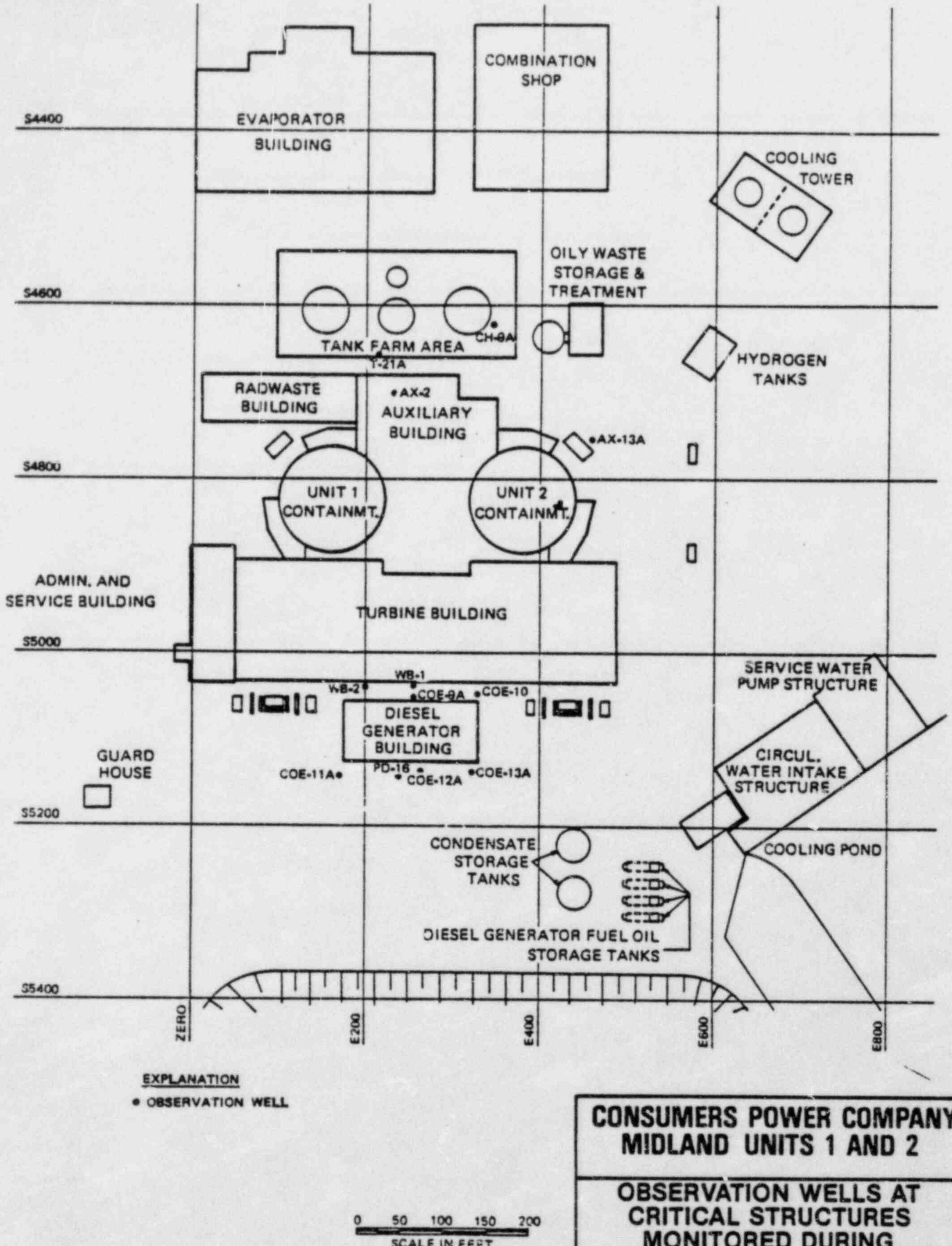


CONSUMERS POWER COMPANY
MIDLAND UNITS 1 AND 2

PLAN OF PERMANENT DEWATERING SYSTEM

FIGURE V-2

G-1986-23

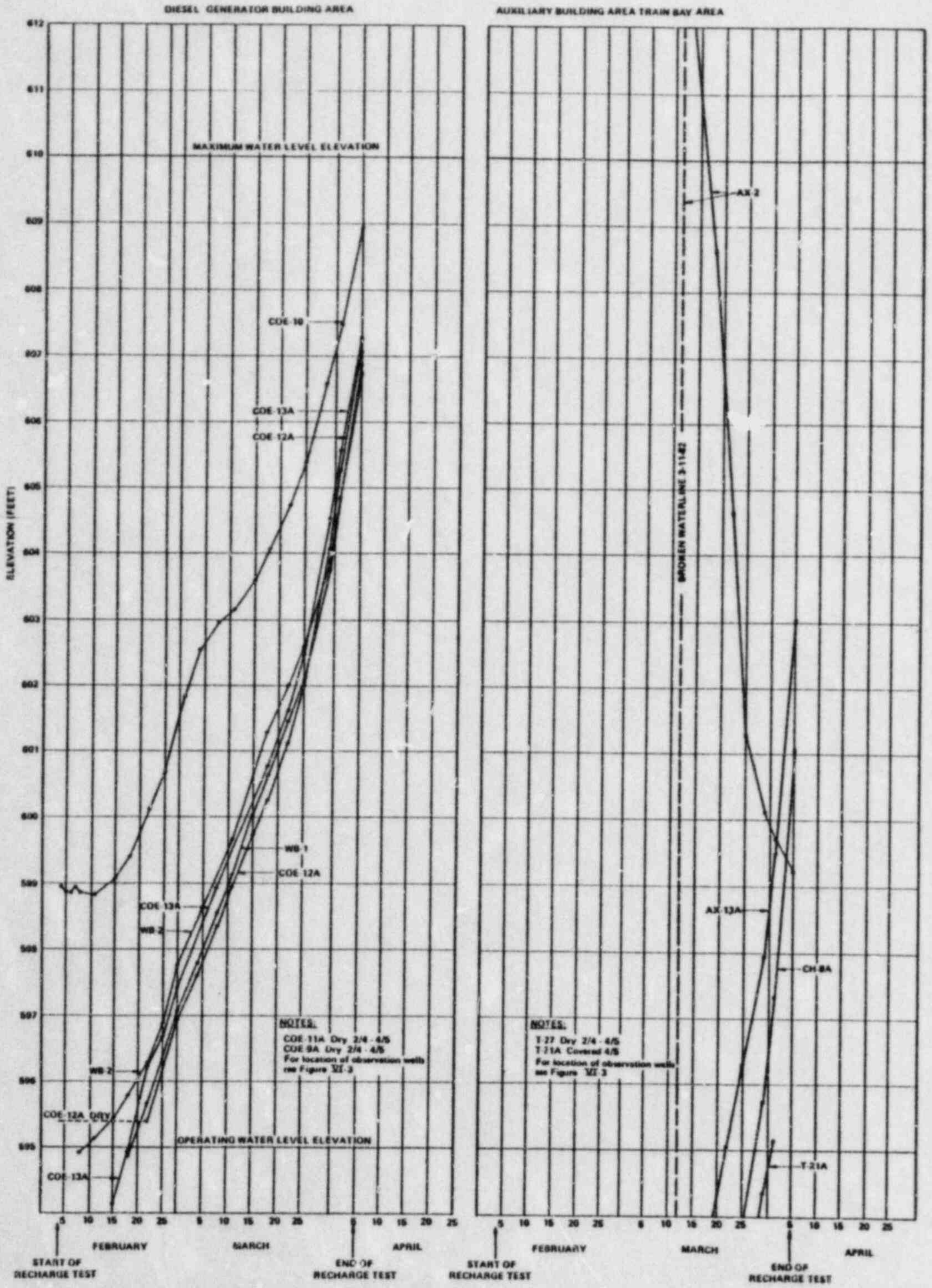


CONSUMERS POWER COMPANY
MIDLAND UNITS 1 AND 2

OBSERVATION WELLS AT CRITICAL STRUCTURES MONITORED DURING RECHARGE TEST

FIGURE V-3

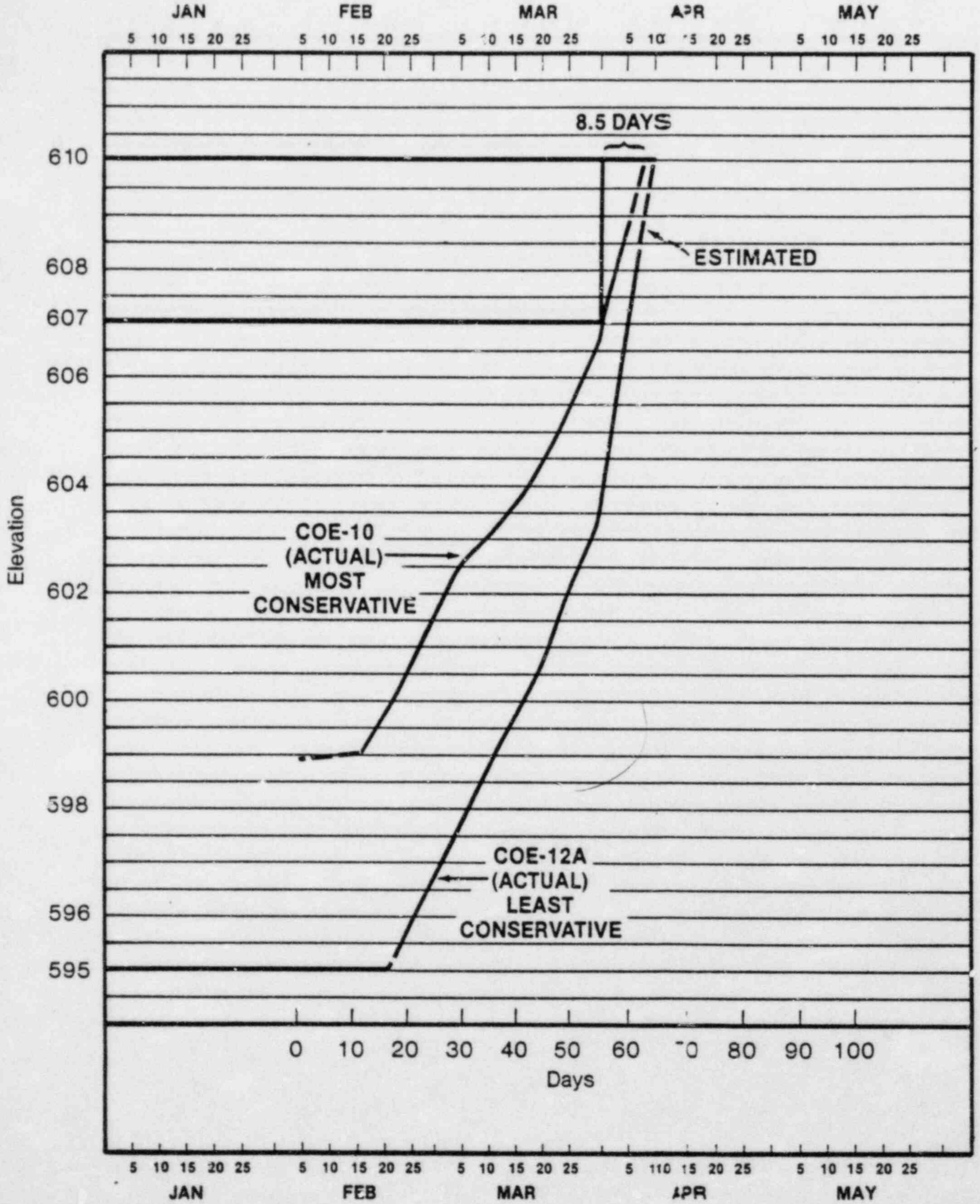
G-1986-24



CONSUMERS POWER COMPANY
MIDLAND UNITS 1 AND 2

GROUNDWATER LEVELS MEASURED AT CRITICAL AREAS DURING RECHARGE TEST

FIGURE V-4



MIDLAND UNITS 1 AND 2
ACRS 4/7/82

ASSUMPTIONS:

1. 1½ DAYS TO COLD SHUTDOWN
2. 7 DAYS TO OPERATE DIESELS AFTER COLD SHUTDOWN
3. WELL OR WELLS CANNOT BE REPAIRED OR REPLACED IN SUFFICIENT TIME

CRITERIA:

IF GROUND WATER LEVEL EXCEEDS ELEVATION 607.0 AT ANY OBSERVATION WELL AT THE DIESEL BUILDING OR AUXILIARY BUILDING TRAIN BAY THE PLANT WILL BE SHUT DOWN.

NOTE: FOR LOCATION OF OBSERVATION WELLS SEE FIGURE VI-3.

CONSUMERS POWER COMPANY MIDLAND UNITS 1 AND 2
DEWATERING CRITERIA FOR PLANT SHUTDOWN
FIGURE V-5

G-1986-15

PART VI: UNDERGROUND UTILITIES

1.0 INTRODUCTION

The Applicant has conducted an investigation to evaluate the adequacy of underground Seismic Category I utilities. The underground utilities included are:

a. Diesel fuel oil piping and tanks - This system provides fuel supply and return between the emergency diesel generators and diesel fuel oil storage tanks buried in the vicinity of the diesel generator building (DGB). There are four 1-1/2-inch supply lines, four 2-inch return lines, and four tanks 12 feet in diameter and 44 feet long.

b. Borated water piping - This piping provides borated water for volume and reactivity control from the borated water storage tanks (BWSTs) for normal functions and for such postulated accidents as a pipe break in the reactor coolant system. There are four 18-inch lines.

c. Control room pressurization piping and tanks - This system supplies overpressurization air to the main control room during postulated accidents such as releases of hazardous gases. There is one 4-inch line, one 1-inch line, and two tanks, each 5 feet in diameter and 25 feet long, buried in the vicinity of the auxiliary building.

d. Electrical duct banks - These concrete duct banks encase electrical power and control cables for various systems needed under normal and accident conditions.

e. Service water piping - This piping supplies water to various systems needed under normal and accident conditions. There are 22 lines ranging from 8 to 48 inches in diameter.

Table VI-1 contains a detailed listing of the Seismic Category I piping. Figure VI-1 shows the locations of the buried piping and tanks.

Because of the location of these utilities and the depth at which they are buried, all pipes, associated tanks, and duct banks listed above rest on compacted backfill material.

The investigation included test borings, measurements, and analysis. The remedial plan resulting from these investigations ranges from acceptance of the existing utilities to selected replacement. A selective monitoring program has also been adopted to ensure that intended functions are maintained over the life of the Midland plant.

2.0 REMEDIAL PLAN

The remedial plan for the Seismic Category I underground utilities is summarized below:

a. Diesel fuel oil lines and tanks - As a result of piping flexibility and small expected settlements for the piping and tanks, no remedial measures are indicated.

b. Borated water lines - This piping will be partially rebedded. This action, in conjunction with the settlement monitoring of the BWSTs, will provide assurance of the piping's continued serviceability.

c. Service water piping - Extensive measurements have been taken to define the present condition of the service water piping. A monitoring program for strain measurement and settlement will provide assurance of continued serviceability for a majority of the piping. The 36-inch diameter piping will be replaced. The two 26-inch diameter pipelines adjacent to the circulating water intake structure (CWIS) will be rebedded and the material beneath them replaced to preclude the potential for soil liquefaction.

d. Control room pressurization piping and tanks - The predicted differential settlement effects have been included in the design. No further action is required.

e. Electrical duct banks - The predicted settlement will not adversely impact the ability of the electrical duct banks to perform their function.

Details of the investigation, analysis, and agreements that support this remedial plan are presented in the remaining sections.

3.0 GEOTECHNICAL INVESTIGATIONS AND RESULTS

3.1 RESULTS OF TEST BORINGS

The records of exploration borings throughout the site indicate that the consistency of the fill at the location of buried utilities varies from soft to hard for silty clays and loose to dense for sands. Generally, the fill soils can be classified as medium stiff or medium dense below invert elevations of buried piping and other utilities. Fill foundation conditions have been greatly improved in the vicinity of the DGB as a result of the surcharge loading program that was conducted in 1979. Exploration borings in the area of the BWST indicate that the fill soils generally range from stiff to very stiff.

3.2 SETTLEMENT

