J. Kane
13/B5

RECORD OF TELEPHONE CONVERSATION

DATE: March 26, 1982 8:30 am to 11:30 am PROJECT: Midland

RECORDED BY: Joseph Kane CLIENT: ______________

TALKED WITH:

CPC	Bechtel	COE	GEI	NRC
T. Thiruvengadam	J. Anderson	H. Singh	S. Poulos	J. Kane
K. Razdan	L. McKelvey			
R. Ramanujam	C. Russell			
	W. Paris			
	S. Afifi			
	N. Swanberg			

ROUTE TO: Information

G. Lear	S. Poulos
L. Heller	H. Singh
D. Hood	R. Landsman
F. Rinaldi	R. Gonzales

MAIN SUBJECT OF CALL: The purpose of this call was for the Staff to respond to CPC on three review issues which had been discussed during the March 16-19, 1982 audit. During the audit the Staff indicated they would respond to Consumers by March 26, 1982. The review issues involved include adoption of soil spring constants, settlement monitoring and construction dewatering for underpinning work related to the Service Water Structure.

ITEMS DISCUSSED:

1. Adoption of Soil Spring Constants. (Reference - Calculations by Bechtel DQ-29(Q) dated 12/10/81 and 2/22/82; Calculations dated 12/11/81 by G. W. Cowling; App. A entitled Service Water Pump Structure, Three-Dimensional, Finite Element Model provided by Carl Dirnbaver on 2/23/82).

   Bechtel indicated that values of soil spring constants determined in G. W. Cowling's calculation for System 5, (Long term condition) are K_{main} = 190 KCf and $Ku/_{pin}$ = 230 KCf for adopted differential settlement condition of 0.2" beneath portion on till and 0.5" beneath underpinning wall. For adopted differential settlement condition of 0.4" beneath portion on till and 0.1" beneath underpinning wall the spring constant values are k_{main} = 95 KCF and $Ku_{/pin}$ = 1150 KCf. The Staff indicated its agreement with the values of k_{main} = 190 KCf and $Ku/_{pin}$ = 230 KCf as being reasonably conservative for the design of the SWS. The Staff did not object to adoption of the other spring constant valves but do not feel the differential settlement which is assumed in this condition is likely to occur.

The values of the short term soil springs for seismic loading (System 4) are not identified in Table 1, App. A but were provided in previous testimony and technical reports by Consumers. The Staff has not completed their review of the seismic spring constants and will respond to Consumers at a later date on the appropriateness of these values.

The values of spring stiffnesses adopted for system 3 (k_{main} = 150 KCF, $ku_{/pin}$ = 400 KCF) and FSAR load combination conditions are acceptable to the Staff. The Staff noted that the values adopted for system 5 (k_{main} = 190 KCF and $ku_{/pin}$ = 230 KCF) would also be a reasonable estimate of stiffnesses for FSAR load combination conditions under System 4.

There was considerable discussion on Systems 1 and 2 in Table 1, Appendix A with respect to how structural stresses have been determined due to jacking loads and also due to settlements which have already occurred. The geotechnical area of concern is identifying the appropriate soil stiffnesses to use for these loading conditions. Because of the inability to reach an agreement on these issues, it was suggested that L. McKelvey from Bechtel and S. Poulos from GEI get together by conference call next week to work out an understanding on what conditions require analysis. Following their discussions, another conference call would be arranged with the parties of today's conference call to inform both Consumers and the Staff on what agreements could be reached and what soil spring stiffnesses are required.

2. Settlement Monitoring During Service Water Structure (SWS) Underpinning. (Reference - Drawing C-2003). As agreed upon between Consumers and the Staff during the March 19, 1982 audit, a permanent bench mark will be added near the southeast corner of the SWS. Locating this new permanent benchmark on the east wall as far south as it can conveniently be positioned near SW-3 is acceptable to the Staff. The major reason for this addition is to permit the differential settlement to be accurately established between the portion of the SWS founded on till and the overhanging portion presently founded on the fill.

   The Staff and its Consultants indicated their difficulties with Consumers previously stated intent not to require control of underpinnning operations with established allowable settlement limits. NRC difficulties include the strain gage approach proposed by Consumers may not be a sensitive control and may not give as early a warning as measured settlements on the bench marks. The Staff and its Consultants recommended that, similar to what is being carried out for the Auxiliary Building, allowable settlement limits be established at the permanent benchmarks based on a structural analysis, where critical stresses due to differential settlement beyond rigid body motion have been calculated.

Consumers indicated their concerns with establishing allowable differential settlement limits which included recognition of the more rigid structure and considerably shorter length of the main portion of the SWS on till in comparison to the Auxiliary Building. These conditions, in Consumers opinion, would result in very small settlement limits which would be impractical to measure and would possibly be overshadowed by daily fluctuations due to climate changes. This matter, after considerable discussion, was not resolved but an understanding was reached that the Staff would not take a position until after their review of the strain monitoring program had been completed. This program was to be provided by Consumers in early April. Consumers agreed to consider the Staff's recommendation and provide a more definite indication on the magnitude of differential settlements which are involved.

With respect to the frequency of readings indicated in Step 3 on Drawing C-2003, the Staff made the following recommendations:

a. At least one week prior to the start of excavation below the foundation slab of the SWS, good background settlement data should be obtained for the three permanent benchmarks by increasing the frequency to a minimum of three times a day in order to observe the effects of climate changes. Consumers indicated their agreement to this Staff request.

b. When excavations below the SWS foundation slab proceed in order to install corner piers 1, 2, 3 and middle pier 4 and during the jacking of these piers, the frequency of settlement monitoring should be increased to a minimum of twice per shift. The frequency of readings can be lengthened to once every 24 hours after corner piers 1, 2 and 3 are completed and before work on pier 4 is initiated, if access to the piers is by way of an excavation outside the SWS as presently being considered by Consumers. Consumers expressed agreement with this request.

c. Following the successful construction of piers 1, 2, 3 and 4, the frequency of readings as presently proposed by Consumers on Drawing C-2003 would be acceptable to the Staff.

   It was indicated to Consumers that this increased frequency of readings during the more critical underpinning operations was also applicable to the strain gage monitoring program. Consumers agreed to consider the need for increased frequency of strain monitoring in their current work in developing this program. The frequency of readings proposed by Consumers for building settlement markers other than the permanent bench marks is acceptable to the Staff.

3. Construction Dewatering. (Reference - Plan of Construction Dewatering, Sections A-A and B-B, and Dewatering Procedure transmitted February 26, 1982 from Spencer, White and Prentis, Inc. to Bechtel Power Corporation). The Staff indicated their acceptance of the locations of the dewatering wells and piezometers (observation wells) and made the following recommendations on the above referenced information:

   a. The depths of the six proposed piezometers should extend to at least elevation 570. The top of the filter sand in the piezometers should extend to elevation 590 and then be sealed above this elevation.

   b. The type of piezometer to be installed should be sensitive (e.g., 3/8" maximum inside diameter or an air pressure cell type) to sudden piezometric changes in order to avoid long periods of time lag responses.

   c. The specification for installing and operating the construction dewatering system should establish a construction control on the upper phreatic surface. This control should require a minimum 2 foot depth between the upper phreatic surface being controlled by dewatering and the bottom of any open underpinning excavation at any given time. The depths of the proposed dewatering wells should then be drilled accordingly to accomplish this construction control on the upper phreatic surface.

   d. Placement of the filter sand in the dewatering wells should be by the tremie method rather than shoveling from the ground surface in order to avoid segregation of the filter sand particles in holes larger than 6-inch in diameter. Consumers agreed to modify the construction dewatering plans and specifications to incorporate the Staff's recommendations for above items 3a., 3b., 3c., and 3d.

   e. The Staff recommended that paragraph 7.2 of the dewatering procedures provided by Consumers be modified to define what constitutes soil particles in the discharge water (inorganic, nonmetallic materials coarser than 0.005 millimeter) and to indicate the frequency of testing similar to the agreements reached with the Staff as reflected in the June 18, 1981 letter from R. Tedesco to J. Cook. Consumers responded that the construction dewatering is a temporary system and therefore not subject to the agreements reached in the June 18, 1981 letter which covered the permanent dewatering system. There was considerable discussion on the safety significance and impact that a filter media criteria of 0.005 mm versus 0.05 mm (Consumers) could have on underpinning operations for both the Auxiliary Building and Service Water Structure. The Staff acknowledged that the 0.05 mm criteria proposed by Consumers would not result in a serious removal of foundation soil particles if this condition persisted for a period of several weeks. On the other hand the Staff considered the volume of soil fines which could be removed to be excessive if soils finer than 0.05 mm were being removed by dewatering over the anticipated one year period when underpinning work was being completed. The past monitoring procedures employed by Consumers on construction dewatering did not permit a conclusion to be reached as to whether a real problem existed for fines sized between 0.05 mm and 0.005 mm. The Staff suggested a compromise where the Staff would agree with Consumer's proceeding with construction dewatering using the 0.05 mm filter criteria but requiring the testing for soil particles at both the 0.05 mm and 0.005 mm filters. Consumers agreed to bring the results to the attention of I&E inspectors and NRR of any test where the amount of soil particles in the discharge water exceeded the limiting 10 ppm when measured on the 0.005 mm filter. At that time an engineering evaluation would be made as to the seriousness of the developing condition based on actual seepage pumping rates which are not now available. If the loss of soil particles were deemed significant enough during the remaining period of underpinning the Staff would require remedial actions to reduce that loss.

In response to Consumer's request for NRC approval in their proceeding with construction dewatering, the Staff indicated their concerns have now been resolved but that approval must come from Division of Licensing. The Staff indicated D. Hood would be notified of the results of today's conference call and suggested that Consumers directly contact him sometime later this afternoon.

On a matter not directly related to the subject of today's call, Bechtel indicated they are presently considering running a loading test at locations near, but not in the actual pier locations at both the Auxiliary Building and Service Water Structure areas. This planning is in the early stages and it is Consumers intention to submit the load test procedures and details for Staff review in the very near future.

SERVICE WATER STRUCTURE J. Kane Rec'd 9/8/81

14/185

James W Cook
Vice President - Projects, Engineering and Construction

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

August 26, 1981

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

MIDLAND PROJECT
DOCKET NOS 50-329, 50-330
SOILS SETTLEMENT REMEDIAL ACTION
FOR THE SERVICE WATER PUMP STRUCTURE (SWPS)
FILE: 0485.16, B3.0.8 SERIAL: 13738
REFERENCES: JWCOOK TO HRDENTON, SERIAL 11625 DATED MARCH 23, 1981.
ENCLOSURES: MIDLAND UNITS 1 AND 2 - TECHNICAL REPORT
ON UNDERPINNING THE SERVICE WATER PUMP STRUCTURE.

In the referenced correspondence of March 23, 1981 we advised the NRC of the underpinning concept for the overhanging portion of the service water pump structure which is a full length wall extending into the natural till material. This full length wall concept was adopted to replace the original remedial action, a driven pile support concept, as a result of the increased seismic requirements imposed by the staff. We are forwarding thirty (30) copies of the enclosed report entitled "Technical Report on Underpinning the Service Water Pump Structure" which describes the design and construction requirements of this SWPS remedial action.

The design and construction criteria contained in the attached report has been written to provide the NRC with information which substantially exceeds the construction permit level of detail. Included in this report are the following types of information: (1) drawings showing the underpinning scheme and a description of the construction sequence for this scheme; (2) dewatering for construction; (3) the design and acceptance criteria for the underpinning scheme, including load combinations, bearing pressures, structural stresses, and seismic loads; (4) applicable codes; and (5) scope of the quality assurance requirements.

