This contains some important changes made to 6/11/82 draft which were made while I was on annual leave JDK

J. Kane

19/B5

UNITED STATES
NUCLEAR REGULATORY COMMISSION
WASHINGTON, D. C. 20555

JUN 16 1982

Docket Nos. 50-329/330

MEMORANDUM FOR: Elinor Adensam, Chief
Licensing Branch #4
Division of Licensing

FROM: George Lear, Chief
Hydrologic and Geotechnical Engineering Branch
Division of Engineering

SUBJECT: QUESTIONS, POSITIONS AND REVIEW COMMENTS ON CONSUMERS APRIL 22, 1982 SUBMITTAL

Plant Name: Midland Plant, Units 1 and 2
Licensing Stage: OL
Responsible Branch: Licensing Branch #4, D. Hood, R. Hernan, LPM's
Review Status: Continuing

We have enclosed the review comments of the GES staff and its Consultants on the April 22, 1982 submittal by the Applicant concerning the Borated Water Storage Tanks and Service WaterPump Structure underpinning work. (Letter to H. Denton from J. W. Cook dated April 22, 1982).

The enclosure indicates the staff's two major areas of difference with the Applicant are (1) having measured vertical deflections (differential settlements) control the underpinning work rather than an unreasonably large strain criterion and (2) the Applicant's apparent reluctance to fully comply with the staff's past recommendations on construction dewatering that were previously transmitted in the April 2, 1982 letter from R. Tedesco to J. Cook on construction dewatering for the Service Water Pump Structure underpinning work

We recommend contact with the Applicant to resolve these differences following a reasonable period for Consumers and its consultant to review the enclosed evaluation.

This input was prepared by Joseph Kane.

Lyman W. Heller
for George Lear, Chief
Hydrologic and Geotechnical
Engineering Branch
Division of Engineering

Enclosure:
As stated

cc: See next page

8206240401

XA 2 pp.

cc w/o encl:
R. Vollmer

cc w/encl:
J. Knight
G. Lear
F. Schauer
L. Heller
M. Fliegel
R. Gonzales
D. Hood
R. Hernan
H. Singh, COE
S. Poulos, GEI
J. Kane

Midland Plant, Units 1 and 2
Docket Numbers: 50-329/330
Subject: Geotechnical Engineering Evaluation of Reference 1
Prepared by: Joseph D. Kane, HGEB, DE, NRR
Reference 1: April 22, 1982 letter from J. W. Cook to H. R. Denton on Responses to the NRC Staff Request for Additional Information Required for Completion of Staff Review of the Borated Water Storage Tank and Underpinning of the Service Water Pump Structure

The following comments and questions are based on the reviews of Reference 1 by the Geotechnical Engineering Section Staff, HGEB, DE and its consultants, Dr. S. Poulos, Geotechnical Engineers, Inc. and H. Singh, U. S. Army Corps of Engineers.

Q.1. (Issue 1, Page 2, Par. 3) Provide the range in layer thicknesses that the oil-impregnated sand will be placed beneath BWST IT-60 tank and the construction controls to be required for its placement and compaction.

Q.2 (Issue 2, Page 3, Par. 2) Averaging the strain over a 20-foot gage length is not acceptable to the Staff because this averaging could lead to underestimating stresses and unacceptable cracking. Installing shorter length gages over the 20-foot length is recommended. The Staff's concern with the single 20-foot gage length is further discussed in Q.5.

Q.3 (Issue 2, Page 3, Par. 3) As a minimum, the BWST ring beams should be monitored for increasing strains at a frequency of at least once a year, following the initial 5 year period of plant operation.

Q.4 (Issues 1 and 2, Pages 5 and 6). The Applicant's responses to issues 1 and 2 are inadequate with respect to the basis for adopting the soil spring stiffness of 4,000 KCF and with respect to determining the effects of differential settlement on the existing SWPS. The Applicant should either justify the adoption of the soil spring stiffness value of 4000 KCF or alternately use a stiffness of K = 400 KCF for the glacial till, a value which is considered reasonable and acceptable to the Staff and its consultants.

Q.5 (Issue 3, Page 6). The proposed 5/16-inch displacement (extension) criterion over a 20-foot gage length is not acceptable to the Staff or its consultants. More gages of shorter lengths would be preferable to permit identification of the more highly stressed sections. The Staff and its consultants recognize the advantages of the proposed strain monitoring program but consider measurement of the vertical differential settlement, similar to what is being carried out for the Auxiliary Building underpinning work, to be the more positive and sensitive construction control that would permit corrective action to be taken before overstressing the SWPS would occur. For these reasons the Staff requires that underpinning of the SWPS be controlled by monitoring of vertical differential settlement to tolerable limits established before starting this work.

Q.6 (Issue 6 Page 7). The Applicant's response to issue 6 does not provide the calculations for sliding resistance of the SWPS under seismic loading which were requested at the March 16 through 19, 1982 design audit. For this reason Item 2.2 of Enclosure 8 to the May 25, 1982 letter from D. G. Eisenhut to J. W. Cook again requests this information.