Settlements that have been observed at buried utilities are primarily a result of the fill settling under its own weight. Areas that have been subjected to surcharge loading, such as the DGB and BWST areas, exhibit additional settlement from surcharging. The buried utilities add little, if any, weight to the fill; therefore, they have very little impact on present and future settlement below their invert elevations.

Records of monitored settlement within the fill have been utilized to predict future settlement for buried utilities. Borros anchors have been installed at nine locations in the vicinity of buried utilities not influenced by surcharge loadings. Settlement readings for anchors that have been established at depths of 7 feet to 12 feet below the surface were used in the analysis, because this depth represents the depth of most buried utilities. Soil conditions at these locations represent the variable soil conditions encountered throughout the fill.

Based on these records, future maximum settlement of buried utilities is conservatively estimated to be 3 inches or less. This maximum settlement estimate also includes future predicted settlement resulting from site dewatering and possible seismic shakedown. Future settlement of buried utilities in the vicinity of the DGB and BWST will be considerably less than the maximum value predicted because better fill conditions exist in these areas. Future settlement of the service water lines to be reinstalled in the vicinity of the service water pump structure (SWPS) and CWIS will be approximately 1-1/2 inches or less.

4.0 ANALYSES OF EXISTING UTILITIES

The analyses for buried utilities because of the remedial soils activities were initially presented in a technical report submitted December 15, 1981. They were discussed in meetings held with the staff in Bethesda, Maryland, on October 6, 1981; January 21 and 22, 1982; February 11, 1982; and were addressed in testimony at the Atomic Safety and Licensing Board (ASLB) soils hearings February 18 and 19, 1982. The following paragraphs summarize those reports, discussions, and testimony.

4.1 DIESEL FUEL PIPING AND STORAGE TANKS

The diesel fuel oil lines were installed in June 1980 after completion of the DGB surcharge program. The small diameter lines are flexible enough to accept the predicted future plant fill settlement without exceeding allowable limits. The maximum settlement stress was calculated for the maximum predicted settlement and was found to be within the allowable value.

The diesel fuel oil storage tanks were installed approximately 2 years after the fill was placed. This isolated the tanks from the effects of the initial settlement of the fill. The tanks were filled with water and the settlement monitored for approximately 8 months. Tank settlement during this period was minimal (less than 0.2 inch). It has been estimated that during plant life the tanks will experience about 1-1/4-inch long-term settlement, which includes settlement from site dewatering and seismic shakedown. The buried tanks will settle with the surrounding soil. The connecting pipes will also settle with the tanks in the surrounding soil. Thus, the differential settlement between the pipes and tanks will be small. Nozzle loads due to settlement have been calculated and are insignificant.

4.2 BORATED WATER PIPING

The borated water lines will be rebedded from the BWST valve pits to the dike around the tanks (see Figure VI-1). These lines have been cut loose from the valve pits to isolate them from the settlement caused by the valve pit surcharge. This partial rebedding in conjunction with the existing program to monitor future settlement of the BWST, settlement of the auxiliary building, and strain at the pipe anchors will provide sufficient ensurance of the piping's continued serviceability.

4.3 CONTROL ROOM PRESSURIZATION LINES AND TANKS

The control room pressurization lines and tanks were installed in early 1981. Installation after the occurrence of major fill settlement provides sufficient ensurance of continued serviceability of the pipes and tanks in this system.

4.4 ELECTRICAL DUCT BANKS

The seismic analysis of buried electrical duct banks complies with the requirements in FSAR Subsection 3.7.3.12 and was discussed in detail in the response to Question 30 of NRC Requests Regarding Plant Fill.

4.5 SERVICE WATER PIPING

4.5.1 Locations and Alignment

Extensive measurement data have been taken to define the present condition of the service water piping. The original position immediately after installation is not clearly defined. It is difficult to ascertain precisely how much of the current profile resulted from construction tolerances. To ensure serviceability, it has been conservatively assumed that all deviations from design location are due to settlement.

In 1979, elevation or profile data were taken for one pipeline in each pipe trench. In June 1981, the Applicant retained Southwest Research Institute to develop a more accurate measurement technique and to reprofile all the service water piping that is 26 inches and larger in diameter using the new technique. The measurement technique uses pressure and ultrasonic transducers and is accurate to 1/16 inch. The current location of the piping is very well defined from these accurately measured profile data taken at 5-foot intervals along the pipe length. The circumferential weld joints have also been identified between pipe spool lengths.