The proposed service water structure remedial underpinning is approximately a 4-foot thick, reinforced concrete wall that is approximately 30 feet high with a flared base at the north wall and is constructed to act as a continuous member under the perimeter of that portion of the structure founded on backfill material. In addition, a predetermined jacking force will be applied to the full perimeter of the SWPS overhang during construction to provide adequate load transfer from the structure to the underpinning wall.

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While we believe that the enclosed report provides sufficient information to permit the NRC to review and provide its concurrence with the proposed underpinning scheme, we suggest that a technical review meeting be held during the week of August 31, 1981 to respond to any outstanding NRC concerns. Please contact us to establish a mutually agreeable day for this meeting.

Your expeditious review and approval would be most appreciated to support the hearings and construction of the remedial work.

James W. Cook

JWC/RLT/cr

CC Atomic Safety & Licensing Appeal Board, w/o
Atomic Safety & Licensing Board Panel, w/o
Charles Bechhoefer, Esq, w/o
MMCherry, Esq, w/o
RJCook, Midland Resident Inspector, w/o
Dr FPCowan, w/o
RSDecker, w/o
NRC Docketing Service Section, w/a
SGadler, w/o
RWHuston, Washington, w/a
JDKane, NRC w/a
FJKelley, Esq, w/o
WHMarshall, w/o
MIMiller, Esq, w/a
WOtto, US Army Corps of Engineers, w/a
WDPaton, Esq, w/o
MSinclair, w/o
BStamiris, w/o
HSingh, US Army Corps of Engineers, w/a

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BCC RCBauman/TRThiruvengadam, P-14-400, w/o
WRBird, P-14-418A, w/a
JEBrunner, M-1079, w/a
GSKeeley, P-14-113B, w/a
DBMiller, Midland, w/a
NRamanujam, P-14-100, w/a
TJSullivan/DMBudzik, P-24-517, w/o
RLTeuteberg, P-24-513, w/a
ALBoos, Bechtel, w/a
Dr AJHendron, Bechtel Consultant, w/a
DFJudd, B&W, w/o
Dr Ralph B Peck, Becthel Consultant, w/a
SSAfifi, Becthel, w/a
JARutgers, Bechtel, w/a
WJCloutier, P-24-611, w/a
KLRazdan, P-13-220, w/a

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TECHNICAL REPORT ON UNDERPINNING
THE SERVICE WATER PUMP STRUCTURE
FOR
MIDLAND PLANT UNITS 1 AND 2
CONSUMERS POWER COMPANY
DOCKET NUMBERS 50-329 AND 50-330

AUGUST 25, 1981

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TECHNICAL REPORT ON UNDERPINNING
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TECHNICAL REPORT ON UNDERPINNING

THE SERVICE WATER PUMP STRUCTURE

1.0 INTRODUCTION

This report describes the design and construction requirements of the remedial action for the service water pump structure (SWPS) necessitated by the settlement potential of the plant fill underlying the structure.

2.0 PRESENT CONDITION

The SWPS is a two level, rectangular, reinforced concrete structure. Below el 617', it measures 86 feet by 71 feet 11 inches; above el 617' it measures 106 feet by 86 feet. The maximum overall height is 69 feet [See Figures 1 and 2 (FSAR Figures 3.8-56 and 3.8-57)].

The structure was designed to be supported by the two foundation slabs, one at el 587'-0" and the other at el 617'-0". The lower slab rests on undisturbed natural material and the upper slab rests on fill material placed during construction in 1977.

After discovering settlement of the fill under the diesel generator building, an investigation of the plant fill revealed some questionable areas under the upper base slab, el 617'-0", of the SWPS.

3.0 REMEDIAL ACTION

For the part of the structure resting on plant fill, a continuous underpinning wall, resting on undisturbed natural material, is provided to support the structure adequately under all design load conditions. The underpinning wall provides the necessary vertical and horizontal support to the affected part of the structure. To ensure adequate load transfer, the underpinned structure is jacked from the underpinning walls (Refer to Figure 3).

4.0 DESIGN FEATURES

The proposed underpinning 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 allowable 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 safety factors of 2 for dynamic loading and 3 for static loading.

A predetermined jacking force is applied to the overhang perimeter to provide adequate load transfer from the structure to the underpinning.

The connection between the underpinning wall and the existing structure is made by 2-inch diameter rock bolts at the vertical interfaces and 2-3/4-inch diameter anchor bolt assemblies at the horizontal interfaces (Refer to Figures 4 and 5). The connectors are designed to transfer shear and tension forces to the underpinned wall. The connectors are not subject to stresses during the jacking procedures because the rock bolts have not yet been installed and the anchor bolts have not been tightened (Refer to Subsection 5.3.2). After the underpinning wall is connected to the existing structure, the connectors are stressed by loads applied to the underpinned structure.

5.0 CONSTRUCTION

The construction procedures discussed in this report are recommended for underpinning the SWPS. If subcontractor recommendations result in improved procedures, they will be incorporated. For details of construction and the construction procedures, refer to Figures 4 and 5.

5.1 DEWATERING

To construct the underpinning, the SWPS site is dewatered: The groundwater level is lowered to el 587 (approximately) by using temporary dewatering wells. These wells will be sealed after the underpinning wall is completed. The acceptance criteria for the dewatering system require that the system produces an effluent that has less than 10 parts per million of soil particles larger than 0.05 millimeters.

5.2 BUILDING POST-TENSIONING

Construction site dewatering removes the buoyancy force on the overhang portion of the structure, resulting in additional loading on the overhang. To compensate for this additional loading of the overhang, a temporary post-tensioning system applies a compressive force to the upper part of the building along each north-south wall. This post-tensioning allows the additional force to be transferred from the overhang by beam action to the adjoining walls which rest on undisturbed natural material (Refer to Figure 6). The post-tensioning system is removed after the initial jacking loads are applied.

5.3 CONSTRUCTION PROCEDURES

The underpinning is constructed as individual piers tied together by threaded reinforcing bar couplers and shear keys to form a continuous wall. Refer to details and procedures in Figures 4 and 5.

5.3.1 Initial Construction Activities

To preserve the structural integrity of the building, the underpinning wall is constructed in small sections (piers) from tunnels which are advanced simultaneously from access shafts located at the northeast and northwest corners of the building. The tunnels initially extend only far enough to construct an approximately 30-foot deep, 5 foot by 4 foot, sheeted pit at each corner of the overhang. The pit is hand dug. The shear strength of the subgrade soil is assessed with a Corps of Engineers cone penetrometer, model CN-973. Under a maximum force of 150 pounds, the cone should not penetrate the surface more than 1/2 inch. After the subgrade is inspected and approved by a geotechnical engineer, reinforcement, subgrade settlement monitoring instrumentation, and anchor bolt assemblies to tie the pier to the underside of the slab, are installed. The pier is then cast with concrete pumped from the access shaft. After at least 48 hours of curing, an initial jacking load is applied to the overhang from jacks placed on the pier top. To ensure adequate support to the building, the tunnel is not advanced to the next stage until the pier is jacked.

Simultaneously with applying the jacking force, the tunnels are advanced to the location of the next pier, which is constructed in a similar manner to the first pier. The piers are tied together with threaded reinforcing bar couplers and shear keys to form a continuous underpinning wall. The threaded reinforcing bar couplers (see Detail 1, Figure 5) conform to the requirements of Section III, Division 2 of the American Society of Mechanical Engineers Boiler and Pressure Vessel Code, 1980 Edition, 1980 and 1981 Summer Addenda. The tensile strength of the splice system is not less than 125% of the specified minimum yield strength of the spliced bar.

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 the monitoring program is used to evaluate the time required to dissipate shrinkage and creep of the concrete and creep of the undisturbed natural material. The rate of settlement decreases with time. At the proper point on the settlement-time curve (as determined by the geotechnical engineer), the final jacking operations (as described below) begins.

what established the proper point

5.3.2 Final Jacking Stage

how established

After Piers 10 (Figure 4) are constructed, the underpinning wall has progressed to within 6 feet of the vertical interfaces with the existing structure, and the final jacking load is applied. Settlements caused by this load are monitored. When the geotechnical engineer judges that the settlement rate has decreased to a proper value, 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. Piers 11 are poured, encasing rock anchors that were previously drilled into the vertical face of the existing structure and thereby connecting the underpinning wall to the vertical face of the existing stucture (Refer to Detail 5, Figure 5). 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 (which tie the top of the piers to the foundation slab) are tightened (Refer to Detail 7, Figure 4). The underpinning wall is connected to the structure at both the vertical and horizontal interfaces.

how established

5.3.3 Completion of the Underpinning Wall

The tunnel is backfilled with lean concrete beginning at the vertical interface and at the north wall. The completion of the tunnel backfilling terminates at the locations of Piers 12. These piers are then constructed, completing the underpinning wall.

6.0 MONITORING REQUIREMENTS

During construction, the underpinning of the existing structure is monitored for settlement and crack propogation. The long-term surveillance program of the building after the construction of the underpinning is being evaluated.

6.1 SETTLEMENTS

The elevations of settlement markers attached to the structure are measured in accordance with a schedule based on construction procedures. Expected building movements during underpinning operations are small. These movements are recorded, and those exceeding 1/4 inch will be evaluated and reported to the NRC.

6.2 CRACKS

Monitoring of existing or new cracks appearing during the underpinning construction is scheduled. Because of the sequencing of construction procedures, it is not anticipated that existing cracks will significantly widen or new cracks will appear. However, any new structural cracks or changes in existing structural crack widths exceeding 0.010 inch will be evaluated and reported to the NRC.

7.0 ANALYSIS AND DESIGN

The SWPS was originally designed in accordance with FSAR requirements for Seismic Category I structures. A preliminary analysis of the underpinned structure was made which complied with these FSAR requirements, and added a jacking load to the load combinations. The seismic loads used in this analysis were extrapolated from the seismic loading from a previous underpinning design based on piles. When the final seismic loads become available, they will be incorporated in the final design.

In the final design, seismically induced forces and instructure response spectra of the structure are generated in accordance with FSAR Section 3.7. The revised model portrays the structural behavior including the effects of the underpinning and associated foundation modification.

The mathematical seismic model and a description of the soil-structure interaction coefficients to be used in the seismic analysis will be submitted to the NRC in September 1981.

The static structural analysis uses an analytical model capable of representing the structure behavior. The interface between the existing structure and the underpinning wall is modeled to transfer direct loads without providing rotational restraint. The soil media are represented by springs of appropriate stiffness at the base of the structure. The detailed analysis will be performed by conventional methods such as beam theory and/or plate theory or by using the computer program Bechtel Structural Analysis Program (BSAP). For details of the BSAP computer program see FSAR Subsection 3.8.3.4.

7.1 STRUCTURE BEHAVIOR

The vertical loads of the structure are transmitted to the foundation medium through the existing base slab at el 587'-0" and the underpinning wall bearing area. The lateral forces due to seismic and tornado loads are resisted by the shear walls in the structure. These lateral loads are transferred to the foundation medium by the combined action of the base slab at el 587'-0" and the underpinning wall bearing area. To ensure this action, the underpinning walls are connected to the existing structure by rock anchors and anchor bolts capable of transferring all direct loads. This connection is a pinned connection that is consistent with the analysis method.

7.2 DESIGN CRITERIA AND APPLICABLE CODES

The underpinned structure is designed as a Seismic Category I structure. The design complies with the requirements of ACI 318-71 and the 1969 edition of the AISC.