Q.7 (Issue 13, Pages 10-12). The following changes and additions should be made to the Applicant's response to issue 13.

a. On 5th line, Page 10, the word "solely" should be deleted.

b. On 2nd line, Par. 3, Page 11, the word "generally" should be deleted. At the end of this paragraph add the following: The correlation between the pier or plate load test results and the penetration tests performed on the foundation soils will be used to correct the correlation graphs and to judge the suitability of the bearing stratum.

c. Last paragraph, Page 11, should be revised to incorporate the following changes. The zone of influence should be defined by extending lines downward at a slope of 1 horizontal (H) to 1 vertical (V) from the edge of the footing into the foundation
braced
soils. If the foundation soil is cohesionless, a braded excavation is required if the excavation must proceed more than 6-inches below the adjacent pier or, if not an immediately adjacent pier, then 6-inches below the intersection of the pier footing with the 1H to 1V zone of influence slope. Movements of adjacent piers shall be monitored as the excavation proceeds to 18-inches or less. Excavations shall be stopped and construction procedures modified if measured movements are larger than anticipated.

Q.8 (Issue 14, Page 12). The modifications and additions which were required for the pier load test procedures for the Auxiliary Building (Enclosure 2 to the May 25, 1982 letter from D. G. Eisenhut to J. W. Cook, Par. 4) are also required in the procedures for the Service Water Pump Structure. In addition, if the very dense sandy alluvium is ultimately accepted as the foundation for a portion of the SWPS underpinning piers, then either a pier or a plate load test should also be conducted on this foundation material.

Q.9 (Issue 18, Pages 13-15). The following comments and questions are numbered in identical order to the numbering of the contingency plan items given in response to issue 18:

1.c. What procedure is to be followed that will permit a single well failure to be identified from the total system?

2.b. It is unclear what level will be equalized and the time it will take to complete this action. What occurrence (e.g., settlement measurement, etc.) triggers this reaction to uncontrolled groundwater flow?

3.a and 3.b. Is the equipment for carrying out techniques such as forepooling or speeling or grouting to stop ground loss in readiness at the plant site? If not, what time frame is required to make it available?

4.a. Include limits on maximum depth of excavation and zone of influence and requirements for bracing.

4.b. A required increase in bearing area of underpinning piers is a significant change that requires notification of Region III.

5. Recording of excessive pier settlement requires an evaluation of its cause and notification of Region III before proceeding with other piers.

6. The use of wedges and plates would be the routine method to stop movement in the event of a jack failure.

7. A loss in functioning of the important northerly benchmarks would require underpinning work to be stopped until the benchmarks were restored and elevations confirmed.

8. Prior to implementing the listed items of 8a, 8b and 8c the underpinning work should be stopped and the existing excavation faces carefully supported.

The contingency plan should be revised to incorporate the above Staff's comments and Applicant's responses.

Q.10 (Issue 19, Pages 15-16). The following comments should be incorporated into the notes controlling the checking or adjusting of jacking loads.

Jacking will be controlled to limit settlements to acceptance criteria values identified on SWPS-14 (Still to be established by the Applicant and evaluated by the Staff). Wedges and plates will be used to prevent unacceptable movement in the event of a jack failure, both during pier construction and during application of final jacking loads.

During the construction of Piers 1, 2 and 3 the jacks will be monitored at least at the start of every shift and daily during holidays and weekends. More frequent checking and jacking is required until the rate of load decrease is small enough and sufficiently stabilized to permit checking once during each shift.

Q.11 (Issue 20, Page 16). The above comments on jacking control and monitoring frequency are applicable to the transfer of initial to final ~~the~~ jacking load ~~into the permanent underpinning wall~~. Provide the actual value of the "predetermined rate".

Q.12 (Issue 24, Page 19). It is unclear from the Applicant's response whether or not Consumers intends to comply with the Staff's recommendation (April 2, 1982 letter from R. Tedesco to J. Cook, "Staff Concurrence for Installation and Operation of Construction Dewatering and Observation Wells for the Service Water Pump Structure", Enclosure 3, Page 4) to require extension of the six previously proposed piezometers to at least elevation 570. The Staff does not have a problem if the Applicant chooses to add piezometers to the original six and terminate these additional piezometers at "an elevation no lower than approximately 1 foot above the undisturbed natural soil." However, the Staff still requires that the bottom elevation of the original six piezometers be drilled to at least elevation 570.

The Staff does not accept the Applicant's statements on controlling the groundwater level in the SWPS area during underpinning construction for the following reasons:

a. Drawing the water level down to approximately the interface of the fill and natural soil is not a realistic control. Completed borings show this interface surface and soil conditions to be highly variable in the immediate area of the underpinning work with the interface level ranging from Elevation 605 to Elevation 583.

b. Identification of the soil type at the bottom of the dewatering well does not provide assurances that blow outs will not occur at the base of pier excavations because this information does not address the problem of pervious layer stratification and continuity and impervious layers of insufficient thickness.