The results of these measurements show that the service water pipe is 8 to 12 inches from the design elevation in some extreme locations and the majority of the piping is, on the average, approximately 5 inches from its design location.

4.5.2 Ovalization

For the service water piping, the relationship between out-of-roundness/ovalization and strain was used to establish its serviceability. Ovalization is an indirect measurement of the bending stress of the pipe, which may have occurred due to fill settlement. These ovalization measurements were taken internally at the same locations as the profile points.

The results indicate general ovalizations of 1 to 1.5% with some locations of 2% and greater. The maximum ovalization recorded was 3% in one 36-inch diameter pipe where the pipe enters the SWPS.

4.5.3 Terminal End Analysis

A terminal end analysis considering weight, operating, and seismic forces was performed. This analysis started inside the structure at a fixed point (equipment nozzle or anchor) and continued to an assumed anchor point outside the structure. Soil springs were added along the pipe to model soil interaction. An analysis has also been performed to verify that displacements from settlement and seismic motion will not cause pipe contact with the building wall.

4.5.4 Acceptance Criteria

4.5.4.1 American Society of Mechanical Engineers (ASME) Code

The acceptance criteria for those portions of the analyses addressed by the ASME code were easily determined. These acceptance criteria are listed below:

a. Allowable stress in the pipe - Subsection NC

b. Combination of seismic stresses with stresses from other loading conditions - Subsection ND

c. Allowable stresses for the materials and operating temperature relevant to the piping being analyzed - Subsection ND

d. Allowable stress in pipe supports - Subsection NF

4.5.4.2 Ovalization

An acceptance criterion of 4% ovality for 26-inch pipe has been agreed upon with the NRC staff.

No agreement was reached between the Applicant and the NRC staff on appropriate acceptance criteria for the existing 36-inch diameter buried service water piping. Therefore, during the ASLB soils hearings, the Applicant agreed to replace the 36-inch pipe.

On March 16, 1982, the Applicant submitted a technical report describing the monitoring program, which resulted from a series of discussions with the staff. The report presented the relationship between ovalization and longitudinal strain in the pipe. Figure VI-2 shows the relationship used to convert the historical measured ovality to strain for comparison to the acceptance criteria.

4.5.5 Vertical Settlement

The acceptance criteria for settlement markers are based on the conservative upper limit of 3 inches for maximum future settlement. The NRC staff will be notified if 75% of the 3-inch upper limit is reached, and the staff and the Applicant will evaluate the appropriate action to be taken.

4.5.6 Reinstallation Program

The Applicant's March 16, 1982, report includes a reinstallation program that describes the engineering and construction activities necessary to replace the 36-inch diameter pipes and rebed a portion of two 26-inch diameter lines (26"-0HBC-53 and 26"-0HBC-54) immediately adjacent to the CWIS.

Rebedding the 26-inch diameter piping is an additional commitment since the soils hearings, based on the recently evaluated results of the dewatering recharge test. The results indicate that the area immediately north of the SWPS and the CWIS has only 3 days following a dewatering system failure before the groundwater would reach the level for potential soil liquefaction during a seismic event. As a consequence, the fill in the affected area will be replaced down to el 610'. The area covers a zone where the 36-inch diameter piping is being replaced and also a zone where pipelines 26"-0HBC-53 and 26"-0HBC-54 are buried. The fill replacement with acceptable fill will eliminate the potential for liquefaction.

The reinstallation program identifies the structures, facilities, and utilities that may be affected by the reinstallation activities. The underground utilities that will be exposed during the excavation work will be supported and protected as necessary to preclude damage. The quality program requirements applying to the reinstallation work were also discussed.

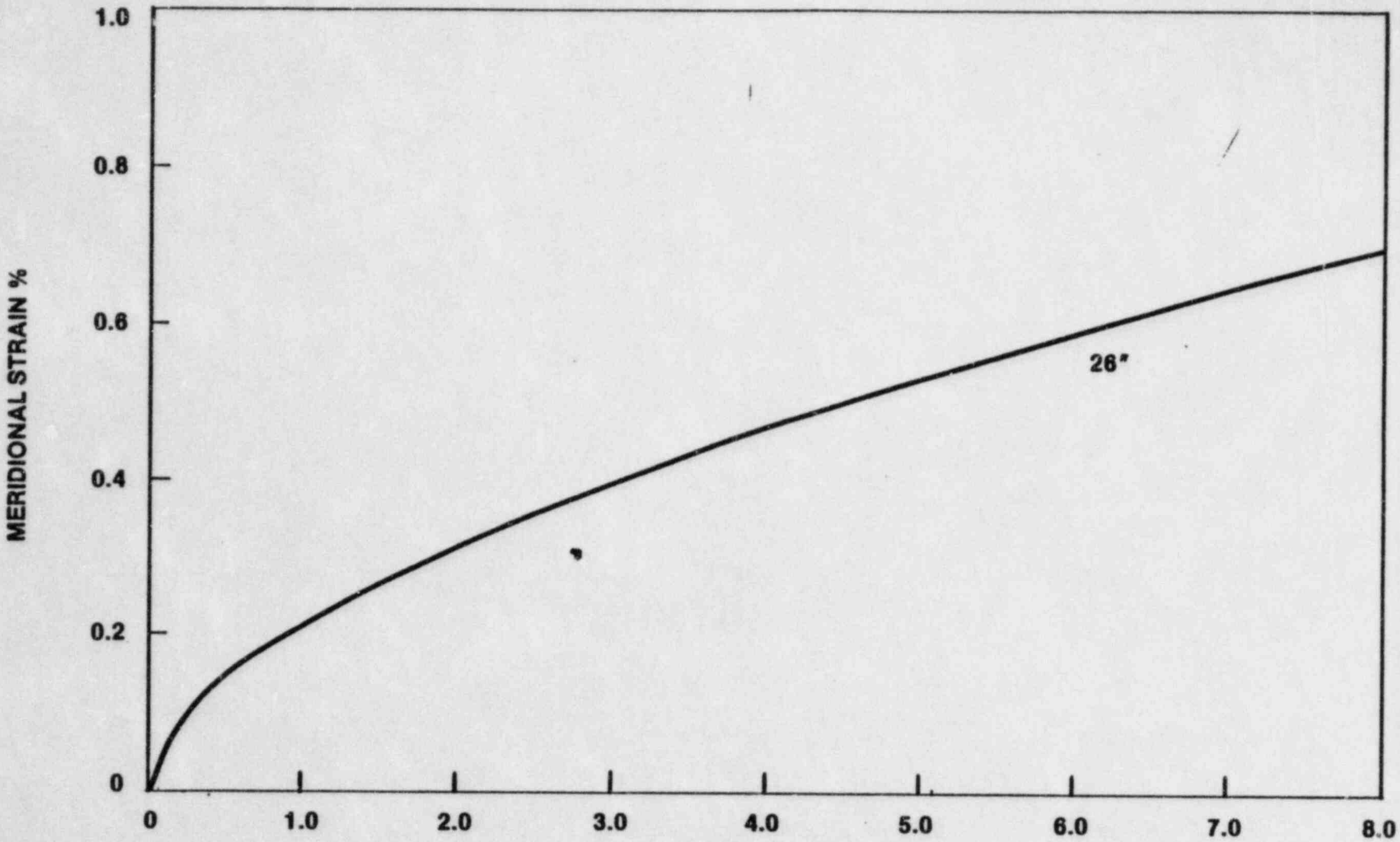
4.5.7 Monitoring Program

The future monitoring program submitted March 16, 1982, covers two types of monitoring: vertical settlement monitoring and pipe strain monitoring. The monitoring program describes the monitoring station locations and the details of selection criteria, monitoring frequency, acceptance criteria, and instrumentation for both types of monitoring. The reinstalled pipe will have no special monitoring program because the underlying fill will be replaced with suitable fill material.

The effect of future soil settlement on the service water piping will be monitored using externally mounted strain gages. The location of these instruments has been presented in the monitoring program submitted March 16, 1982. The location of these monitoring points are shown in Figure VI-1.

The initial monitoring frequency will be every 90 days, with reevaluation after 5 years. All locations are to be monitored immediately following an unusual event. If the technical specification limit is reached at a monitoring station, the frequency will be increased to monthly until remedial measures have been established.

The submittal of this monitoring program and the reinstallation program on March 16, 1982, provided the remedial action necessary to resolve the NRC concerns expressed in the ASLB soils hearing February 18 and 19, 1982.



$$\% \text{ OVALITY} = \frac{D_{MAX} - D_{MIN}}{D}$$

CONSUMERS POWER COMPANY MIDLAND UNITS 1 AND 2
MIDLAND STRAIN vs OVALITY CURVE
FIGURE VI-2

Part VII: QUALITY ASSURANCE

1.0 INTRODUCTION

All remedial soils work, except for underpinning, will be done in accordance with the existing Midland Project Quality Assurance Program. The underpinning activities are unique in that the few technically competent contractors who do this type of specialized work have no formal quality assurance (QA) programs and have little, if any, experience in the nuclear field. To accommodate the acquisition of only the most experienced contractors, a special Quality Assurance Plan for Underpinning has been devised to extend the Midland Project Quality Assurance Program to those contractors.

2.0 QUALITY ASSURANCE PLAN FOR UNDERPINNING

The Quality Assurance Plan for Underpinning, MPQP-1, was transmitted to the NRC on January 7, 1982. In addition to the information provided in the plan, in January and March there were presentations to and discussions with the NRC staff and Region III personnel relative to the plan. The plan has been found acceptable.

Under this plan, a special QA organization has been established for the underpinning work. The organization consists of two groups: a QA engineering group with an authorized staff of six engineers (degreed civil engineers), and an inspection, examination, and test verification group with an authorized staff of five civil inspectors (some of whom have experience directly related to the Midland underpinning work). These two groups report to a soils and remedial QA supervisor (a civil engineer) who, in turn, reports to the civil QA section head (also a civil engineer). Thus, there will be a total of 13 QA persons directly engaged in the underpinning work within the Midland Project Quality Assurance Department (MPQAD), which is independent of the architect-engineer/constructor and which is headed by a director reporting to the the Applicant's vice president for projects, engineering, and construction.

A special quality control (QC) organization also exists for which 23 inspectors are authorized for remedial soils inspection. The inspectors, through the lead inspectors, report to an underpinning QC coordinator who, in turn, reports to the lead civil QC engineer. This QC organization is part of the architect-engineer/constructor organization, but it is independent of the architect-engineer/constructor field construction management. Furthermore, this QC organization is overseen by the totally independent MPQAD described above.