7.3 LOADS AND LOAD COMBINATIONS

The underpinning structure rests entirely on undisturbed natural material. The preliminary analysis of the underpinned structure utilizes the same load combinations used in the original design. However, each load combination is modified by adding the jacking load (P_L). For each loading combination, the jacking load was evaluated with two load factors: a value of 1.0, and the load factor associated with the dead load for that load combination.

For the design of the underpinning and the connections to the existing structure, the safe shutdown earthquake (SSE) forces were increased by 50% to provide for a possible future increase in this loading. The 50% increase was applied to the seismic response of the structure corresponding to the analytical model with the mean soil properties. The existing structure was checked for a 0.12g SSE.

The long-term settlement of the underpinning wall after it is connected to the existing structure will be calculated. The calculation is based on properties of the supporting soil. The long-term settlement effects will be considered in the final analysis of the structure. To provide for these effects, the final analysis is governed by four additional load combinations. These load combinations are discussed in the response to Question 15 of the NRC Requests Regarding Plant Fill (September 1979) and were used in the diesel generator building reanalysis. The load combinations are modified by the addition of the jacking load.

Table 1 lists 26 load combinations, modified for jacking loads. For the preliminary analysis of the underpinned SWPS, the following load combination was most critical:

$$U = 1.0D + 1.0L + 1.0E' + 1.0T_o + 1.25H_o + 1.0R + P_L$$

where

D = dead loads

L = live loads

E' = safe shutdown earthquake

T_o = thermal effects during normal operating conditions

H_o = force on structure due to thermal expansion of pipes under operating conditions

R = local force or pressure on structure or penetration caused by rupture of any one pipe

P_L = load on structure due to jacking preload

In addition to this load combination, the underpinned structure was checked for stability using the load combinations specified in FSAR Suosection 3.8.6.3.4.

A complete analysis of the underpinned structure, using all applicable load combinations, will be made when the final seismic loads become available.

7.4 STRUCTURAL ACCEPTANCE CRITERIA

The acceptance criterion for analyzing the underpinned structure is in accordance with FSAR Subsection 3.8.6.5.

8.0 QUALITY ASSURANCE REQUIREMENT

This project work is a combination of Q- and non-Q-listed work. The construction of the permanent structures such as the underpinning wall and the connectors are Q-listed, as well as any other activity or structure necessary to protect the SWPS. Construction of temporary structures such as the access shafts and tunnels is non-Q-listed. A detailed quality plan shall be prepared by the subcontractor to identify those specific activities which are required to have a safety "Q" quality program applied along with the major quality program elements for these activities. This quality plan shall be approved by Bechtel and Consumers Power Company prior to the start of any Q-listed work.

9.0 ADDITIONAL NRC REQUIREMENTS

For information purposes, an analysis of the critical sections of the underpinned structure will be made conforming to the provisions of ACI 349-76 as supplemented by NRC Regulatory Guide 1.142.

TABLE 1

LOAD EQUATIONS FOR THE SERVICE WATER PUMP STRUCTURE MODIFIED TO INCLUDE PRELOAD

Responses to NRC Requests Regarding Plant Fill, Question 15

a. Normal Operating Condition

$$U = 1.05D + 1.28L + 1.05T + P_L \qquad (1)$$

$$U = 1.4D + 1.4T + P_L \qquad (2)$$

b. Severe Environmental Condition

$$U = 1.0D + 1.0L + 1.0W + 1.0T + P_L \qquad (3)$$

$$U = 1.0D + 1.0L + 1.0E + 1.0T + P_L \qquad (4)$$

Loading Under Normal Conditions

a. Concrete

$$U = 1.4D + 1.7L + P_L \qquad (5)$$

$$U = 1.25\ (D + L + H_o + E) + 1.0T_o + P_L \qquad (6)$$

$$U = 1.25\ (D + L + H + W) + 1.0T_o + P_L \qquad (7)$$

$$U = 0.9D + 1.25\ (H + E) + 1.0T_o + P_L \qquad (8)$$

$$U = 0.9D + 1.25\ (H + W) + 1.0T_o + P_L \qquad (9)$$

For ductile moment resisting concrete frames and for shear walls

$$U = 1.4\ (D + L + E) + 1.0T_o + 1.25H_o + P_L \qquad (10)$$

$$U = 0.9D + 1.25E + 1.0T_o + 1.25H_o + P_L \qquad (11)$$

Structural Elements Carrying Mainly Earthquake Forces, Such as Equipment Supports

$$U = 1.0D + 1.0L + 1.8E + 1.0T_o + 1.25H_o + P_L \qquad (12)$$

b. Structural Steel

$$D + L + P_L \text{ (stress limit} = f_s) \qquad (13)$$

$$D + L + T_o + H_o + E + P_L \text{ (stress limit} = 1.25f_s) \qquad (14)$$

Table 1 (Continued)

$$D + L + T_o + H_o + W + P_L \text{ (stress limit} = 1.33f_s) \quad (15)$$

In addition, for structural elements carrying mainly earthquake forces, such as struts and bracing:

$$D + L + T_o + H_o + E + P_L \text{ (stress limit} = f_s) \quad (16)$$

Loading Under Accident Conditions

a. Concrete

$$U = 1.05D + 1.05L + 1.25E + 1.0T_A + 1.0H_A + 1.0R + P_L \quad (17)$$

$$U = 0.95D + 1.25E + 1.0T_A + 1.0H_A + 1.0R + P_L \quad (18)$$

$$U = 1.0D + 1.0L + 1.0E' + 1.0T_o + 1.25H_o + 1.0R + P_L \quad (19)$$

$$U = 1.0D + 1.0L + 1.0E' + 1.0T_A + 1.0H_A + 1.0R + P_L \quad (20)$$

$$U = 1.0D + 1.0L + 1.0B + 1.0T_o + 1.25H_o + P_L \quad (21)$$

$$U = 1.0D + 1.0L + 1.0T_o + 1.25H_o + 1.0W' + P_L \quad (22)$$

b. Structural Steel

$$D + L + R + T_o + H_o + E' + P_L \text{ (stress limit} = 1.5f_s) \quad (23)$$

$$D + L + R + T_A + H_A + E' + P_L \text{ (stress limit} = 1.5f_s) \quad (24)$$

$$D + L + B + T_o + H_o + P_L \text{ (stress limit} = 1.5f_s) \quad (25)$$

$$D + L + T_o + H_o + W' + P_L \text{ (stress limit} = 1.5f_s) \quad (26)$$

where

U = required strength to resist design loads or their related internal moments and forces

For the ultimate load capacity of a concrete section, U is calculated in accordance with ACI 318-71.

F_y = specified minimum yield strength for structural steel

f_s = allowable stress for structural steel; f is calculated in accordance with the AISC Code, 1963 Edition for design calculations initiated prior to February 1, 1973.

f is calculated in accordance with the AISC Code, 1969 Edition, with Supplements, 1, 2, and 3 for design calculations initiated after February 1, 1973.

Table 1 (Continued)

D = dead loads

L = live loads

P_L = load on structure due to jacking preload

R = local force or pressure on structure or penetration caused by rupture of any one pipe

T_o = thermal effects during normal operating conditons

H_o = force on structure due to thermal expansion of pipes under operating conditions

T_A = total thermal effects which may occur during a design accident other than H_A

H_A = force on structure due to thermal expansion of pipes under accident condition

E = operating basis earthquake (OBE)

E' = safe shutdown earthquake load (SSE)

B = hydrostatic forces due to the postulated maximum flood (PMF) elevation of 635.5 feet

W = design wind load

W' = tornado wind loads, including missile effects and differential pressure

ϕ = capacity reduction factor

The capacity reduction factor (ϕ) provides for the possibility that small adverse variations in material strengths, workmanship, dimensions, control, and degree of supervision, although individually within required tolerances and the limits of good practice, occasionally may combine to result in undercapacity.

NOTES:

1. In the load equations, the following factors are used:

   ϕ = 0.90 for reinforced concrete in flexure

   ϕ = 0.75 for spirally reinforced concrete compression members

   ϕ = 0.70 for tied compression members

   ϕ = 0.90 for fabricated structural steel

Table 1 (Continued)

ϕ = 0.90 for reinforced steel in direct tension

ϕ = 0.90 for welded or mechanical splices of reinforcing steel

2. Unity load factor is shown for P_L. An alternative load factor to be considered in all load combinations is the load factor associated with dead load (D) in that loading combination.

For load combinations 23-26:

Maximum allowable stress in bending and tension is 0.9 F_y. Maximum allowable stress in shear is 0.5 F_y.

For these load combinations, the maximum allowable stress except for local areas that do not affect overall stability is limited to 0.9 F_y for bending, bearing, and tension and 0.5 F_y for shear. The time phasing between loadings is used where applicable to satisfy the above equations.

Structural components subjected to postulated impulse loads and/or impact effects are designed in accordance with BC-TOP-9-A, Rev 2, using ductility ratios not exceeding 10.

Structural members subjected to missile and pipe break loads are designed in accordance with Bechtel's BC-TOP-9-A, Rev 2, and Bechtel's BN-TOP-2, Rev 2.