For the above reasons, the Staff reiterates its position that there should be a control on the upper phreatic surface which requires a minimum 2-foot depth between the upper phreatic surface being controlled by dewatering and the bottom of any underpinning excavation at any given time. As a minimum, the six originally proposed piezometer locations are to be used to verify that the groundwater is being maintained to this level during underpinning. It is recognized that localized temporary dewatering techniques such as sumping may be necessary to produce hydrostatically relieved conditions in areas of entrapped water.

Q.13 (Fig. SWPS-14). A correction to Note 9 is needed to indicate that all instrumentation and material identified in the Monitoring Matrix is to be Q-listed unless otherwise shown not to be required. A separate request of the Applicant to provide the following drawings identified on Fig. SWPS-14 has been made.

Drawing Nos.	Subject
C-2040 thru C-2043-11	Crack Monitoring Requirements
C-2003 and C-2004	Building Settlement Monitoring Requirements
C-2035 and C-2036	Details of Wall and Pier Settlement Monitoring

DRAFT 6/8/82

Final

UNITED STATES
NUCLEAR REGULATORY COMMISSION
WASHINGTON, D. C. 20555

ACTION

Docket Nos. 50-329/330

MEMORANDUM FOR: Elinor Adensam, Chief Licensing Branch # 4, ~~Robert L. Tedesco, Assistant Director for Licensing~~
Division of Licensing

~~THRU: James P. Knight, Assistant Director for Components and Structures Engineering Division of Engineering~~

FROM: George Lear, Chief
Hydrologic and Geotechnical Engineering Branch
Division of Engineering

SUBJECT: QUESTIONS, POSITIONS AND ~~GEOTECHNICAL ENGINEERING~~ REVIEW COMMENTS ON CONSUMERS APRIL 22, 1982 SUBMITTAL

Plant Name: Midland Plant, Units 1 and 2
Licensing Stage: OL
Responsible Branch: Licensing Branch No. 4, D. Hood, R. Hernan LPM's.
~~Requested Completion Date: Not Scheduled~~
Review Status: Continuing

We have enclosed the review comments of the GES staff and its Consultants on the April 22, 1982 submittal by the Applicant concerning the Borated Water Storage Tanks and Service Water Pump Structure underpinning work. (Letter to H. R. Denton from J. W. Cook dated April 22, 1982).

The enclosure indicates the staff's two major areas of difference with the Applicant are (1) having measured vertical deflections (differential settlements) control the underpinning work rather than an unreasonably large strain criterion and (2) the Applicant's apparent reluctance to fully comply with the staff's past recommendations on construction dewatering that were previously transmitted in the April 2, 1982 letter from R. Tedesco to J. Cook on construction dewatering for the Service Water Pump Structure underpinning work.

We recommend contact ~~that a conference call be arranged~~ with the Applicant to resolve these differences following a reasonable period for Consumers and its consultant to review the enclosed evaluation.

This input was prepared by Joseph Kane ~~and has been coordinated with SEB.~~ ~~We anticipate that SEB will provide both HGEB and DL with any comments they may have in areas of overlapping concern.~~

Knight told me this is a no-no. i.e. he's telling for Tedesco of our coordination problems. 6/15/82

"This is one of the reasons" it hasn't sent this forward - even by 6/15/82.

Lyman W. Heller
George Lear, Chief
Hydrologic and Geotechnical
Engineering Branch
Division of Engineering

Enclosures:
As stated

cc: w/o enclosures
R. Vollmer
~~J. Knight~~

w/enclosures: J. Knight
G. Lear
F. Schauer
~~P. T. Kuo~~
~~F. Rinaldi~~
L. Heller
~~T. Sullivan~~
M. Fliegel
R. Gonzales
D. Hood
R. Hernan
H. Singh, COE
S. Poulos, GEI
J. Kane

Midland Plant, Units 1 and 2
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Subject: Geotechnical Engineering Evaluation of Reference 1
Prepared by: Joseph D. Kane, HGEB, DE, NRR
Reference 1: April 22, 1982 letter from J. W. Cook to H. R. Denton on Response to the NRC Staff Request for Additional Information Required for Completion of Staff Review of the Borated Water Storage Tank and Underpinning of the Service Water Pump Structure

The following comments and questions are based on the reviews of Reference 1 by the Geotechnical Engineering Section Staff, HGEB, DE and its consultants, Dr. S. Poulos, Geotechnical Engineers, Inc. and H. Singh, U.S. Army Corps of Engineers. ~~The Applicant's response to Confirmatory Issues 4, 5, 7, 8, 9, 10, 11, 12, 22 and 23 for the Service Water Pump Structures are related to structural engineering and it is anticipated that review of these responses will be performed by Structural Engineering Branch, DE.~~

Q.1. (Issue 1, Page 2, Par. 3) Provide the range in layer thicknesses that the oil-impregnated sand will be placed beneath BWST IT-60 tank and the construction controls to be required for its placement and compaction.