The MPQAD performs the primary QA activities for the underpinning work, whereas the QC organization performs the primary inspection activities to the standards and requirements established by MPQAD. The following is a brief description of the major MPQAD activities and the objectives of each.

Design documents are originated and issued through the architect-engineer's design process with all controls of the existing Project Quality Assurance Program being applied to the design process. However, before their issuance, MPQAD reviews and approves the documents to ensure that they are sufficiently specific with regard to the quality characteristics and to ensure that these characteristics are inspectable or testable.

For construction contracts, MPQAD establishes the requirements by which the contractors attain quality, although the QC and MPQAD organizations will ensure that quality is attained. Requirements applied to contractors may deal with document controls, preparation of detailed construction procedures, personnel training, handling and storage of materials, and performing process corrective action, when necessary. These types of requirements are intended to promote the prevention of nonconformances or, at worst, their early detection and the correction of their root causes.

MPQAD reviews and approves construction procedures to ensure that the procedures impose the necessary quality prerequisites, that they provide sufficient specificity with which to ensure the consistent attainment of the design requirements, and that the QC inspection hold points are integrated into the construction procedures at the appropriate points in the process. MPQAD also integrates the MPQAD overinspection hold points into the construction procedures.

MPQAD reviews and approves the detailed QC inspection procedures to ensure that they are complete with regard to the necessary inspections and to ensure that they are sufficiently specific with regard to the methods of inspection, the points of inspection, and the inspection data to be recorded.

MPQAD plans and performs its own overinspections. These overinspections are on a large sampling basis and are applied to the most significant quality characteristics for the purpose of ensuring that the construction work is being done properly and ensuring that the QC inspection decisions are being made properly. On a periodic basis, quality system audits of the constructor and contractors are also performed by MPQAD to ensure compliance with the QA standards and requirements. In addition to MPQAD, an entirely separate Applicant audit section performs periodic system audits. MPQAD ensures the correction of nonconformances as well as the identification and elimination of their root causes.

3.0 QUALITY ASSURANCE COVERAGE

As a result of the March discussion with the NRC, it has been agreed that the Quality Assurance Plan for Underpinning will be implemented for essentially all elements of the underpinning work and not just for the specific activities or structures deemed to be safety related. The plan is being modified to reflect this additional coverage. A mechanism will be provided by which to take any exception which may be desired, but this mechanism will include assurance that Region III personnel have concurred with the exception prior to doing the work. The MPQAD and QC staffing levels described above were arrived at in recognition of this extended coverage. The NRC has concurred that the staffing levels to date have been appropriate to the level of work.

PRINCIPAL STAFF			
RA		ENF	
D/RA		SCS	
A/RA		PAO	
DPRP		SLO	
DRMA			
DRMSP			
DE			
ML			
OL		FILE	

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EDWARD S. ISHAM. 1872-1902
ROBERT T. LINCOLN. 1872-1889
WILLIAM G. BEALE. 1885-1923

Docketed USNRC JUL 25 1983 Office of the Sec. Docketing & Serv. Branch

July 18, 1983

Mr. Michael N. Wilcove
U. S. Nuclear Regulatory Commission
Washington, D. C. 20555

Re: Consumers Power Company - Midland Plant Docket Nos. 50-329-OM; 50-330-OM; 50-329-OL; and 50-330-OL.

Dear Mike:

Enclosed in accordance with our telephone conversation are the July, 1983 crack maps of the Midland containments at the buttress-base slab junctions. Actually they are really more sketches than maps. My understanding is that "H.L." stands for "Hairline" and means that the crack width is less than .005 inches. Only five of the six junctions are included; the sixth area is the Room 110 area which was mapped on Field Engineer's Report Form CC-183 on December 17, 1982. Dr. Shunmugavel visited the site last week and found that the cracks in Room 110 have not changed. Apparently for that reason the Room 110 area was not remapped.

We are filing affidavits today from Dr. Corley and Dr. Shunmugavel in response to Ms. Stamiris' Motion to Reopen the Record. Both Dr. Corley and Dr. Shunmugavel refer to these July crack maps. I decided not to include them in our response since they don't seem to add much. However, we are sending these to you and the other parties for whatever use you want to make of them.

Sincerely,

Philip P. Steptoe

PPS:es

enc.

cc Service List (w/enclosures)

AUG 1 1983

DS03

Please Resubmit Corrected Document in Its Entirety Within NA Weeks

AAO-1040 7220

G-321-D

Category NA

AAO-1041 7220

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VENDOR	2
CLIENT	1
FIELD EQ	2
ARCH.	1
CIVIL/SOIL	1
ELECT	
P/D	
MECH	
C SYS	
INSP	
STARTUP	
RSE	1
RECORD	1
BECHTEL	

7-14-83

BECHTEL ASSOC. PROFESSIONAL CORP. — JOB NO. 7220-001

SUPPLIER DOCUMENT STAMP

1. ☐ WORK MAY PROCEED.
2. ☐ WORK MAY PROCEED. SUBMIT FINAL DOCUMENT.
3. ☐ REVISE AND RESUBMIT. WORK MAY PROCEED SUBJECT TO INCORPORATION OF CHANGES INDICATED.
4. ☐ REVISE AND RESUBMIT. WORK MAY NOT PROCEED.
5. ☐ REVIEW NOT REQUIRED. WORK MAY PROCEED.

☒ FOR INFORMATION ONLY.

PERMISSION TO PROCEED DOES NOT CONSTITUTE ACCEPTANCE OR APPROVAL OF DESIGN DETAILS, CALCULATIONS, ANALYSES, TEST METHODS OR MATERIALS DEVELOPED OR SELECTED BY THE SUPPLIER AND DOES NOT RELIEVE SUPPLIER FROM FULL COMPLIANCE WITH CONTRACTUAL OBLIGATIONS.

REVIEWED	A	C	E	J	M	N	P	Q
	/	DS	/	/	/	/	/	/

BY W. Ahiqmataly for B. Dhar   Date 7-14-83

7220-200 REV 9/82

DWG REV/SEQ NO. 7220-C-198-2857-1
STARTUP SYSTEMS OH AREAS ≤ 90 DAYS OF TURNOVER, OR AFTER TURNOVER IF AFFECTED BY CHANGES:

System/ Area No.	Hardware Change (Yes or No)
NA	

7220-242

7220-C-198-2857-1

TOTAL # OF PAGES -11

SCALE: ONE [illegible] EQUALS ONE FOOT

AUXILIARY BUILDING

DATE 7/12/83

ELEVATION 591'6"

COMPARATOR NO. C1-1

CALIBRATION DUE DATE N/A

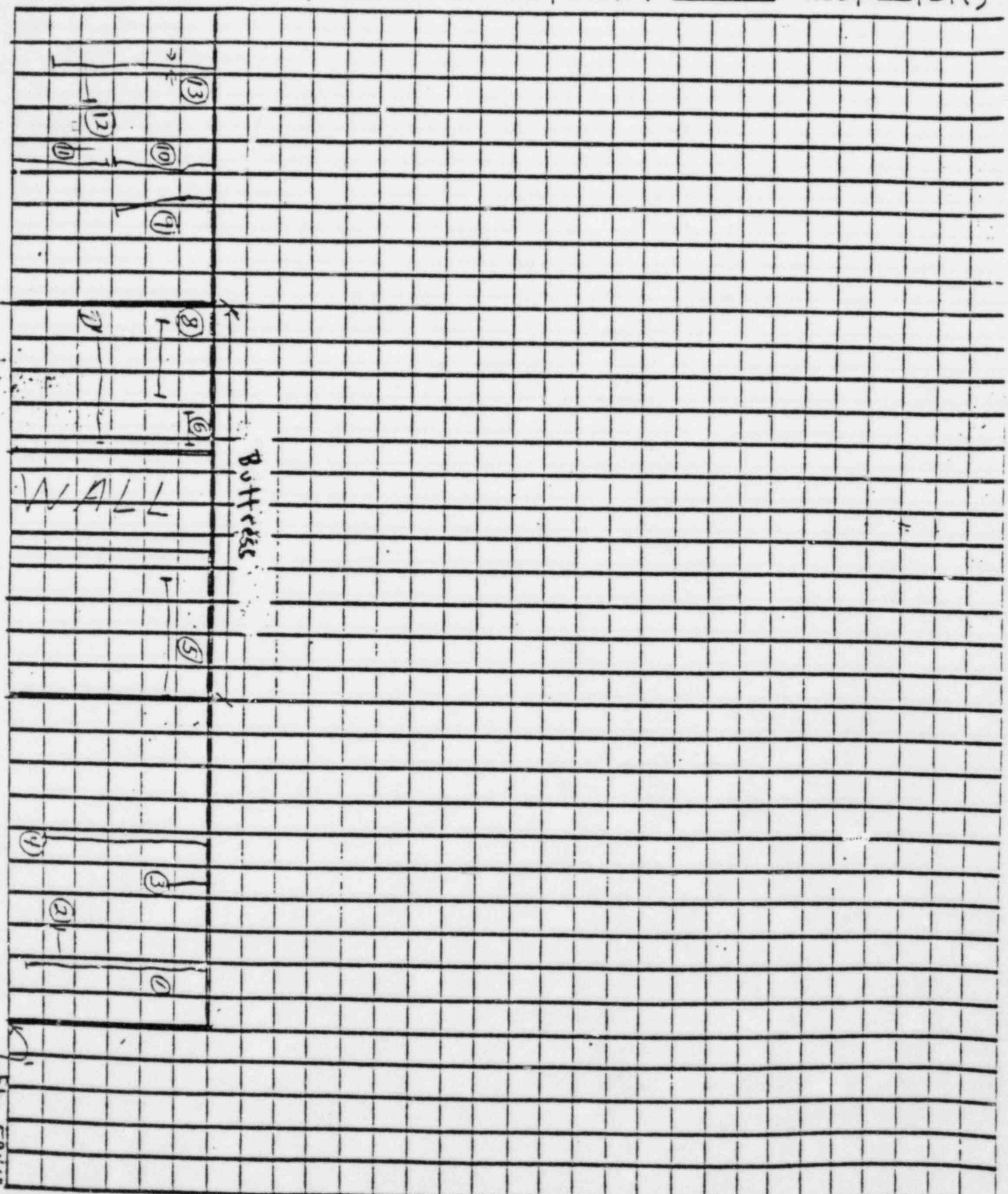
LOCATION Unit #2 East

SEQUENCE NO. 1

SURVEYED BY [illegible]