14/135

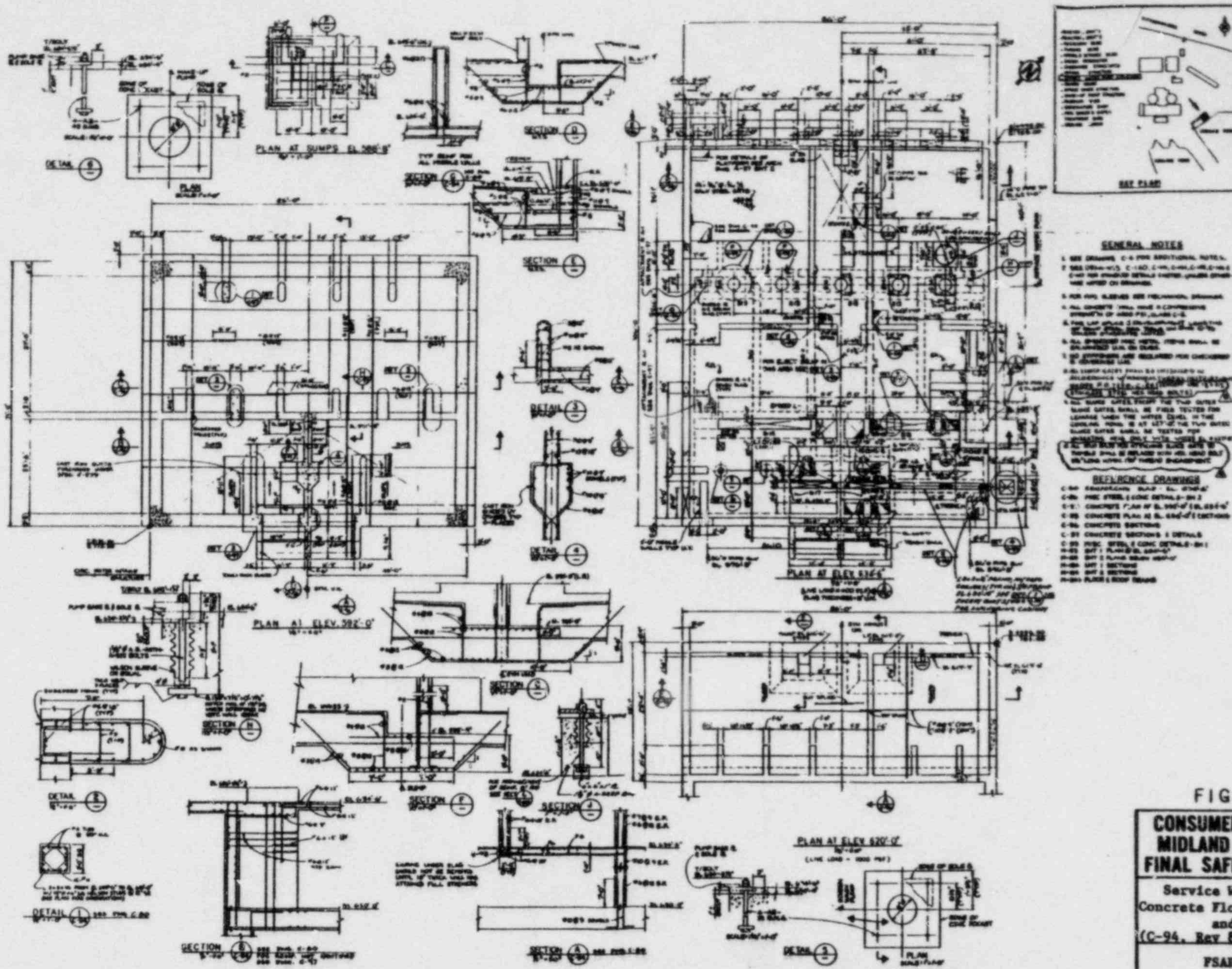


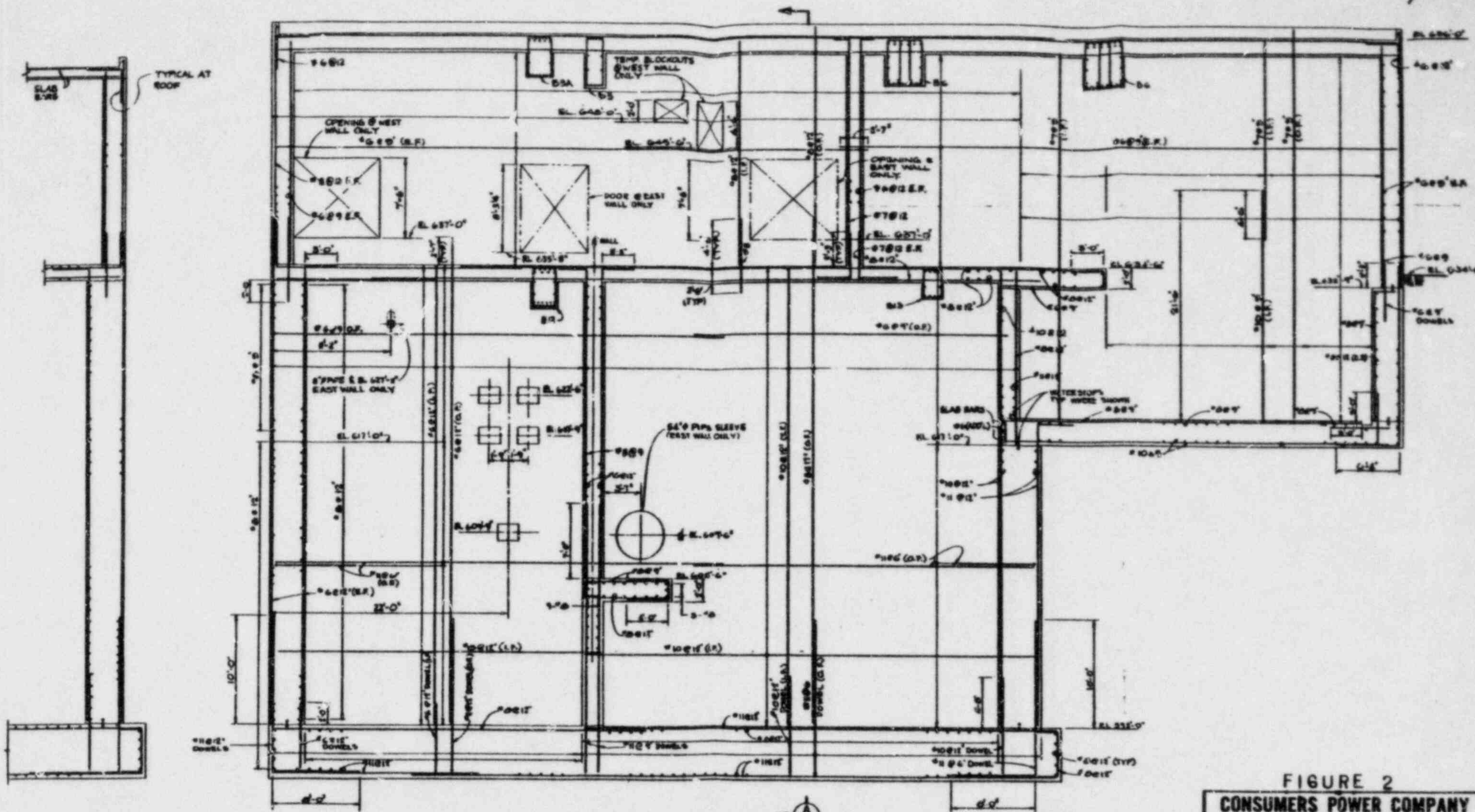
FIGURE I

CONSUMERS POWER COMPANY
MIDLAND PLANT UNITS 1 & 2
FINAL SAFETY ANALYSIS REPORT

Service Water Pump Structure
Concrete Floor Plans at El. 592'-0" and El. 634'-6"
(C-94, Rev 8)

FSAR Figure 3.8-56

14/135



SECTION A

ELEVATION - WEST WALL
ELEVATION EAST WALL (OPP. HAND)

FIGURE 2

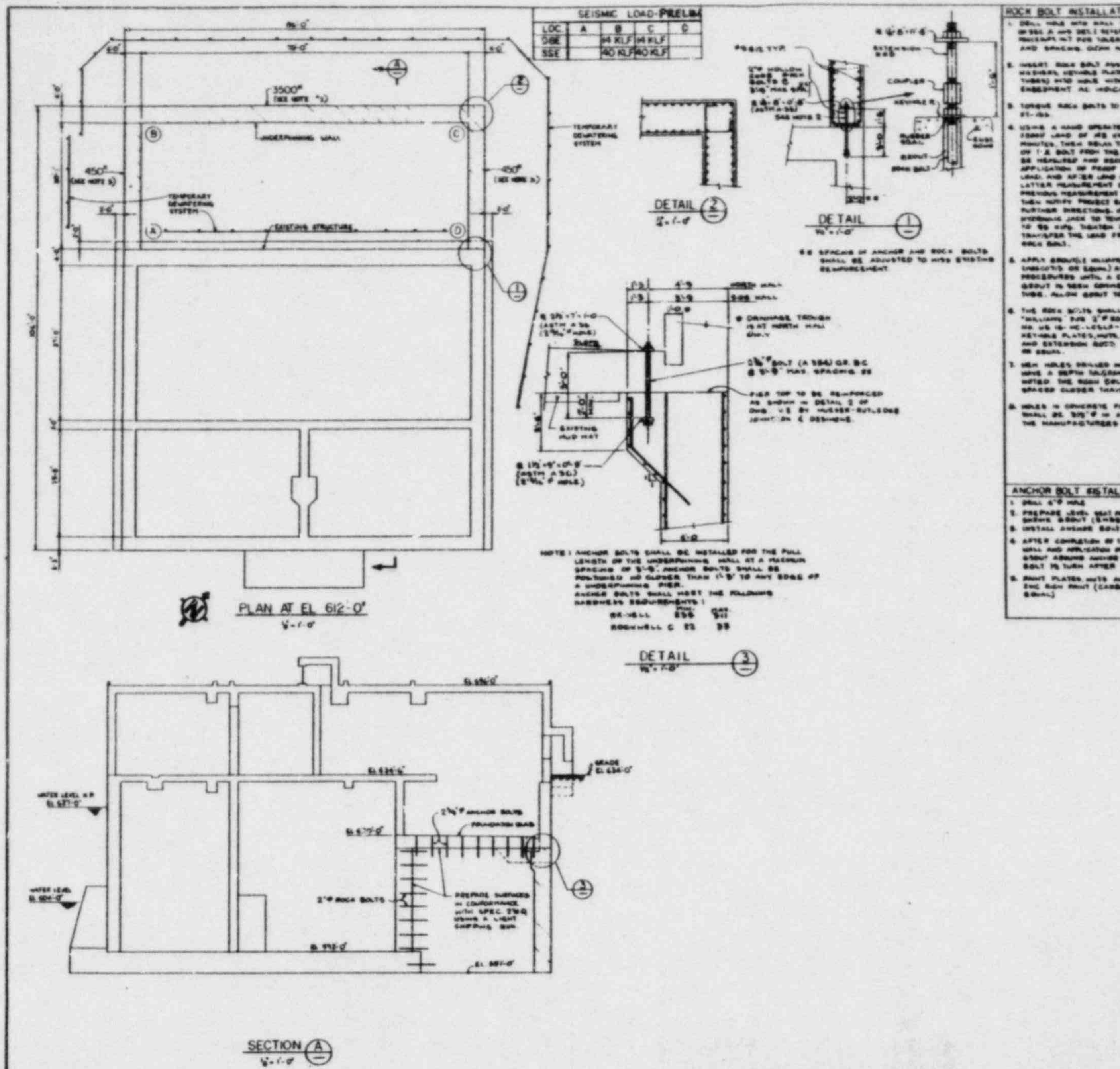
CONSUMERS POWER COMPANY
MIDLAND PLANT UNITS 1 & 2
FINAL SAFETY ANALYSIS REPORT

Service Water Pump
Structure Section

(C-97, Rev 2)