Q.2. (Issue 2, Page 3, Par. 2) Averaging the strain over a 20-foot gage length is not acceptable to the Staff because this averaging could lead to underestimating stresses and unacceptable cracking. Installing shorter length gages ~~(maximum length of 5 feet)~~ over the 20-foot length is recommended. The Staff's concern with the single 20-foot gage length is further discussed in Q.5.

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Internal action

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Combine with Q2

~~and would result in cracking during underpinning~~. More gages of shorter lengths ~~(e.g., maximum length of 5 foot)~~ would be preferable to permit identification of the more highly stressed sections. The Staff and its consultants recognize the advantages of the proposed strain monitoring program but consider measurement of the vertical differential settlement, similar to what is being carried out for the Auxiliary Building underpinning work, to be the more positive and sensitive construction control that would permit corrective action to be taken before overstressing the SWPS would occur. For these reasons the Staff requires that underpinning of the SWPS be controlled by monitoring of vertical differential settlement to tolerable limits established ~~by appropriate analysis~~ before starting this work.

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Q.13 (Fig. SWPS-14). A correction to Note 9 is needed to indicate that all instrumentation and material identified in the Monitoring Matrix is to be Q-listed unless otherwise shown not to be required. A separate request of the Applicant to provide the following drawings identified on Fig. SWPS-14 has been made.

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C-2003 and C-2004	Building Settlement Monitoring Requirements
C-2035 and C-2036	Details of Wall and Pier Settlement Monitoring

BWST & SWPS -MIDLAND

J Kane
Rec'd 4/26/82
from Roger Huston

11/B5

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 22, 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
RESPONSE TO THE NRC STAFF REQUEST FOR ADDITIONAL INFORMATION REQUIRED FOR COMPLETION OF STAFF REVIEW OF BORATED WATER STORAGE TANK AND THE UNDERPINNING OF THE SERVICE WATER PUMP STRUCTURE
FILE: 0485.16, B3.0.8 SERIAL: 16656
ENCLOSURE: RESPONSE TO THE NRC STAFF REQUEST FOR ADDITIONAL INFORMATION REQUIRED FOR COMPLETION OF STAFF REVIEW OF THE BORATED WATER STORAGE TANK AND UNDERPINNING OF THE SERVICE WATER PUMP STRUCTURE

During the Staff audit held at Bechtel's Ann Arbor offices on March 16-19, 1982, the NRC Staff identified various concerns for our response. We are responding to these Staff requests by forwarding the enclosed document which addresses each individual NRC Staff concern identified for the Borated Water Storage Tank and the Service Water Pump Structure.

We believe the enclosed information combined with the discussion of these responses at our March 19, 1982 meeting, and the Atomic Safety and Licensing Board hearing testimony for borated water storage tank, responds to the request and individual concerns identified for us by the Staff. The responses contained in the enclosure to this correspondence lend further support to our conclusion that the design issues related to the service water pump structure and borated water storage tank have been adequately resolved. With the physical completion of the confirmatory issues open items noted in the enclosed document, we believe that the Staff should be in a position to concur with our request to proceed with the work.

JAMooney for JWCook

JWC/RLT/mkh

oc0482-0075a100

~~8205040531~~ 45pp.

CC Atomic Safety and Licensing Appeal Board, w/o
CBechhoefer, ASLB, w/o
MMCherry, Esq, w/o
FPCowan, ASLB, w/o
RJCook, Midland Resident Inspector, w/o
RSDecker, ASLB, w/o
SGadler, w/o
JHarbour, ASLB, w/o
GHarstead, Harstead Engineering, w/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

MIDLAND PLANT UNITS 1 AND 2
RESPONSE TO THE NRC STAFF REQUEST FOR ADDITIONAL INFORMATION
REQUIRED FOR COMPLETION OF STAFF REVIEW OF THE BORATED WATER
STORAGE TANK AND THE UNDERPINNING OF THE
SERVICE WATER PUMP STRUCTURE

BORATED WATER STORAGE TANK(S) FOUNDATION REPAIR

CONFIRMATORY ISSUE 1

Provide a detailed releveling procedure for the Unit 1 tank.

RESPONSE

A detailed procedure has been developed to define a plan of action to relevel BWST 1T-60. The tank will be lifted, the ring beam and the sand below the tank bottom will be leveled, and the tank will be reconnected to the foundation. A summary of the procedure is provided below. This procedure is supported by an analysis which 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.

1. Lifting Procedure

The anchor bolts will be disconnected from the tank by removing the nuts, and strain gages will be mounted on the tank wall and bottom.

Protected steel cable will be looped about the three heater tubes, strung through the nozzle G vent, and fastened to provide support as well as to minimize deflection of the heater tubes during the lift.