REVIEWED BY [illegible]

WJE (LEVEL II INSPECTOR)



7220-C-198-2857-1

TAB[illegible] 43T

MEASURED CRACK WIDTH SUMMARY

591'6" Unit #2, East, 1"

CRACK NO.	DATE C-1-1 7/12/83											
1	H.L.											
2	H.L.											
3	H.L.											
4	H.L.											
5	H.L.											
6	H.L.											
7	H.L.											
8	H.L.											
9	H.L.											
10	H.L.											
11	H.L.											
12	H.L.											
13	.005											

7220-C-198-2857-1

SCALE: ONE ▪ EQUALS ONE FOOT

AUXILIARY BUILDING

DATE 7/11/83

ELEVATION 591' 6"

COMPARATOR NO. C1-1

CALIBRATION DUE DATE N/A

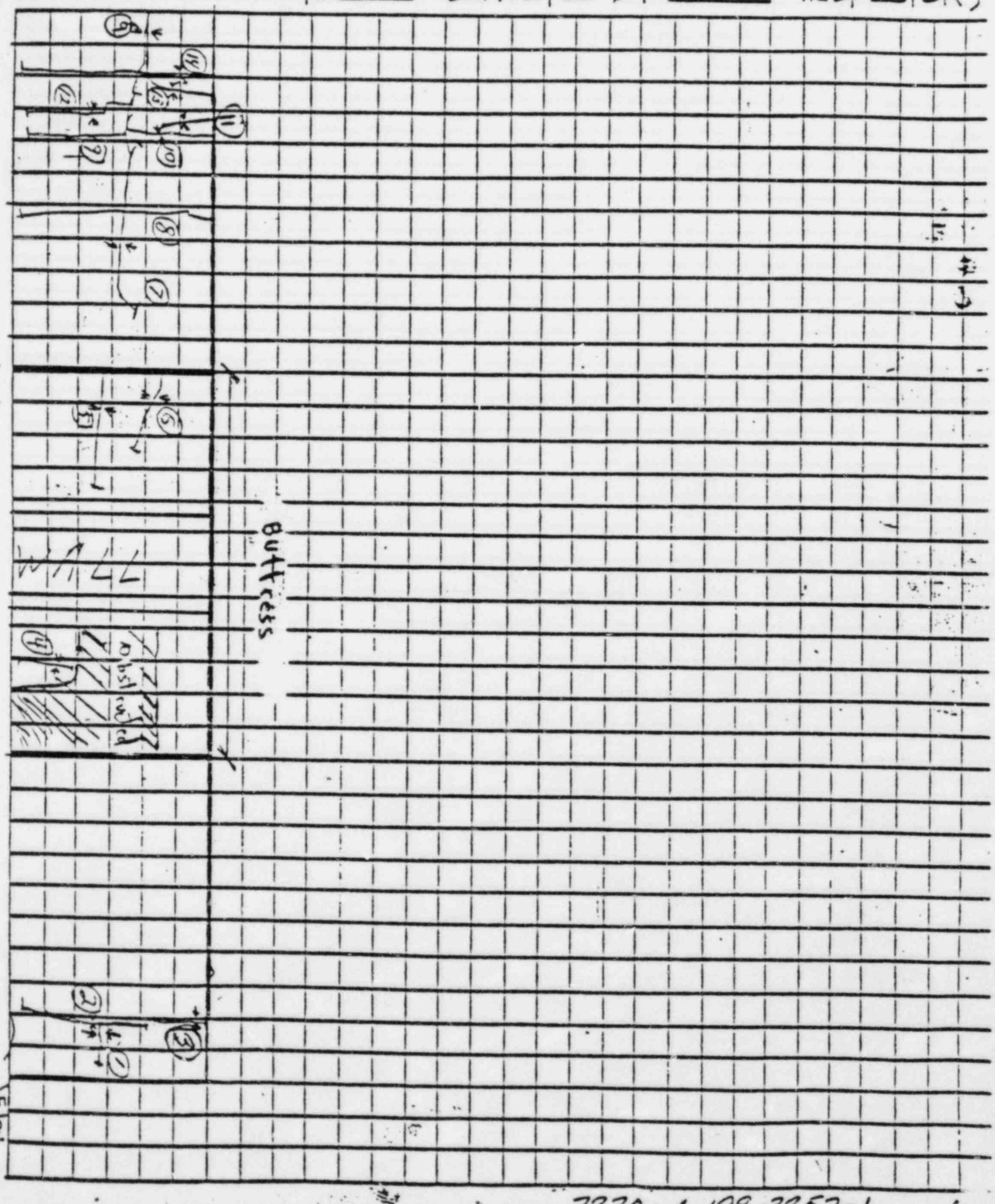
LOCATION Unit #1 West

SEQUENCE NO. 1

SURVEYED BY SK

REVIEWED BY [illegible signature]

NJE (LEVEL II INSPECTOR)



7220 · C-198 · 2857-1

A

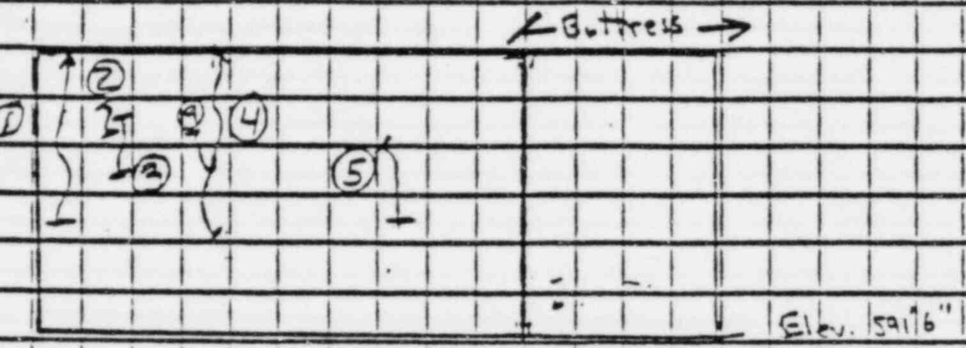
TABLE 43T

MEASURED CRACK WIDTH SUMMARY

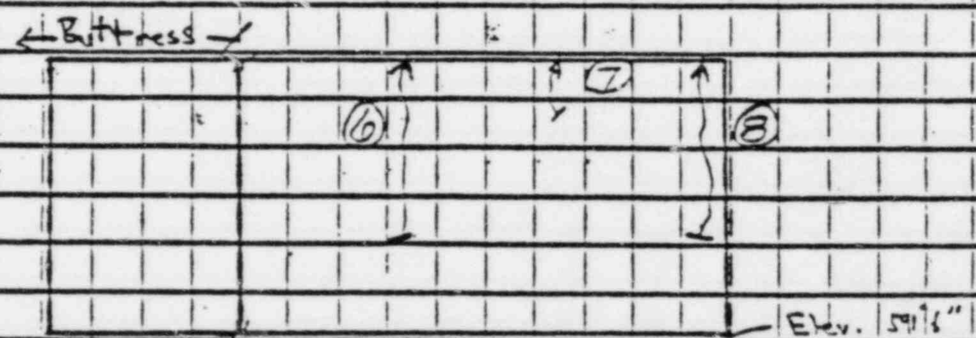
591'6" Unit #1, West
'1"

CRACK NO.	DATE L1-1 7/11/83											
1	H.L.											
2	.005											
3	H.L.											
4	H.L.											
5	H.L.											
6	H.L.											
7	H.L.											
8	H.L.											
9	.005											
10	H.L.											
11	.005											
12	H.L.											
13	H.L.											
14	H.L.											

SCALE: ONE ▦ EQUALS ONE FOOT	AUXILIARY BUILDING	DATE 7-12-83
ELEVATION 591'-6"	LOCATION UNIT 2 NORTH	REVIEWED BY M Coma
COMPARATOR NO. C1-1	SEQUENCE NO. 2	WJE (LEVEL I
CALIBRATION DUE DATE NA	SURVEYED BY [illegible]	INSPECTOR)



WEST SIDE



EAST SIDE

TABLE 43T

MEASURED CRACK WIDTH SUMMARY

59' 1" UNIT 2 NORTH

CRACK NO.	DATE											
	4-1 7/12/83											
1	HL											
2	HL											
3	HL											
4	HL											
5	HL											
6	HL											
7	HL											
8	HL											
9	HL											

7220-C-198-2857-1

EQUALS ONE FOOT

ELEVATION 591'-6"

COMPARATOR NO. C1-1

CALIBRATION DUE DATE N/A

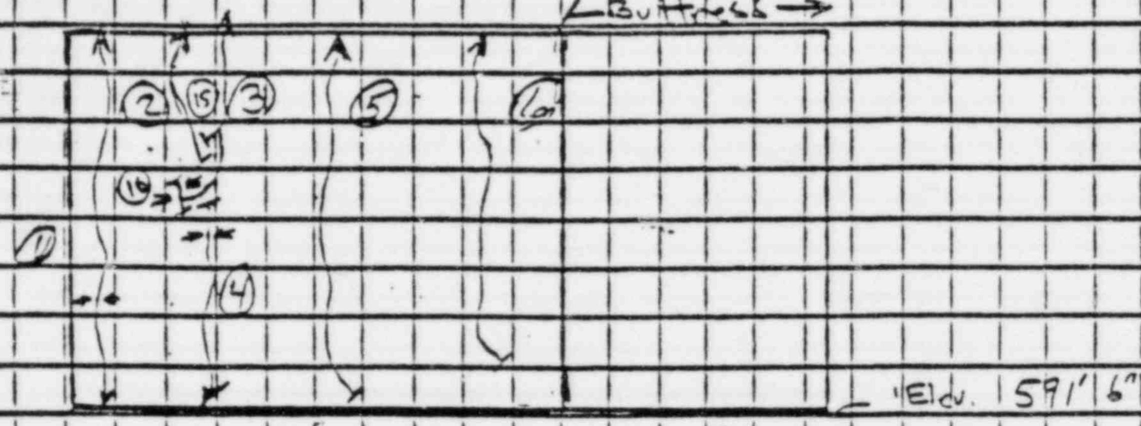
LOCATION UNIT 1 NORTH

SEQUENCE NO. 1

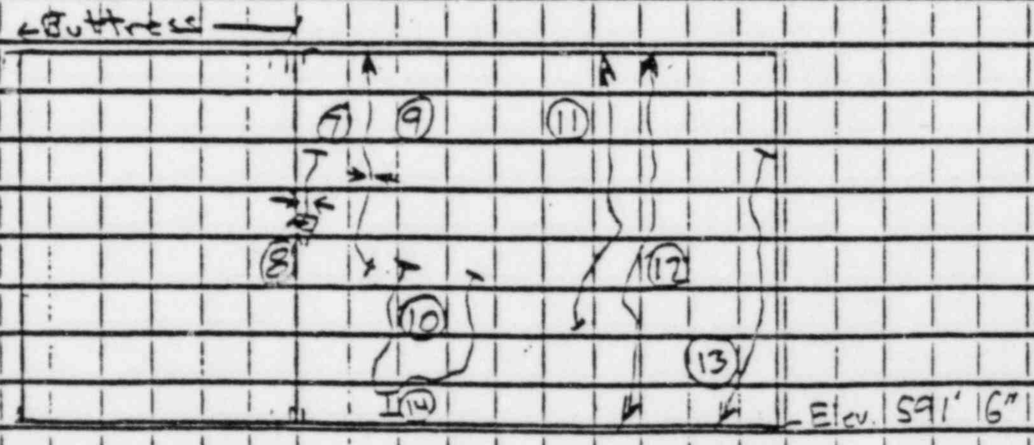
SURVEYED BY [illegible]