FSAR Figure 3.8-57

14/135

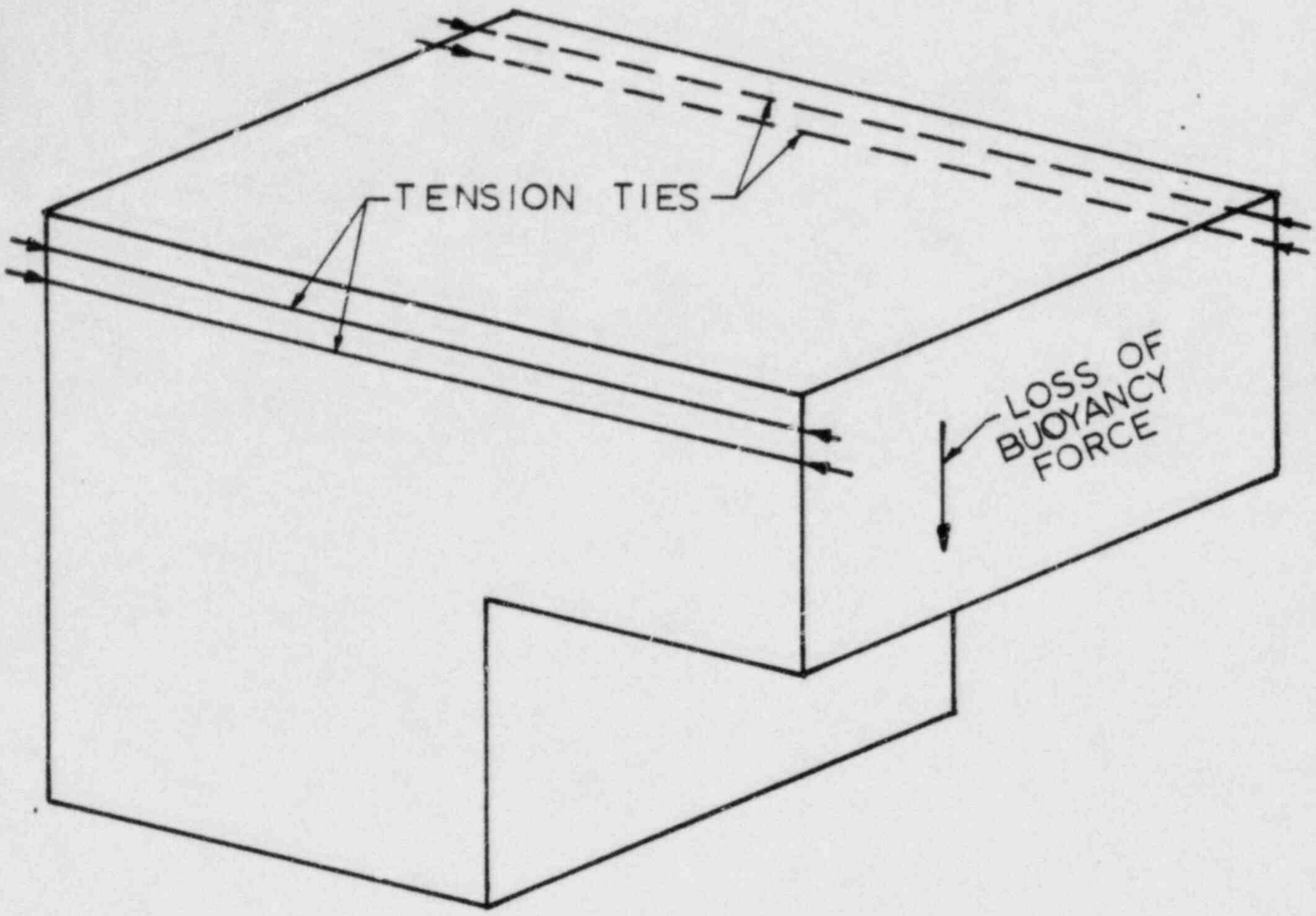


CONSUMERS POWER COMPANY
MIDLAND PLANT UNITS 1 & 2

SERVICE WATER PUMP STRUCTURE
UNDERPINNING REQUIREMENTS

Figure 3

14/135



CONSUMERS POWER COMPANY
MIDLAND PLANT UNITS 1 & 2

SERVICE WATER PUMP STRUCTURE
TENSION TIES

Figure 6

SW Structure

J Kane
Rec'd 11/9/81
15/B5

James W Cook
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November 6, 1981

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

MIDLAND PROJECT
MIDLAND DOCKET NOS 50-329, 50-330
RESPONSE TO NRC STAFF REQUEST FOR ADDITIONAL INFORMATION PERTAINING TO THE PROPOSED UNDERPINNING OF THE SERVICE WATER PUMP STRUCTURE
FILE 0485.16, B3.0.8 SERIAL 14843
ENCLOSURE: RESPONSES TO THE NRC STAFF REQUEST FOR ADDITIONAL INFORMATION PERTAINING TO THE PROPOSED UNDERPINNING OF THE SERVICE WATER PUMP STRUCTURE

On September 17, 1981, a request for additional information relating to the service water pump structure was made by the Staff in a meeting at the NRC's offices in Bethesda, Maryland. We are responding to this request by forwarding the above enclosure. The enclosure addresses each of the individual Staff concerns transmitted to us in the September 17, 1981 meeting.

We believe the enclosed information adequately responds to the request and individual concerns identified for us by the Staff. The discussion and data contained in the enclosure to this correspondence lend further support to our conclusion that the design of the service water pump structure combined with the remedial actions are adequate and appropriate for this structure.

G.S. Keeley for JW Cook

JWC/RLT/dsb

oc1181-0473a100

8111100553

24pp.

CC Atomic Safety and Licensing Appeal Board, w/o
CBechhcefer, ASLB, w/o
MMCherry, Esq, w/o
FPCowan, ASLB, w/o
RJCook, Midland Resident Inspector, w/o
RSDecker, ASLB, w/o
JHarbour, ASLB, w/o
DSHood, NRC, w/a (2)
DFJudd, B&W, w/o
[redacted]
FJKelley, Esq, w/o
RBLandsman, NRC Regioa III, w/a
WHMarshall, Esq, w/o
JPMatra, Naval Surface Weapons Center, w/a
WOtto, Army Corps of Engineers, w/a
WDPaton, Esq, w/o
FRinaldi, NRC, w/a
HSingh, Army Corps of Engineers, w/a
BStamiris, w/o

oc1181-0473a100

J Kane
Rec'd 11/4/82

RESPONSE TO THE NRC STAFF REQUEST
FOR ADDITIONAL INFORMATION PERTAINING TO
THE PROPOSED UNDERPINNING OF THE
SERVICE WATER PUMP STRUCTURE

CONSUMERS POWER COMPANY
MIDLAND PLANT UNITS 1 AND 2

MIDLAND PLANT UNITS 1 AND 2

RESPONSE TO THE NRC STAFF REQUEST

FOR ADDITIONAL INFORMATION PERTAINING TO

THE PROPOSED UNDERPINNING OF THE

SERVICE WATER PUMP STRUCTURE

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FIGURES

MIDLAND PLANT UNITS 1 AND 2

RESPONSE TO THE NRC STAFF REQUEST

FOR ADDITIONAL INFORMATION PERTAINING TO

THE PROPOSED UNDERPINNING OF THE

SERVICE WATER PUMP STRUCTURE

1.0 INTRODUCTION

On September 17, 1981, representatives of Consumers Power Company, Bechtel Power Corporation, and the NRC met in Bethesda, Maryland, for a presentation of the proposed remedial action for the Midland plant service water pump structure (SWPS). The discussion of the proposed underpinning construction resulted in several requests for additional information. This report responds to these requests and supplements the Technical Report on the Service Water Pump Structure Underpinning (Reference 1).

2.0 REQUESTS FOR ADDITIONAL INFORMATION

2.1 ASSUMPTIONS AND CONCLUSIONS FOR THE PRELIMINARY ANALYSIS OF THE UNDERPINNED STRUCTURE

2.1.1 Stability Analysis

2.1.1.1 Discussion

The underpinned structure was analyzed for sliding, overturning, and resistance to buoyancy for the design flood condition in conformance with Final Safety Analysis Report (FSAR) Subsection 3.8.6.3.4. Sliding in the north-south direction was critical and overturning was critical in the east-west direction.

The critical load combination for sliding and overturning is:

$$D + H + E'$$

where

D = dead load of structure and equipment

H = lateral earth pressure

E' = safe shutdown earthquake load

2.1.1.2 Assumptions

a. The normal groundwater was assumed at the level of the pond (el 627').

b. The long-term shear strength parameters are $\phi' = 36°$ and $C' = 0.73$ ksf, based on Woodward Clyde Consultants' test data at the SWPS location.

c. The lateral earth pressure dynamic increment was obtained by using FSAR Figure 2.5-45.

d. The forces from the safe shutdown earthquake (SSE) were increased by 50% to provide for a possible increase in this requirement.

e. Because of the flexibility of the underpinning wall, only the side walls and approximately 25% of the north underpinning wall are considered effective in resisting the force that attempts to cause sliding. The validity of this assumption will be verified in the final analysis.

2.1.1.3 Conclusions

The minimum factor of safety against sliding is 1.17 and is based on a sliding force of 16,500 kips and a total resistance of 19,200 kips. This figure is calculated for sliding in the north-south direction and exceeds the allowable factor of safety of 1.1.

The minimum factor of safety against overturning is 1.45 versus an allowable factor of safety of 1.1. This value is based on an overturning moment of 1.9×10^6 ft-kips compared to a stabilizing moment of 2.75×10^6 ft-kips. The east-west direction is the critical direction for overturning.

The building has a factor of safety of 2.1 versus the required 1.1 against the buoyancy force for a flood level of el 631. The building has a total dead weight of 42,000 kips and a buoyancy force of 20,000 kips.

2.1.2 Lower Foundation Slab

2.1.2.1 Discussion

The lower foundation slab is 90 feet long, 74 feet wide, and 5 feet thick and forms the base for the SWPS sump. Interior walls divide the foundation into three slabs: two small slabs 45 feet by 30 feet with effective span lengths of 38 feet, 9 inches by 25 feet, 9 inches and a large slab 90 feet by 44 feet with effective span lengths of 79 feet, 6 inches by 30 feet, 6 inches. The large slab was judged most critical and was analyzed for the following load combinations:

$$U = 1.4D + 1.7L + P_L$$

$$U = 1.4D + 1.7L + 1.4P_L$$

$$U = D + L + P_L + E'$$

$$U = 1.25\ (D + L + P_L + E)$$

where

U = required strength to resist design loads or their related internal moments and forces

D = dead load of the structure and equipment

L = conventional floor and roof live loads (includes movable equipment loads or other loads which vary in intensity)

P_L = load on structure due to jacking

E' = SSE load

E = operating basis earthquake

2.1.2.2 Assumptions

a. The groundwater was assumed at the level of the pond (el 627').

b. The plant fill under the upper foundation slab offers no vertical support for the upper slab.

c. The effects of dead load, live load, and jacking load are carried only by the lower foundation slab. All other loads are transferred to the foundation composed of the lower slab and the underpinning wall.

2.1.2.3 Conclusions

The maximum imposed out-of-plane moment of 180 ft-kips was exceeded by the moment capacity of the slab, which amounts to 200 ft-kips. The maximum soil pressure was 11.3 ksf.

2.1.3 <u>Effect of Construction Dewatering on the Lower Foundation Slab</u>

2.1.3.1 Discussion

Fluctuations of the water table will affect the values of the soil pressures under the foundation slab. The drawdown of the groundwater for constructing the underpinning wall will decrease the buoyancy of the structure, causing an increase in bearing pressure.

2.1.3.2 Assumptions

a. The original groundwater is assumed at the level of the pond (el 627').

b. The groundwater will be drawn down to el 587' at the north underpinning wall.

c. The shape of the drawdown curve is parabolic.

d. The drawdown is uniform for the full width of the structure.

2.1.3.3 Conclusions

Considering dead load, live load, and buoyancy, and the assumed groundwater at el 627'-0," the bearing pressure under the slab varies with a maximum value of 5.35 ksf at the north edge. For the construction condition, dewatering to el 587'-0", this pressure increases to 8.12 ksf, which is well below the allowable pressure of 16.7 ksf. This pressure, 8.12 ksf, will be reduced as the construction of the underpinning wall proceeds because the addition of jacking forces reduces the weight of the structure supported by the lower base slab.

The pressures from the underpinning construction condition are less than the values used in Subsection 2.1.2 of this report and are not considered critical in analyzing the slab.

2.1.4 <u>Upper Foundation Slab</u>

2.1.4.1 Discussion

The slab is 86 feet long, 38 feet wide, and 3 feet thick. An interior wall divides the slab into two slabs of unequal size. The smaller slabs are 38 feet by 35 feet and 51 feet by 38 feet. The larger slab, with effective span dimensions of 48 feet, 3 inches by 25 feet, 4 inches, was analyzed for the following load combination, which included the effects of compartment flooding to a depth of 12.5 feet.

$$U = 1.0D + 1.0L + 1.0E' + 1.0T_o + 1.25H_o + 1.0R + P_L$$

2.1.4.2 Assumptions

a. The fill under the upper foundation slab offers no vertical support. The slab is simply supported on four sides but is continuous over the interior wall.

b. The seismic effects and the containment of water to a depth of 12.5 feet does not occur simultaneously.

2.1.4.3 Conclusions

The maximum imposed moment of 109 ft-kips (from the analysis) is less than the slab capacity of 150 ft-kips. Therefore, the slab is considered to be adequate.