Fourteen hydraulic jacks (see Figure BWST-1) will be located beneath the anchor bolt chairs and sequentially raised to lift the bolt chairs beyond the top of the bolts. The tank will be supported by wooden dunnage. At this point, 14 electromechanical jacks placed between the hydraulic jacks will be connected to complete the lift (see Figure BWST-2). The electromechanical jacks will be controlled from a central panel allowing the jacks to operate in unison to raise and lower the tank. The total tank lift will be at least 3 feet. Wooden dunnage will be placed between the tank bottom and ring wall for stable support during subsequent work. Strain gages will be monitored to confirm that the tank stresses remain within allowable limits.

GE

GE

2. Leveling Procedure for Ring Wall

Leveling will be accomplished by use of shims adjusted to a datum plane at least 1-1/2 inches above the lowest point on the original ring wall. The level datum plane will be determined using a transit and the benchmark leveling procedure. Forty numbered, prefabricated, 1-1/2 inch thick shims (see Figure BWST-3) will be placed between anchor bolts on the existing ring wall. (At least a 1-1/2-incn gap must exist between the bottom plate and the original ring wall to permit flow of grout (see Item 3 below). The top of the shim on the highest point on the ring wall will determine the elevation of the datum plane. The transit will be used to establish how much the other shims must be raised using prefabricated incremental shims. Final shim placement will be checked and documented. Results will be recorded as elevation compared to a suitable site benchmark. The leveled shim differential elevation shall be within $\pm 1/8$ inch within any 30 feet of circumference and within $\pm 1/4$ inch of the established datum over the entire circumference in accordance with American Petroleum Institute (API) 650 requirements for foundations (API 650, Welded Steel Tanks for Oil Storage, Fifth Edition, July 1973 and Supplement 1 of October 1973). The shims will be fixed in place by packing grout around the stack of shims (see Figure BWST-4). Five-Star Grout manufactured by the American Grout Company will be used; this grout meets site specification requirements.

3. Foundation Preparation and Tank Set-Down

Because the bottom of the tank will be elevated above the previous foundation due to the shims, additional sand must be added and contoured while the tank is supported by the dunnage. First, a cofferdam made of asphalt-impregnated fiberboard (Celotex) will be installed around the inner diameter of the ring wall to dike the additional sand (see Figure BWST-5). Oil-impregnated sand will be added up to the lip of the cofferdam and evenly sloped so that the center of the crown is 3-1/4 inches higher than the sand at the edge. A Celotex pad will be placed on the shims. Because the tank will be elevated 1-1/2 inches, coupling nuts and threaded rods will be used to lengthen anchor bolts as required. Dunnage will be removed and the vessel lowered, ensuring the anchor bolts are aligned with the bolt holes. Anchor bolt nuts will be reinstalled. The original ring wall will be cleaned in preparation for pouring grout. A form will be set up 4 inches outside the edge of the tank bottom (see Figure BWST-6). Grout will be poured and allowed to set at a level no higher than the lip of the tank bottom plate. This procedure will fill the void between the Celotex cofferdam and the bottom of the tank, providing a uniform level support for the tank. After the grout has cured, the anchor bolts will be sequentially tightened.

CONFIRMATORY ISSUE 2

Provide strain monitoring details, procedures, and acceptance criteria for new ring beam.

RESPONSE

After the new ring beam is constructed, the maximum strain areas of each foundation, which are the transition zones between the ring wall and the valve pit, will be monitored using a strain gage system. A summary of this strain gage system is provided below.

1. <u>Locations of Monitoring</u>

During the plant construction and operation periods, the strain measurements will be taken at the locations on the ring beam of both tanks as shown in Figure BWST-7.

2. <u>Apparatus and Procedure for Monitoring</u>

Figure BWST-8 shows details of the strain monitoring apparatus installed at each monitoring location. This apparatus consists of a stainless steel rod embedded at one end in the ring beam and positioned inside a structural steel tube. The other end of the rod protrudes into a square structural tube through a hole in the side. The tube is attached to the new ring beam with embedded studs and has a conduit attached to it; this conduit provides access for the expanding gage block shown in Figure BWST-9. The expanding gage block, when lowered into the structural tube, will fit onto a small positioning rail welded to the tube, which holds the gage block in place. The gage block can be expanded to fill the gap between the end of the rebar protruding into the tube and a small section of rebar welded to the opposite face of the tube. By removing the gage block and measuring its width with a micrometer, the gap length can be determined. By comparing the

CCE Problem w/ 20' length

measured gap length to the initial gap length as installed, the average strain in the 20-foot gage length can be determined.

3. <u>Frequency of Monitoring</u>

The strain monitoring frequency of selected locations is every 60 days during plant construction and every 90 days during the first year of plant operation. Subsequently, it is planned that the frequency of measurement will be established after evaluating the measurements taken during the first year. As a minimum, the BWST ring beams will be monitored annually for the next 5 years of plant operation and then at 5-year intervals thereafter.