REVIEWED BY [illegible]

WJE (LEVEL I INSPECTOR)



WEST SIDE



EAST SIDE

7220-C-193-2857-1 \ 8

TAB... 43T

MEASURED CRACK WIDTH SUMMARY

591' 6" · UNIT 1 NORTH

CRACK NO.	DATE C1-1 7/12/83											
1	.015											
2	HL											
3	HL											
4	.010											
5	HL											
6	HL											
7	.010											
8	.010											
9	.010											
10	HL											
11	HL											
12	HL											
13	HL											
14	HL											
15	HL											
1[illegible]	.010											

SCALE: ONE [illegible] EQUALS ONE FOOT

AUXILIARY BUILDING

DATE 7-13-

ELEVATION 591'-6

COMPARATOR NO. C2-1

CALIBRATION DUE DATE NA

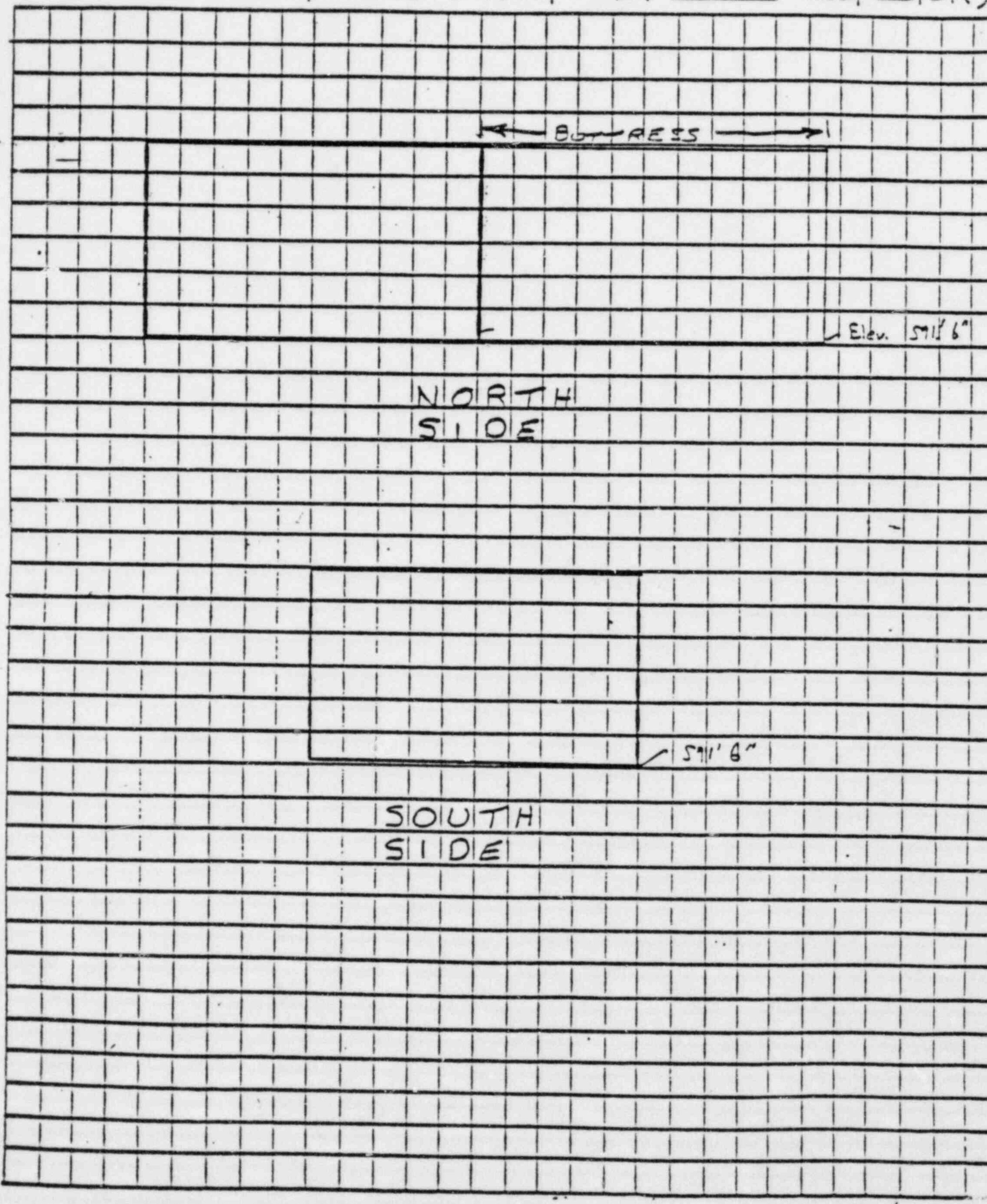
LOCATION Unit 2 West

SEQUENCE NO. 1

SURVEYED BY [illegible]

REVIEWED BY [illegible]

WJE (LEVEL INSPECTOR)



7220-C-198-2857-1 1

TAB. 43T

MEASURED CRACK WIDTH SUMMARY 59'6" Unit #2, West 1"

CRACK NO.	DATE											
	7/15/83											
	No cracks											

RFW

DEC 0 6 1983

Docket No. 50-329
Docket No. 50-330

Consumers Power Company
Midland Nuclear Plant
ATTN: Mr. D. L. Quamme
Site Manager
P. O. Box 1963
Midland, MI 48640

SUBJECT: REMEDIAL SOILS WORK ACTIVITIES

Dear Mr. Quamme:

This letter documents four verbal agreements made between members of the NRC staff and the licensee's staff during three telephone calls on September 22, September 27, November 28, and during the Stone & Webster public meeting on November 10, 1983.

The licensee agreed not to release the soils stop work (imposed because of drawing deficiencies) until the crack monitoring program deficiencies are resolved and reviewed by the NRC staff.

The licensee agreed not to jack the grillage assemblies at Pier 8 to 160% of the design load as proposed, until the NRC staff can review the building analysis for this new loading condition.

Subsequent to the above request, the NRC staff agreed to allow the licensee to increase the jacking load on the grillage assemblies at Pier 8 from 125% to 135% as required in the SSER #2 in order to reduce future building elevation losses.

The licensee agreed not to continue the upward jacking of the auxiliary building beyond those limits previously specified in the SSER #2 Supplement pending the establishment of new allowable jacking limits with the NRC staff.

Should you have any questions concerning the above agreements, please feel free to contact me.

Sincerely,

Original signed by J. J. Harrison

J. J. Harrison, Chief
Section 2, Midland

~~8312120147~~

cc: See attached distribution list

RIII	RIII	RIII
RBL	RNG	JJH
Landsman/ls	Gardner	Harrison
12/06/83		12/6/83

cc:
DMB/Document Control Desk (RIDS)
Resident Inspector, RIII
The Honorable Charles Bechhoefer, ASLB
The Honorable Jerry Harbour, ASLB
The Honorable Frederick P. Cowan, ASLB
The Honorable Ralph S. Decker, ASLB
William Paton, ELD
Michael Miller
Ronald Callen, Michigan
  Public Service Commission
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Barbara Stamiris
Mary Sinclair
Wendell Marshall
Colonel Steve J. Gadler (P.E.)
Howard Levin (TERA)
Billie P. Garde, Government
  Accountability Project
Lynne Bernabei, Government
  Accountability Project
Stone and Webster Michigan, Inc.

RELIMINARY NOTIFICATION OF EVENT OR UNUSUAL OCCURRENCE--PNO-III-83-51 Date: June 21, 1983

his preliminary notification constitutes EARLY notice of events of POSSIBLE safety or ublic interest significance. The information is as initially received without verification or evaluation, and is basically all that is known by the staff on this date.

acility: Consumers Power Company
Midland Nuclear Power Plant
Docket No: 50-329, 50-330
Midland, MI 48640

Licensee Emergency Classification:
____ Notification of Unusual Event
____ Alert
____ Site Area Emergency
____ General Emergency
xx Not Applicable

ubject: AUTHORIZATION OF FIRST MAJOR UNDERPINNING WORK UNDER SAFETY-RELATED BUILDING

Region III (Chicago) has authorized Consumers Power Company to proceed with excavation under the auxiliary building (electrical penetration area) and installation of temporary support beams which will support the electrical penetration areas during further underpinning activities.

The work authorized is the first major activity under a safety-related structure, the auxiliary building, in the licensee's remedial soils work (underpinning). Previous work authorized has included construction of twelve support piers under the turbine building.

The new work involves tunneling under the electrical penetration areas to the foundat of the two reactor containments. The support beams -- each about twenty feet long -- will be installed between one set of piers beneath the turbine building and the edge of reactor containment base mat (foundation). Jacks will then be installed to support the electrical penetration area while a new foundation wall is built but this additional work has not yet been authorized. The new work authorization will result in the recall of workers laid off June 17 and 18 (see PNO-111-83-49, dated June 17, 1983).

The licensee has informed local news media of the work authorization, and Region III has been responding to inquiries.

The State of Michigan will be notified.