2.1.5 <u>Sidewalls of the Overhang</u>

2.1.5.1 Discussion

The exterior walls at the face of the overhang were analyzed for shear and bending stress for the load combination of:

$$U = D + L + E' + P_L$$

2.1.5.2 Assumptions

a. The groundwater was assumed at the level of the pond (el 627').

b. The fill under the upper foundation slab offers no support.

c. The resisting section at the face of the overhang consists of a box section and the attached underpinning walls. The box section is composed of the exterior walls of the overhang, the roof slab, and the foundation slab. The support offered by the interior walls was ignored. The resisting section was modified for the effects of shear lag.

2.1.5.3 Conclusions

The maximum computed compressive stress in the walls was 0.32 ksi and the maximum shear stress is 0.103 ksi. The largest tensile stress in the reinforcement is 2.2 ksi. All values are below the American Concrete Institute (ACI) 318-71 allowable values.

2.1.6 Interface Connectors

2.1.6.1 Discussion

The underpinning walls are designed to act as integral parts of the structure. Application of jacking loads and the use of anchor bolts will ensure that loads are adequately transferred between the structure and the underpinning walls. Rock bolts and anchor bolt assemblies will be used to ensure that the walls and structure do not separate. Because the construction procedure requires that the anchor bolts and rock anchors be installed after the application of the jacking loads, the connectors are not affected by the jacking operation or the dead load of the structure.

2.1.6.2 Assumptions

a. The connectors will be designed to carry all loads on the structure, except the jacking loads.

b. The behavior of the connection is governed by shear friction requirements.

c. The connectors were designed for the following load combinations:

$$U = 1.4D + 1.7L + P_L$$

$$U = D + L + E' + P_L$$

2.1.6.3 Conclusions

The maximum shear load to be transferred at each vertical interface is 1,300 kips. Nine 2-inch diameter, hollow core rock anchors at a maximum spacing of 3 feet, 9 inches are required to fulfill the shear friction requirements. A maximum shear of 1,700 kips will be transferred at the horizontal interface by 2-3/4-inch diameter anchor bolts at a maximum spacing of 3 feet, 9 inches.

2.1.7 Underpinning Wall

2.1.7.1 Discussion

The underpinning wall extends from the underside of the upper foundation to firm bearing on undisturbed soil. The wall is 4 feet thick and 30 feet high. The base of the north wall is widened to 6 feet. The wall is connected to the existing structure with rock and anchor bolts.

The wall was analyzed for the following load combination:

$$U = D + L + E' + P_L$$

2.1.7.2 Assumptions

a. The wall was analyzed as a shear wall for in-plane forces.

b. Because the north wall has a horizontal span length of approximately 86 feet, the wall at midlength was analyzed as a vertical simply supported beam and was also analyzed with partial restraint at the base for out-of-plane forces.

2.1.7.3 Conclusions

For in-plane forces, each side wall carries a moment of 50,000 ft-kips and a shear of 400 kips. The capacity of the wall is 75,000 ft-kips for moment and 1,000 kips for shear. Because the aspect ratio of the north wall is much more favorable, it was considered not critical in the preliminary analysis. The analysis of the north wall for out-of-plane forces showed the maximum moment to be 150 ft-kips per foot of wall, which is less than the 190 ft-kip moment capacity. Shear was not critical.

3.0 <u>DESCRIPTION OF PROTECTION FOR THE EXISTING STRUCTURE DURING CONSTRUCTION</u>

3.1 CONSTRUCTION PROCEDURE (Refer to Figure 4 of Reference 1)

Protecting the existing structure while constructing the underpinning wall is a major concern. This concern is reflected in the procedure that was established for constructing the underpinning. This procedure was developed with the purpose of providing the maximum degree of safety to the structure.

As a precautionary measure, the upper portion of the north-south exterior walls will be post-tensioned before the permanent dewatering begins. The dewatering will reduce the buoyancy force acting on the overhang and will increase bending stresses in the walls. Post-tensioning the upper portion of the exterior walls will induce compression in the walls and will minimize the effects of the tensile forces caused by dewatering.

The first three piers, which are located at the northwest and northeast corners of the structure, will be constructed from tunnels proceeding simultaneously from the access shafts at the east and west sides of the building. In this way, the jacking force will be symmetrically applied to the structure. The construction procedures prevent advancing either tunnel to the area where the next pier is to be constructed until the jacking load is placed on the completed pier. Thus, the decrease in soil support of the upper foundation slab is kept to a minimum.

After the corner piers are in place, the construction procedures call for the installation of the center piers under the north wall. This requires advancing the tunnel approximately 25 feet to the next pier. To prevent excessive loss of support, the following provisions will be made.

3.1.1 Only one tunnel will be extended from the pier 3 to pier 4 location at one time. When the first pier 4 and pier 5 are load bearing, the other tunnel will be extended to the remaining pier 4.

3.1.2 Measurement devices will be provided at piers 1, 2, and 3 to monitor variations in applied loads to the piers. If a sudden increase in pier loading of the magnitude of approximately one-third is indicated while the tunnel is being advanced from pier 3 to pier 4, tunnel construction will be stopped. Pier 8 will then be constructed as a series of piers instead of as a large monolithic pier. This procedure will provide a gradual increase in the jacking support to the overhang as the tunnel is advanced to pier 4.

3.1.3 When the tunneling operation toward the center begins, the three piers on each end will have a total jacked load of 465 kips. This results in an average bearing pressure of 5.8 ksf in the till. The till is considered adequate for an allowable bearing intensity of 19.2 ksf at a safety factor of 2.5 against bearing failure. These figures indicate that a total allowable bearing load of 1,600 kips for each pier group is available to adequately support the overhang portion of the structure. The north wall is adequate at ACI-acceptable stresses to span between the end pier groups if necessary. Analysis of the north wall for this condition, considering the wall as a deep concrete beam and assuming no vertical soil support to the overhang, shows that the compressive stress amounts to 0.250 ksi and tension in the concrete amounts to 0.300 ksi which is less than the modulus of rupture, 0.475 ksi.

3.2 CRACK MONITORING

In anticipation of the underpinning wall construction, a crack mapping program has been started. Existing crack locations and widths have been accurately measured. Future mappings, to monitor the existing cracks and the appearance of new cracks, are scheduled to take place before and after major underpinning construction procedures, such as post-tensioning, dewatering, and jacking.

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 or changes in existing structural cracks exceeding 0.01 inch will be evaluated and if any crack widths reach 0.03 inch, construction in the affected area will be modified or suspended until the reasons for excessive cracking are established and appropriate remedial measures are implemented.

3.3 SETTLEMENT MONITORING

In addition to the crack monitoring program, a program to closely monitor structure settlement has been planned. Besides the four existing settlement markers at each corner of the building, five additional markers will be installed on the building (Refer to Figure 1) and a settlement dial indicator will be installed at each of the two building corners where the underpinning will be constructed. The dial indicators will be attached to the building with their probes connected to permanent bench marks founded in undisturbed soil approximately 50 feet below the bottom of the underpinning wall. The depth at which the tip of the bench mark is located ensures that the bench mark movement will be negligible. The settlement markers will be monitored before and after major construction procedures as discussed in Section 3.2. Building movement and crack data will enable the project engineer to evaluate the effects of the underpinning construction on the existing structure.

4.0 DISCUSS THE BEARING CAPACITY OF THE UNDISTURBED NATURAL SOIL SUPPORTING THE UNDERPINNING

The estimated, ultimate bearing capacity is based on the many borings taken in the area by Dames and Moore and others including the recent borings taken by Woodward-Clyde Consultants. The soil samples and laboratory analysis of the most recent borings indicate the soil has shear strength conservatively estimated at 8 ksf and an ultimate bearing capacity of 48 ksf.

5.0 EVALUATE THE DIFFERENTIAL SETTLEMENT BETWEEN THE MAIN PART OF THE STRUCTURE AND THE UNDERPINNED PORTION

The construction procedure requires that jacking loads be applied to the piers soon after the pier is constructed. This load is sustained for sufficient time to dissipate the major portion of the long-term settlement of the underpinning. The underpinning is not attached to the structure until after the settlement has taken place.

is a rotational deformation

Variations in deformations over the entire foundation, assuming a flexible structure, are predicted to be on the order of 0.2 inch. Soil springs are being developed to reflect total deformations including variations. The structure will be modeled and analyzed with the resulting supporting springs. In the soil-structure system modeling, the rigidity of the structure is considered. The interaction of the flexible springs and rigid structure reflects the true behavior of the structure.

changed to 0.3"

6.0 DESCRIPTION OF PROCEDURE FOR TIMING OF FINAL JACKING LOCK OFF

6.1 METHODOLOGY

The final jacking loads will not be locked off until it is determined that the major portion of the pier settlement has occurred. By comparing predicted concrete and soil behavior curves and instrumented observations of the pier deflections, the optimum time for locking off the jacking load will be determined.

Vertical deflections at the top of the underpinning piers will result from the summation of several time-related properties of the pier concrete and the underlying soil. During the underpinning work, the soil deflection will be monitored at the top of each pier by connecting a settlement indicator to the top of a rod that extends to a plate at the base of the pier (refer to Section D-D of Figure 5, Reference 1). The rod is greased and placed within a tube to separate it from the concrete. The total top of pier deflections will be measured by another settlement indicator on top of the pier. The difference between these two deflection readings will represent the behavior of the concrete in the pier and the supporting soil.

The monitored pier deflections will be compared to predicted values. The expected concrete behavior is based on observations reported in recognized engineering standards. Four deflection curves for the pier concrete and glacial till are shown in Figures 2 through 5. The curves are plotted as displacement versus the logarithim of time. Figure 2 depicts a plot of the predicted top of pier deflection due to the creep of concrete under compressive load. As indicated, the total deflection will amount to approximately 0.03 inch. Figure 3 plots the top of pier deflection due to concrete shrinkage as the concrete dries and cures. The 10,000-day line is equal to about 27 years of elapsed time after pier construction. As shown in Figure 3, the total shrinkage-caused pier deflection is estimated at about 0.2 inch with the deflection leveling off after approximately 90 days. Figure 4 is a plot of the anticipated top of pier deflection due to soil consolidation. This graph indicates the settlement within a minimum and maximum range of values. The indicated total settlement due to soil consolidation is expected to be between 0.4 and 0.5 inch.

By combining the curves of predicted pier deflection due to concrete behavior, as shown in Figures 2 and 3, and the soil deflection curve shown in Figure 4, a composite top-of-pier-deflection-versus-log-time curve can be drawn. This is shown in Figure 5 using the maximum predicted soil settlement. The initial jacking of Stage 1 load (as shown in Figure 4 of Reference 1) into the pier several days after concrete placement will result in early rapid deflection, as shown. After about 90 days of Stage 1 loading, the jacking load will be increased to the final level which will result in another, but smaller, dip in the deflection curve. This increase in jacking load will combine with the shrinkage effect, which is greatest between 10 and 90 days' time. At about 110 days, the curve will flatten so it will appear as a straight line on this semi-log plotting. On a linear time scale, the deflection rate would appear much flatter. This semi-log straight line prediction is a typical observation for soil reaction after an initial elastic reaction period and is based on numerous test observations in the laboratory, as well as long-term field observations on in-place structures and buildings. The key factor in the process of final jacking and locking-off is determining when this more predictable phase has begun. This will be done at the site by plotting deflection curves, both at the top and bottom of the piers, while maintaining the final jacked loadings. This phase of the settlement curve is anticipated to occur soon after the final load level is applied assuming that all pier concrete is more than 90 days old.

6.2 ACCEPTANCE CRITERIA

The final jacking load will total 4,400 kips and will be imposed on underpinning piers 1 through 10. At that time, all piers will be at least 90 days old. This load level will be maintained for a period of about 2 weeks or until the settlement rate is within acceptable limits. The previous plottings of pier deflections under load will form a performance record which will greatly influence the determination of final acceptance and locking off.