CoE comment

4. <u>Acceptance Criteria</u>

a. <u>Allowable Strain</u>: If the cumulative increase in gap width exceeds 0.4 inch at any time during the monitoring period, at any monitoring location, the monitoring interval will be increased to at least every 60 days to permit evaluation of the strain. If it is determined to be necessary, observation pits will be made to expose the ring beam for inspection of possible cracks.

b. <u>Absolute Strain</u>: The absolute strain, as a measure of the cumulative increase in gap width, during 40 years of plant life, for all the reference monitoring locations is 0.5 inch.

Strain monitoring procedures, including frequency of monitoring and acceptance criteria, will be included as part of the technical specifications in the final safety analysis report.

SERVICE WATER PUMP STRUCTURE

CONFIRMATORY ISSUE 1

Provide basis for establishing existing structural stresses.

RESPONSE

The overhang portion of the service water pump structure (SWPS) rests on fill. Part of the load from the overhang is supported by the fill while the remainder receives its support from undisturbed natural material under the lower basemat. Load transfer for the overhang load is primarily through the outside north-south walls to the lower basemat and then to the undisturbed natural material.

The loading on the fill under the overhang portion of the building is indeterminate because of the soil conditions. Therefore, it is not possible to calculate the existing stresses in this portion of the building. Evaluation of the building in its current state, however, has not revealed any structural distress.

COE

During the underpinning installation, the structure will be jacked to transfer the load from the overhang to undisturbed natural material under the base of the underpinning wall. Part of the jacking load will relieve the structural load supported on fill while the balance will relieve the load and the corresponding existing stresses being transmitted to the lower basemat by the north-south outside walls. This then allows an analysis to determine the stresses in the structure.

CONFIRMATORY ISSUE 2

Provide justification for use of a subgrade modulus of 4,000 kcf during final jacking.

GEI

RESPONSE

As described during the March 16 through 19, 1982, NRC staff audit, the effects of preload are obtained by subtracting the results of System 2 from System 1. Details of these systems were described during the audit.

When evaluating preload, the effects of differential settlement are not considered. Differential settlement effects are considered in another system and are combined with the effects of preload by applicable loading combinations.

GEI

Therefore, it is necessary to use a stiff structural spring as a boundary element in the model when considering preload effects. The value of 4,000 kcf is large enough to make the effects of differential settlement negligible while being acceptable in the computer analysis.

CONFIRMATORY ISSUE 3

Provide acceptance criteria for allowable differential settlement during underpinning installation.

CE & GEI Comments

RESPONSE

During underpinning installation, the effects of differential settlement will be monitored by a strain monitoring program. Four extensometers will be mounted on the east and west exterior walls (refer to Figure SWPS-14 provided with the response to Confirmatory Issue 15). A 5/16-inch displacement of the 20-foot gage length will cause underpinning activities to be stopped until the cause of the displacement is determined and appropriate corrective actions are taken. The 5/16-inch criterion is based on the reinforcing steel approaching two-thirds of its yield strain in the monitoring area.

CONFIRMATORY ISSUE 4

Recheck tendon anchor analysis for shear at the plate and wall and provide results.

SEB

RESPONSE

The post-tensioning anchorage has been reanalyzed for shear at the wall face, as requested during the March 16 through 19, 1982, NRC staff audit. The resulting shear stress is 94 psi. This is below the American Concrete Institute (ACI) allowable shear stress of 126 psi ($2\sqrt{f'_c}$) for one-way action and is, therefore, acceptable.

CONFIRMATORY ISSUE 5

Reevaluate the use of drilled-in dowels regarding embedment or use of rock bolts.

SEB

RESPONSE

SEB

Because of spacing and capacity limitations, rock bolts have been eliminated as a means of connecting the existing structure to the underpinning wall. Grouted-in reinforcement will be used at the vertical interface. This reinforcement will consist of two rows of No. 9 bars spaced at 12 inches center to center.

Embedment length will be based on the splice length for a class C splice as defined in the ACI 349-76 Code.

CONFIRMATORY ISSUE 6

Perform sliding calculations using site-specific response spectra (SSRS) seismic loads and provide results.

RESPONSE

CE & GEI

The stability analysis calculations have been refined using seismic loads equal to 1.5 times the Midland FSAR safe shutdown earthquake (SSE) loads. These exceed the SSRS seismic loads. Factors of safety against sliding are now 1.45 in the north-south direction and 1.5 in the east-west direction. These values exceed the required value of 1.1. Hence, the foundation is acceptable.

CONFIRMATORY ISSUE 7

Complete the calculation for an empty forebay cell and provide results.

SEB

RESPONSE

The calculation for an empty forebay cell has been completed. The structural capacities of the four enclosing walls and the base slab exceed the imposed forces. The most critical loading, 60.1 ft-kips/ft, occurs on the east wall. The capacity of this wall at the critical section is 70.6 ft-kips/ft.

CONFIRMATORY ISSUE 8

Provide maximum rebar stress in all elements of the base slab at elevation 620'.