Region III informed Consumers Power of the authorization on June 21, 1983. This information is current as of 10 a.m. (CDT), June 21, 1983.

R Landsman J. Harrison

Contact: R. Landsman J. Harrison
FTS 384-2587 384-2635

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~~8306270181~~

Applicable Resident Site 1235

Regions I 1208, II 1210, IV 1214, V 1216 Licensee (Corporate Office) 1241

Rev. 1/19/83

TASK NO. 83 - 40 RFW
DATE: APRIL 18 1983
TAC #: 51341

TASK INTERFACE AGREEMENT

PRINCIPAL STAFF

PROBLEM: Midland 1/2 - Soils Issue

LEAD OFFICE: /_/ I&E /_/ NRR /X/ REGION III /_/ JOINT

NOTIFICATION:

REFERENCES: Memo to TNovak fm RWarnick dated 03/16/83, subject: NRR Assistance in Resolving Midland Soils Issue

ACTION PLAN:

NRR: 1. Assist Region III in reviewing the remedial soils work at Midland. Assistance is expected to include evaluation of possible deviations from licensee commitments in the SER, advice to the Region III reviewer, and occasional site visits. (SGTEB)

The exact schedule cannot be defined but the PM forcasts that NRR assistance after ~~12/83~~ 6/84 is unlikely.

Region III will contact NRR (PM) on case basis.

NRR: Designate Lead Project Manager to assign TACS and coordinate correspondence, meetings, and reports (ORB# /LB#4 - D. Hood).

OFFICE COORDINATORS:

RHW T. Ippolito (X27415)

R. Vollmer 3/31/83 (X27207)

T. Novak 4/5 (X27425)

(X)

(X)

APPROVED:

N.A./RHW C. Norelius I&E

Region- III

W. J. ... for F. J. Miraglia 4/15 (X27492)

NRR

cc: V. Stello, ROGR
Regional Admin.
J. Taylor, I&E
E. Jordan, I&E
R. Baer, I&E
W. Mills, I&E

J. Sniezek, I&E
R. DeYoung, I&E
J. Heltemes, AEOD
H. Denton, NRR
E. Case, NRR
R. Mattson, NRR
H. Thompson, NRR

T. Speis, , NRR
D. Eisenhut, NRR
R. Vollmer, NRR
G. Lainas, NRR
T. Novak, NRR
F. Miraglia, NRR
F. Rinaldi, NRR

G. Holahan, NRR
Lead Project Manager
R. Purple, NRR
R. Wessman, NRR
J. Knight, NRR
G. Lear, NRR
J. Kane, NRR

APR 25 1983

UNITED STATES
NUCLEAR REGULATORY COMMISSION
REGION III
799 ROOSEVELT ROAD
GLEN ELLYN, ILLINOIS 60137

MAR 1 6

MEMORANDUM FOR: T. Novak, Assistant Director for Licensing, Division of Licensing

FROM: R. F. Warnick, Director, Office of Special Cases

SUBJECT: NRR ASSISTANCE IN RESOLVING MIDLAND SOILS ISSUE

Region III has assumed all responsibility for reviewing the remedial soils work at the Midland site. However, we expect the licensee to periodically request relief from commitments made in the SSER. NRR's assistance will be requested when this occurs.

The expertise of NRR will also be required from time to time for consultation with Mr. Ross Landsman during his review of the remedial soils activities. A schedule cannot be defined at this time. NRR's assistance will be requested on a case by case basis as the need arises.

We also recommend that periodic site visits be made in order for your personnel to maintain their awareness of the underpinning effort. These visits could be limited to observations of critical work activities such as the pier 11 load tests and the drift work to the control tower. The schedule for these activities can be obtained from Ross Landsman. L/RIII

Should you have any questions please contact Wayne Shafer (FTS 384-2656).

RF Warnick

R. F. Warnick, Director
Office of Special Cases

cc: A. B. Davis
J. H. Sniezek, IE
J. C. Stone, IE
D. Hood, NRR M/S-116

~~8312140044~~

UNITED STATES
NUCLEAR REGULATORY COMMISSION
REGION III
799 ROOSEVELT ROAD
GLEN ELLYN, ILLINOIS 60137

D. Hood

MAR 1 6 1983

MEMORANDUM FOR: T. Novak, Assistant Director for Licensing, Division of Licensing

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Should you have any questions please contact Wayne Shafer (FTS 384-2656).

RF Warnick
R. F. Warnick, Director
Office of Special Cases

cc: A. B. Davis
J. H. Sniezek, IE
J. C. Stone, IE
D. Hood, NRR

8312140044

Landsman

UNITED STATES
NUCLEAR REGULATORY COMMISSION
REGION III
799 ROOSEVELT ROAD
GLEN ELLYN, ILLINOIS 60137

MAR 16

MEMORANDUM FOR: T. Novak, Assistant Director for Licensing, Division of Licensing

FROM: **R. F. Warnick, Director, Office of Special Cases**

SUBJECT: **NRR ASSISTANCE IN RESOLVING MIDLAND SOILS ISSUE**

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Should you have any questions please contact Wayne Shafer (FTS 384-2656).

RF Warnick
R. F. Warnick, Director
Office of Special Cases

cc: A. B. Davis
J. H. Sniezek, IE
J. C. Stone, IE
D. Hood, NRR

8312140044

UNITED STATES
NUCLEAR REGULATORY COMMISSION
REGION III
799 ROOSEVELT ROAD
GLEN ELLYN, ILLINOIS 60137

April 9, 1982

MEMORANDUM FOR: Darrell Eisenhut, Director, Division of Licensing, NRR

FROM: R. L. Spessard, Director, Division of Project and Resident Programs

SUBJECT: RECOMMENDATION OF BOARD NOTIFICATION (MIDLAND)

Recognizing the Atomic Safety and Licensing Board's keen interest in matters impacting on Consumers Power Company's quality assurance activities in connection with Midland, Region III believes that the Board should be made aware of two issues being pursued by Region III relative to the remedial soils work. These are:

1. Members of the Region III staff who participated in recent meetings with the applicant concerning the application of quality assurance in the underpinning activities have expressed concern that information provided by the applicant's staff regarding the status of instrumentation work completion was misleading. While the technical issues related to this matter are being resolved to our satisfaction, I plan to initiate a more in-depth look into the concerns expressed by the NRC staff members.

2. The applicant is experiencing problems with QA program implementation as they restart work in the underpinning area. While our experience tells us that some problems will occur in the restart of any activity that has been suspended for a long period of time, we believe certain problems should not have occurred (e.g., performing work without adequate implementing procedures; selective application of QA program requirements to work activities). We are continuing to monitor closely the licensee's activities in this area. If we conclude that the program is not being well managed, we will not hesitate to stop the work and take appropriate enforcement action.

The results of our investigation into the possible misleading statements and any continuance of problems with the implementation of the quality assurance program will be brought to the Board's attention promptly.

R. L. Spessard

R. L. Spessard, Director
Division of Project and
Resident Programs

cc: V. Stello, DEDROGR
R. C. DeYoung, IE
H. R. Denton, NRR
J. G. Keppler, RIII
E. L. Jordan, IE
W. Paton, ELD
J. Lieberman, IE
R. G. Adensam, NRR
W. F. Kane, NRR
R. Cook, SRI, Midland

~~8204260326~~ 2pp.
820426

Board Notification 82-39 April 15, 1982

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V. Stello, EDO
E. Christenbury, OELD
J. Scinto, OELD

[redacted]

CC:
[illegible]
G[illegible]er
Landsman
RFile

To Bob Warnick - Region III

From Mike Wilcove [illegible]

This is the stop work order
[illegible] [illegible] [illegible] [illegible] [illegible]

4-Pages

Consumers Power Company

ORAL COMMUNICATIONS RECORD

PROJECTS, ENGINEERING AND CONSTRUCTION - QUALITY ASSURANCE DEPARTMENT

QA5-0

CENOR. FILE NO 0.4.9.20.6

PAGE 1 OF 1

DATE OF COMMUNICATION 5/19/82 QA-PLAC PERSONNEL PARTICIPATING RESevo, DEHorn

TIME OF COMMUNICATION 1:15 PM OTHER PARTY(S) BPeck, SMO Construction Superintendent

PREPARED BY RESevo RES.

SUBJECTS AND/OR STATEMENTS DISCUSSED Stop drilling direction to Kelly (well drilling) per Spec C-118

SUMMARY OF CONVERSATION At 1:30 PM, this date, Mr. Peck requested that MPQA stop drilling operations by Kelley Dewatering per Spec C-118 for Obs. Well 1A in addition to OBS Well-4 which was stopped at 10:30 AM due to void discovered during drilling by the cable tool method. Obs wells 1A and 4 are the last wells to be drilled under this contract. Resumption of drilling Obs-1A would be allowed pending determination of any utilities in the immediate vicinity and that source of void on Obs-4 does not affect drilling operations on Obs-1A.

NOTES:

1. NRC inspectors R. Cook and R. Landsman inspected Obs-4 and void at 1:00 PM during site tour.
2. NCR 4285 validated by Bechtel at 5:00 PM. Tag placed next day.
3. Drilling on well 1A was stopped at 2:40 PM this date by MPQAD.
4. QC activity hold order #4 authorized at 5:00 PM.
5. W. R. Bird and B. W. Marguglio appraised of situation at 4:00 PM and 4:15 respectively.

Q.5. Mr. Bird, can you please describe Bechtel NCR #4199 and #4245 (Landsman Attachments 7E and 7D respectively.)?

A.5. Bechtel nonconformance reports #4199 and #4245 cover damage to a "Q" deep electric duct bank and a void adjacent to the upper portion of the permanent observation well OBS #4.

The duct bank was damaged on April 24, 1982, during drilling of an ejector well for the freeze wall monitoring pit. This happened because the drilling rig was mispositioned by a couple of feet. The root cause of the nonconformance was that the procedural control to not drill closer than two feet to any known buried utility for vertical holes was not adequately implemented. When obstruction was encountered Field Engineering apparently believed that they were hitting a concrete overpour around the duct bank rather than the duct bank itself. They continued drilling until drilling fluid was lost. Subsequently the fluid was observed in the auxiliary building and it was ascertained that the drill had hit the edge of the duct bank. Consumers Power Company Site management stopped further addition of drilling fluid on April 28, 1982 as a result of their involvement in the immediate investigating of the drilling fluid found in the auxiliary building. The CPCo Site Manager issued a letter on April 28, confirming the verbal stop work directive applicable to all drilling operations and sheet-piling activities by Mergantime Corporation and its subcontractors, in all Q and non-Q areas. MPQAD management later issued a formal Quality Stop Work Order to provide tracking and close-out of the corrective action required to lift the stop work. The stop work order was subsequently lifted based on further training and implementation of the excavation permit procedure FIC 5.100 described below.