7.0 DISCUSSION OF THE VALIDITY AND USE OF THE PENETROMETER

To aid the geotechnical engineer in assessing the adequacy of bearing capacity of the soil under the base of each underpinning pier, the construction procedures specify the use of the Waterway Experimental Station cone penetrometer, Model CN-973. The penetrometer consists of a 30° cone with a 1/2-square inch base, an 18-inch extension rod, a proving ring, a dial indicator, and a handle. A force applied through the handle deforms the proving ring and forces the cone to penetrate the soil. The proving ring deformation is proportional to the force applied, and the value of the applied force is indicated on the dial. The force is an index of the shearing resistance of the soil.

To evaluate the allowable bearing capacity of the soil, a family of curves relating allowable bearing capacity to applied force and cone penetration is utilized. These curves are based on the work of G.G. Meyerhof (Reference 2).

8.0 DESCRIPTION OF THE CRITERIA FOR FAILURE OF THE SOIL RESULTING FROM JACKING LOADS

Deflection at the bottom of an underpinning pier which approaches 2 inches is at about 90% of the point at which soil indicates plastic behavior. Other time-versus-rate-of-deflection criteria which are useful are that soil deflection should slow to about 0.01 inch in 3 hours after 3 days of constant load, and 0.02 inch for the interval between 10 and 20 days under constant load.

9.0 DESCRIBE THE PROCEDURE FOR MONITORING GROUNDWATER LEVELS DURING CONSTRUCTION OF THE UNDERPINNING WALL

As part of the temporary dewatering procedure, piezometers will be installed to monitor the groundwater level. Before the access shafts are excavated, a piezometer will be installed adjacent to each shaft. While constructing the tunnel under the north wall of the structure, three piezometers will be installed: one at each end and one at mid-length. When the tunnel is completed, a monitoring system of five piezometers will have been installed. If required, additional piezometers will be installed as the tunnels under the side walls are advanced.

10.0 COMMENT ON BORING CH-2 SHOWING FILL MATERIAL BELOW EL 587.0

The log for Boring CH-2 indicates silty sand to el 583'-8". From the results of other nearby borings and the general excavation plan for the site, it is believed that the predominant soil type is sandy clay till. If this is borne out during pit excavation and the till is compact and well bound, it will be acceptable for bearing at el 587. This acceptance would be based on the judgement of the geotechnical engineer using qualitative criteria, such as taking soil samples for strength analysis. On the other hand, if the till is not compact and well bound, or if it is silty sand, the material will be excavated to adequate till and replaced to el 587' with lean concrete on a pit-by-pit basis.

11.0 EVALUATION OF SOIL SPRINGS VALUES - STATIC AND DYNAMIC LOADING CONDITIONS

The soil springs are presently being evaluated as part of the final analysis of the structure. When this evaluation is completed, the requested information will be submitted.

REFERENCES

1. Consumers Power Company, Technical Report on the Service Water Pump Structure Underpinning, August 26, 1981

2. G.G. Meyerhof, "The Ultimate Capacity of Wedge-Shaped Foundations," Proceedings of the 5th International Conference on Soil Mechanics and Foundations, Paris, 1961