RESPONSE

The load case that causes the largest rebar stress in any element of the base slab at el 620' is: SEB

$$U = 1.0\ (D + F + L + P_L + E')$$

where

D = dead load

F = hydrostatic pressure

L = live load

P_L = preload effects from jacking

E' = SSE

Figure SWPS-1 gives the reinforcing steel stress in all elements for this loading combination. The reinforcing steel stress in all elements is below the allowable value from the ACI 318-71 Code.

CONFIRMATORY ISSUE 9

Identify maximum rebar stress in elements adjacent to identified critical elements and other areas of potential high stress. SEB

RESPONSE

During the March 16 through 19, 1982, NRC staff audit, critical elements were identified where the reinforcing steel stress exceeded 54 ksi ($0.9F_y$). Stress calculations for these elements did not utilize additional reinforcement that is present. These calculations also did not utilize the capacity of the concrete for resisting in-plane shear. (This capacity was reserved for out-of-plane shear.) These elements have been reanalyzed using the additional reinforcement and the available concrete capacity for in-plane shear. Based on the new analysis, the reinforcing steel stress in all the previously designated critical elements is below 54 ksi.

The areas of potential high stress in the various structural components are identified in Table SWPS-1. The stresses for these areas consider the combined effect of out-of-plane shear, in-plane shear, membrane forces, and out-of-plane bending. The capacities of these components have been calculated in accordance with the applicable ACI Code and found to be greater than the applied forces.

CONFIRMATORY ISSUE 10

Complete calculations for out-of-plane shear and provide results. SEB

RESPONSE

The analysis of the existing structure considering out-of-plane shear has been completed. The response to Confirmatory Issue 9 addresses the capacity of the structural elements for this force in combination with other applicable forces.

CONFIRMATORY ISSUE 11 (Same as question 1

Provide more information as to stress condition for existing parts of structure: SEB

Maximum stresses
Critical combination
Identify true critical elements based on actual rebar

(To demonstrate the behavior of the structure, provide the above information for a loading combination which generally gives governing stresses for the structure.)

RESPONSE

The building has been analyzed for all the applicable loading combinations. The various structural components have been designed for the governing load combinations. It has been noted that the following load combination generally governs:

$$U = 1.0 \ (D + F + L + H + S + P_L + E')$$

where

H = lateral earth pressure

S = surcharge

E' = Midland FSAR SSE

For each wall and slab in the structure, plots of the element forces due to static, preload, and seismic forces at a vertical and horizontal line of elements were obtained to study the building behavior. A copy of the graphs for the south wall are attached as Figures SWPS-2 through 10.

CONFIRMATORY ISSUE 12

Provide evaluation of interaction of the SWPS with the circulating water pump structure, retaining wall, and electrical duct banks.

SEB

RESPONSE

The SWPS is separated from the circulating water intake structure (CWIS) and the retaining wall by 1-inch expansion joints.

The maximum combined east-west seismic movement of the SWPS and the retaining wall is less than the 1-inch gap. Hence, there is no contact.

An evaluation of the interaction between the CWIS and SWPS will be performed later.

The concrete duct banks contain no reinforcement at the junction with the SWPS. The connection to the building is considered flexible. Hence, the duct banks offer no resistance to the movement of the SWPS during a seismic event.

CONFIRMATORY ISSUE 13

Provide procedures for acceptance of the bearing stratum. Include a discussion of the maximum differential elevation between pier bottoms and the maximum thickness of lean concrete.

RESPONSE

Approval of the foundation subgrade prior to placement of concrete for the pier will be given by the resident geotechnical engineer for the Midland remedial underpinning operations. Acceptance or rejection of the subgrade will be based ~~solely~~ on the resident geotechnical engineer's observations evaluation and judgment.

GEI

The resident geotechnical engineer's evaluation of the subgrade will consist of a visual inspection of the condition of the bearing stratum to confirm that foundation conditions are as anticipated in the design. In addition, for each foundation, a map of the subgrade will be prepared. The map will include:

a. A visual description of the subgrade material including consistency, color, and texture

b. Presence of any water

c. Elevation of subgrade

Once the subgrade has been approved, photographs will be taken of the subgrade and any exposed sidewalls. The photographs and map will serve as the permanent record.

To aid in evaluating the condition of the subgrade, the resident geotechnical engineer will [redacted] use either a miniature static or dynamic cone penetrometer. The static cone penetrometer will be used whenever cohesive soils are encountered. The dynamic cone will generally be used if any granular soils are encountered. Relationships between the data obtained using these devices and the estimated ultimate bearing capacity of the foundations are presented in Figures SWPS-11 and SWPS-12. The correlation between the pier or plate load test results and the penetration tests performed on the foundation soils will be used to correct the correlation graphs and to judge the suitability of the bearing stratum. Additional geotechnical in situ and laboratory test(s) may be performed if the resident geotechnical engineer believes that initial findings require a further confirmation of conditions. Such testing requirements will be determined on a case-by-case basis.