With respect to NCR #4245, the void associated with observation well #4 appears to be a localized phenomenon near the surface of the well, which was observed on May 11, 1982 during drilling. This void is apparently only indirectly related to another condition associated with OBS #4 observed at approximately the same time, that being the penetration of a twelve inch non-"Q" condensate drain line at a depth of 38 ft. The striking of the line and associated vibration may have contributed to the void formation. The remainder of the void is thought to be from material removal resulting from the drilling process. A final Engineering Report on this subject awaits completion of probing to determine the extent of soil disturbance. The specification for well drilling has been revised to restrict the position of the bailer in relation to the bottom of the casing which should limit excess soil removal for any future application of this drilling technique.

The review prior to drilling for utilities in the vicinity of OBS#4, missed the condensate line because the drawing showing this line was not on the list of drawings requiring review. The new excavation permit system (Attachment 1) has attached to it a listing of drawings, by discipline, which represents the most complete information available on all underground utilities at the Midland site.

Jwarnick
File
JFW

UNITED STATES
NUCLEAR REGULATORY COMMISSION
REGION III
799 ROOSEVELT ROAD
GLEN ELLYN, ILLINOIS 60137

MAR 3 0 1983

Docket No. 50-329
Docket No. 50-330

Michael I. Miller, Esq.
Isham, Lincoln & Beale
Three First National Plaza
Chicago, IL 60602

Dear Mr. Miller:

I am in receipt of your letter of March 22, 1983 (attached) stating Consumer Power Company's (CPCo) intention to resume discovery on Sinclair Contentions 1, 15, 16 and 17. That discovery commenced in the summer of 1982 through the Licensing Board's issuance at CPCo's request of subpoenas to three employees or associates of the Government Accountability Project (GAP). At my request, CPCo and the other parties to the Midland proceedings held in abeyance discovery on the "GAP allegations" and the Zack HVAC issue which underlie Ms. Sinclair's contentions to afford the NRC time to complete its investigatory and inspection efforts on these allegations. In light of the fact that the NRC's investigations and inspections on some of these matters may not be completed for upwards of six months, you have stated CPCo's intention to proceed with discovery.

Your letter correctly states my concern that CPCo might use the information learned during discovery to correct non-conforming conditions in the plant or to change or supplement quality related documentation in a manner that might interfere with or hinder the NRC's investigations or inspections. In order to avoid this problem, you commit in the letter that CPCo will inform Region III of any proposed corrective action prior to the time such action is begun.

Region III finds this condition to provide generally acceptable protection for the NRC's investigatory and inspection efforts into these issues. Should we conclude, however, that any of the corrective actions you propose to take would interfere with the NRC's investigations or inspections, we will request you to hold those corrective actions in abeyance until we

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indicate that they may be undertaken. Subject to this caveat, Region III has no objection to CPCo's resumption of discovery as stated in your letter.

Sincerely,

Original signed by
A. Bert Davis

James G. Keppler
Regional Administrator

Enclosure: Ltr dtd 3/22/83 from
Michael I. Miller, Esq. to
James G. Keppler

cc w/encl:
DMB/Document Control Desk (RIDS)
Resident Inspector, RIII
The Honorable Charles Bechhoefer, ASLB
The Honorable Jerry Harbour, ASLB
The Honorable Frederick P. Cowan, ASLB
The Honorable Ralph S. Decker, ASLB
William Paton, ELD
Michael Miller
Ronald Callen, Michigan
Public Service Commission
Myron M. Cherry
Barbara Stamiris
Mary Sinclair
Wendell Marshall
Colonel Steve J. Gadler (P.E.)

RIII	RIII	RIII	RIII
VHZ	Previously concurred. VHZ		
Lewis/jp	Warnick	Davis	for Keppler
3/30/83	3/30/83	3/30/83	3/30

IE Concurrence per Pawlik, 3/30/83
OI (Telephone) VHZ

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COUNSELORS AT LAW

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202 833-9730

March 22, 1983

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	)	50-330-OL
and 2)	)	

Mr. James G. Keppler
Director, Region III
Division of Inspection and Enforcement
Nuclear Regulatory Commission
799 Roosevelt Road
Glen Ellyn, IL 60137

Dear Mr. Keppler:

As you know, there are certain Contentions in the Midland Operating License Proceeding which have been accepted by the licensing board for litigation and which deal with quality assurance related matters. Specifically, Mary Sinclair Contention 1 relates to miscellaneous quality assurance issues and relies on information supplied by certain anonymous affiants. The information as well as the identity of the affiants is purportedly in the possession of the Governmental Accountability Project ("GAP"). Contentions Mary Sinclair 15, 16, and 17 asserts that there are quality assurance related deficiencies in the HVAC systems at Midland. That Contention is based on the affidavits of Mr. Terry Howard and Ms. Sharon Marello, former Zack Co. employees.

In the summer of 1982 we caused the Licensing Board in the above-captioned proceeding to issue subpoenas directed to three employees or associates of GAP. In August 1982 we met with you regarding the subpoenas and other discovery of these issues we planned to institute.

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At your request, pending completion of the NRC's investigation of both the so-called GAP allegations and the Zack HVAC issue, we deferred enforcement of subpoenas and further discovery regarding these matters. We now understand that conclusion of the NRC's investigations is not likely to occur before the summer of 1983. Accordingly, we wish to pursue our discovery efforts in the operating license proceeding.

It is my understanding that you are concerned that information revealed during the discovery process will be used by Consumers Power Company to correct non-conforming conditions in the plant or to change or supplement quality related documentation. On behalf of the company, I assure you that no such action will be taken secretly or in any way that would hinder the NRC's own investigative efforts. In the event that affiants have any knowledge of non-conforming conditions or documentation at the Midland Plant, and the company deems it appropriate to take corrective action as a result of these disclosures, we will inform you of any proposed corrective action fifteen days prior to the time such action is begun.

Unless I hear from you to the contrary in 14 days, I plan to pursue discovery as outlined above. In any event, before initiating such discovery I will contact your counsel, Bill Payton and Steve Lewis.

Yours truly,

Michael I. Miller

MIM:cjs
cc: Service List

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Carole Steinberg, Esq.
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RFW
orig + 3
~~Shafer~~
~~Hawkinson~~
~~Gardner~~
~~Lindeman~~

March 22, 1983

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

Mr. James G. Keppler
Director, Region III
Division of Inspection and Enforcement
Nuclear Regulatory Commission
799 Roosevelt Road
Glen Ellyn, IL 60137

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At your request, pending completion of the NRC's investigation of both the so-called GAP allegations and the Zack HVAC issue, we deferred enforcement of subpoenas and further discovery regarding these matters. We now understand that conclusion of the NRC's investigations is not likely to occur before the summer of 1983. Accordingly, we wish to pursue our discovery efforts in the operating license proceeding.

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Yours truly,

Michael I. Miller

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Consumers Power Company

James W Cook
Vice President - Projects, Engineering and Construction

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

December 28, 1982

J G Keppler, Administrator, Region III
U S Nuclear Regulatory Commission
799 Roosevelt Road
Glen Ellyn, IL 60137

MIDLAND NUCLEAR COGENERATION PLANT
MIDLAND DOCKET NOS 50-329, 50-330
ACCESS TO JOBSITE BY SOURCES OF ALLEGATIONS
FILE: 15.3 SERIAL: 20355

Dear Mr Keppler:

Region III has received a number of allegations regarding the Midland Project. These have been made by sources, some of whom have been publicly identified and some of whom have apparently requested non-disclosure of their identities. Recently, Region III has requested that one of its investigators be permitted access to the Midland job site with one such source in order to facilitate an NRC investigation of the merits of the allegations.

Consumers Power Company wishes to cooperate fully with the NRC in its investigations into the merits of all allegations regarding the quality of construction at the site. Accordingly, we are pleased to grant the sources of the allegations access to the site in the presence of the NRC investigators. Indeed, we urge the NRC to encourage all sources of allegations to visit the site with NRC investigators to specifically point out the defects, if any, which are the subjects of the allegations.

In accommodating the sources of allegations who come to the site, we wish to maintain the appropriate security measures and obtain an understanding of the technical specifics of the allegations. Accordingly, the routine plant security measures which apply to the NRC (e.g. signing in and out, wearing badges, etc) would apply in the normal course to the sources of allegations who visit the site. Also, in conformance with our normal plant security and insurance procedures, which provide that all site visitors be escorted by an official of Consumers Power Company, we would designate a responsible official to participate in each site visit. The official would be technically competent in the area of the allegation and would record the allegation in accordance with the existing MPQAD procedure which, upon request, includes reasonable measures aimed at protecting the anonymity of the sources of allegations. In addition, depending upon the source and nature of the

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allegation, the Company may desire to have present an additional person from its contractor or consultant organizations. He, too, will honor any request for anonymity. While the Company official may respond to NRC inquiries, during the visit he will not question or challenge the validity of the allegations. This will certainly facilitate the NRC investigations and, to the extent the allegations have any merit, it will enable us to make the necessary repairs, or otherwise resolve the matters.

Because some of the sources may request confidential treatment or restricted disclosure of their identities, we are prepared to schedule the site visits at times consistent with attaining that objective, e.g., site visits may be scheduled for weekends or after hours. Of course, we cannot guarantee that an individual visiting the site will not be recognized; we can, however, assure you that neither we nor our contractors or consultants will engage in any retribution towards such sources.

Some sources of allegations may wish to be accompanied during the site visit by a person other than the NRC investigator and the Company official. Subject to conformance with our normal plant procedures, we will have no objection if any such source requests participation in the site visit by a co-worker on-site or by his or her union representative on-site.

Site visits, under these ground rules, will materially aid NRC investigations and the resolution of the allegations, and will assure the safety of all site visitors without jeopardy to plant security. We applaud your efforts to search out the facts behind the allegations and assure you of our full cooperation.

James W. Cook

CC: RSWarnick, NRC Region III
WDShafer, NRC Region III
RJCook, Midland Resident Inspector