SERVICE WATER PUMP STRUCTURE
SETTLEMENT MARKER LOCATIONS

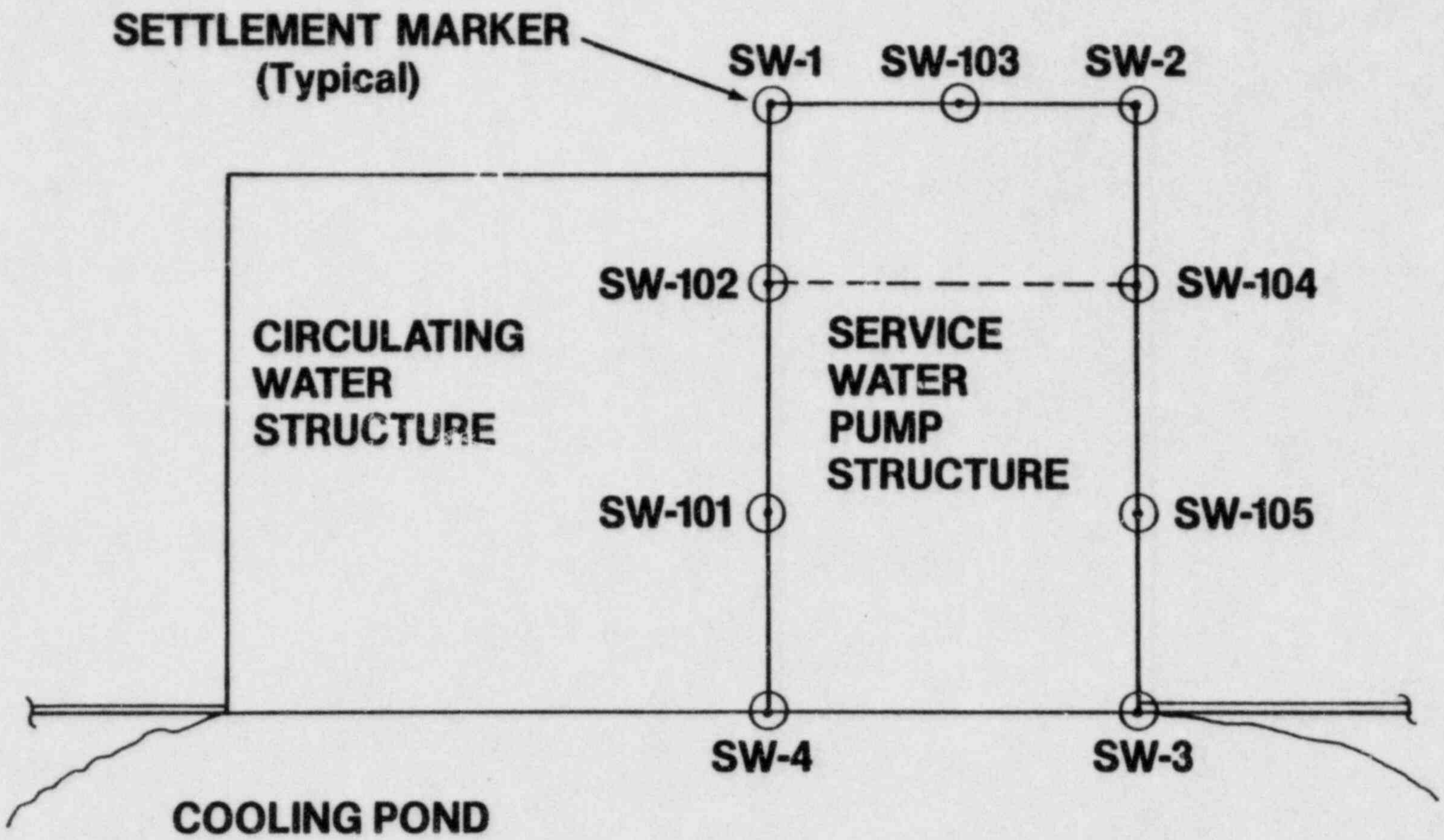

FIGURE 1

SERVICE WATER PUMP STRUCTURE
ESTIMATED TOP OF PIER DEFLECTION DUE TO CREEP OF CONCRETE VS TIME

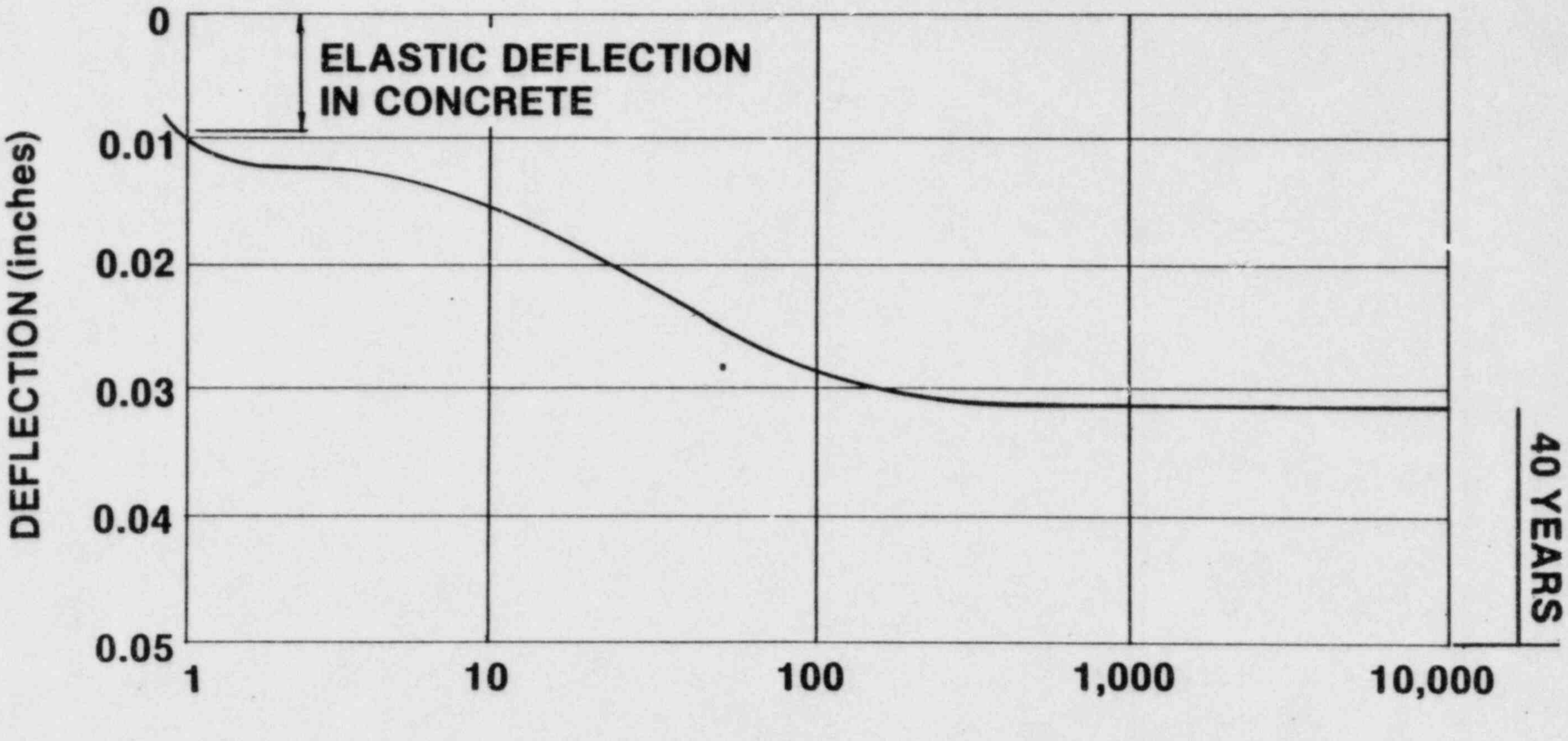

FIGURE 2

SERVICE WATER PUMP STRUCTURE

ESTIMATED TOP OF PIER DEFLECTION DUE TO SHRINKAGE OF CONCRETE VS TIME

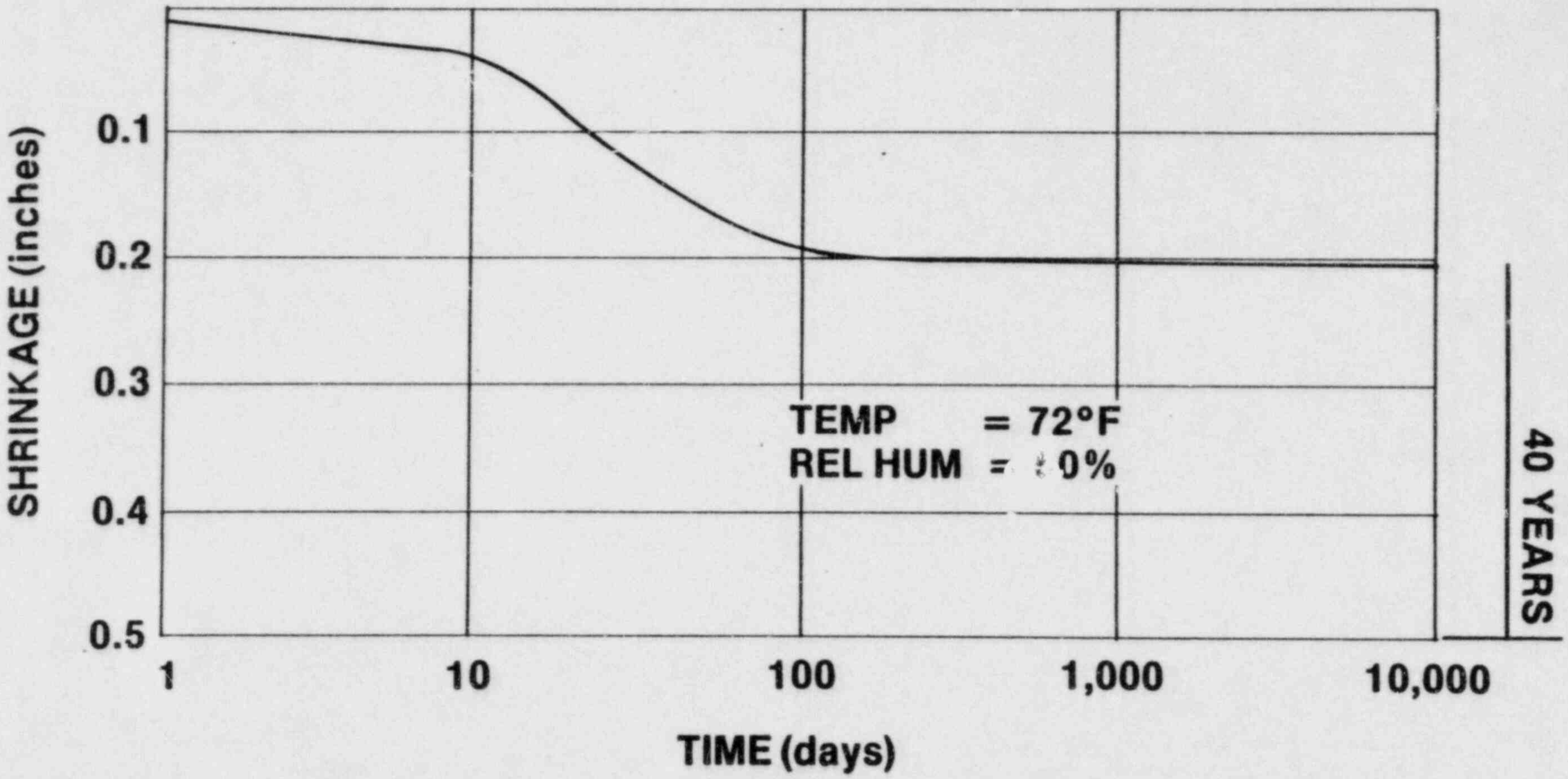

FIGURE 3

G-1584-11

SERVICE WATER PUMP STRUCTURE
ESTIMATED TOP OF PIER DEFLECTION DUE TO CONSOLIDATION OF SOIL VS TIME

(Time Is Measured from Start of Jacking)

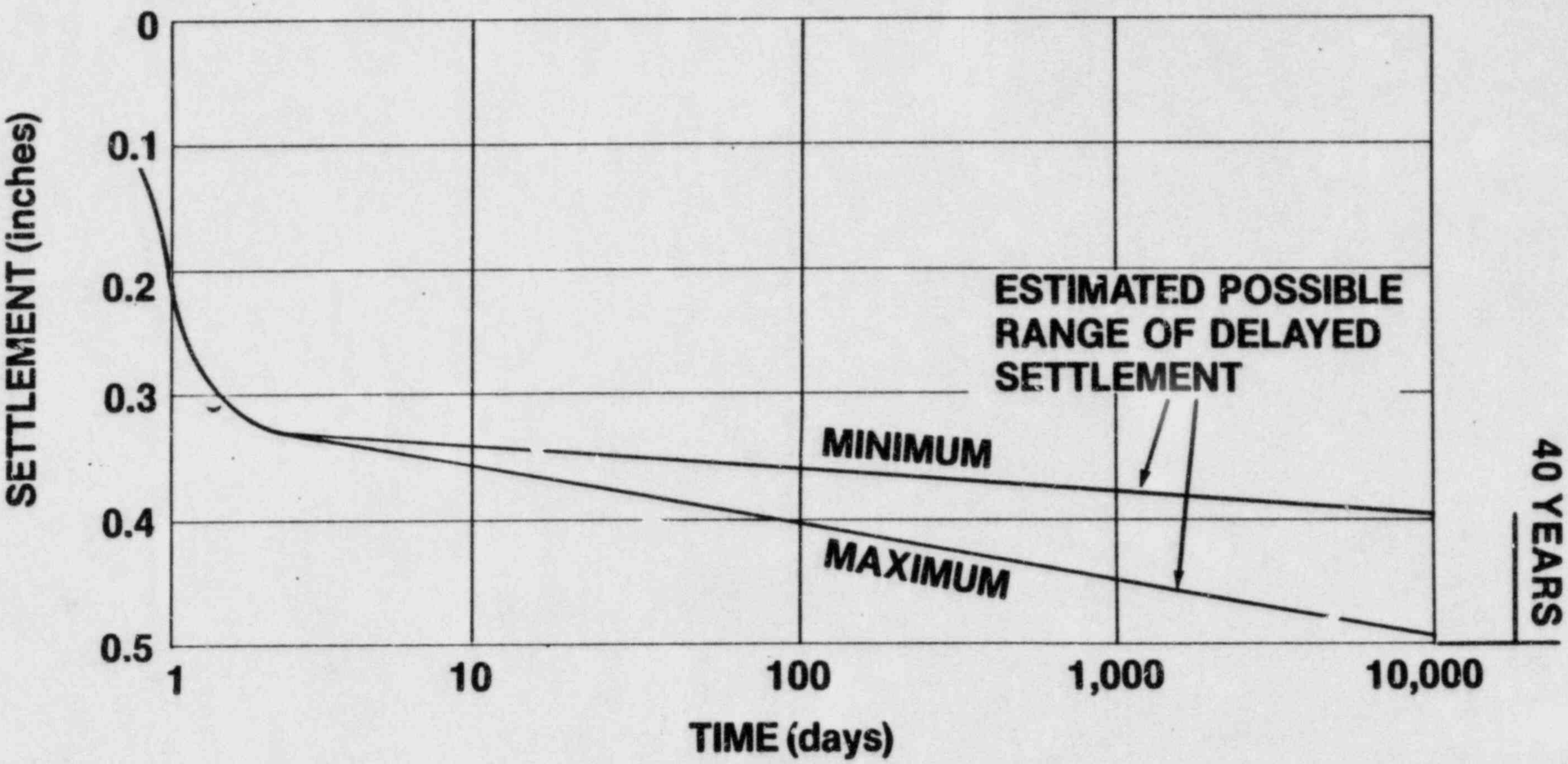

FIGURE 4

G-1584-14

SERVICE WATER PUMP STRUCTURE
ESTIMATED TOP OF PIER DEFLECTION DUE TO TOTAL DEFORMATION VS TIME

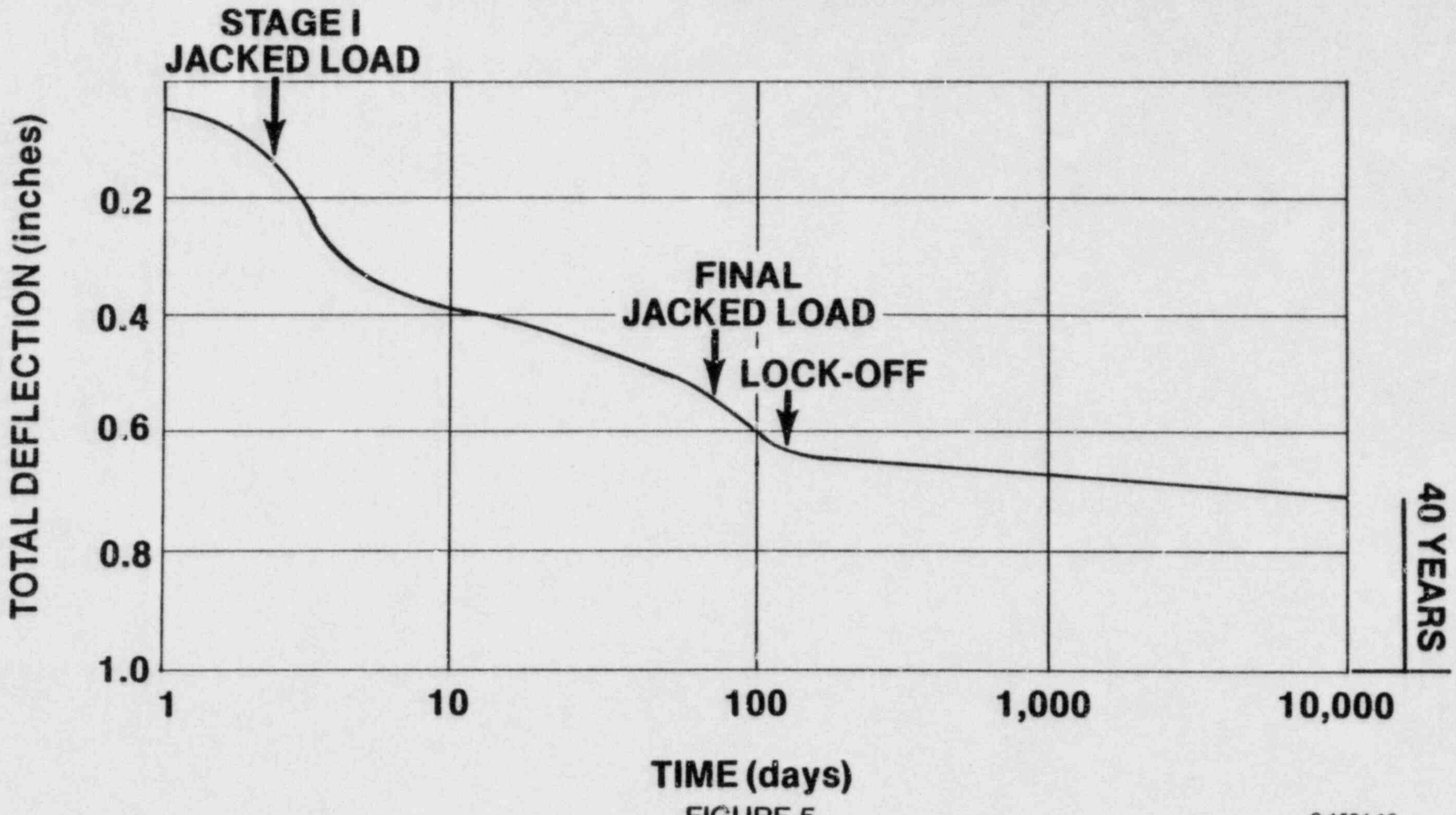

FIGURE 5

G-1584-12