GEI

If the subgrade is not accepted by the resident geotechnical engineer, the piers shall be excavated to a depth where a suitable subgrade is encountered. If such a pier is constructed immediately adjacent to an existing pier, the maximum depth of excavation below the lean concrete mud slab for the existing pier shall not exceed 18 inches. If the pier is not immediately adjacent to an existing pier, it can be extended to any suitable depth, provided that the base of the new pier's mud mat is not more than 18 inches below the zone of influence of any existing piers. The zone of influence of an existing pier shall be defined by lines extending downward from the edge of the footing at a rate of two vertical to one horizontal (see Figure SWPS-13).

CCE

If the subgrade is not acceptable at 18 inches below the existing pier or influence zone, defined in the above paragraph, the contingency plans addressed in the response to Confirmatory Issue 18 will be implemented.

All over-excavation shall be backfilled with lean concrete to the original design footing elevation. The only maximum thickness restriction on the amount of lean concrete that can be placed beneath a footing is the limitation imposed by the above undermining restriction.

CONFIRMATORY ISSUE 14

Provide pier load test procedures.

RESPONSE

A load test will be performed to 1.3 times the* jacking load for one of the initial piers. In addition, a load-reload cycle will be built into the procedure to aid in determining the apparent Young's modulus of the foundation subgrade.

The test procedure will follow American Society for Testing and Materials standard methods for the Test for Load-Settlement Relationship for Individual Vertical Piles Under Static Axial Load, Designation D1143, with modifications deemed appropriate. — meaning?

GEI

The load would be applied in accordance with D1143 in increments of 25, 50, 75, and 100% of the jacking load, then with an overload of 115% and finally to 130%. An intermediate rebound-reload cycle would be included at 100% of the* jacking load. A sufficient length of time shall be allowed for the 100 and 130% test increments so that movements are reduced to rates not exceeding 0.01 inch per hour. — 0.005 inch/hour — Discuss because of structure sensitivity

— highest design load

Carlson pressure cells will be installed near the top and bottom of the shaft and measures will be taken to reduce skin friction effects. The gaps between lagging and any corrugations will be filled and the pier will be lined with thick plastic sheeting to minimize effects due to side friction.

questionable whether s[illegible] friction is sufficiently [illegible]
suggest bitumen coating (1/8" thick)
plastic has ϕ = .2 to .3

CONFIRMATORY ISSUES 15 AND 16

Provide strain monitoring criteria matrix.

Provide drawings on strain monitoring and Carlson meters, including locations and details.

GEI

RESPONSE

Figure SWPS-14 provides the requested information. Also refer to the response to Confirmatory Issue 3.

CONFIRMATORY ISSUE 17

Identify critical construction stages and critical measurements.

RESPONSE

During construction of the SWPS underpinning, two stages are considered most critical. The first stage occurs during construction of corner Piers 1, 2, and 3. After the construction of these corner piers, the entire weight of the overhang can be supported without depending on the fill support. The second stage occurs during adjustment of the jacking load from initial to final loads.

GEI

During the construction of Piers 1 through 3, the extensometers, which monitor strain, and the settlement indicators (refer to Figure SWPS-14) shall be read twice each shift. In addition, during this stage the load-measuring indicators located in Piers 1, 2, and 3 shall be monitored for an increase in load twice each shift.

When the final jacking load is applied, the existing structure is subjected to the maximum jacking load. The extensometers on the east and west exterior walls will be monitored for strain twice each shift.

CONFIRMATORY ISSUE 18

Provide contingency plan and discussion of possible remedial actions.

RESPONSE

Postulated situations that might occur during underpinning construction and the appropriate remedial measures for such situations are listed below. In any situation, one or a combination of measures may be adopted. These measures will be included in a project specification.

1. <u>Failure of Dewatering System</u>

   a. If power fails, use the required backup power system

   b. If the system is inadequate, correct it with additional wells/pumps

   c. If monitoring indicates excessive fines:

      1) Repair well
      2) Replace well
      3) Evaluate effect of total quantity of fines lost during construction

2. <u>Uncontrolled Groundwater Flow into Excavation</u>

   a. Identify the source of uncontrolled flow and correct it

   b. Equalize water level in pier excavation and initiate dewatering as determined by the resident geotechnical engineer

3. <u>Ground Loss</u>

   a. Use techniques such as forepoling or spieling (sheeting) to stabilize ground

   b. Use chemical or cement grouting

4. <u>Unacceptable Bearing Stratum</u>

   a. Excavate below planned elevation and backfill with lean concrete

   b. Increase bearing area of piers

5. <u>Excessive Pier Settlement</u>

   a. Hold load until criteria defined for pier acceptance is met

   b. Increase embedment of future piers

GEI

c. Increase bearing area of future piers

d. Increase jacking load to accelerate settlements (individual piers only)

e. Install additional pier(s)

f. Remove pier, excavate to acceptable material, and replace pier concrete

6. Jacking System

a. If the hydraulic system is defective, use the required backup system

b. If the jack malfunctions, replace the jack

GEI

7. Loss of Monitoring - Extensometer

a. Use backup dial gage

b. Reestablish monitoring point

8. Structural Damage to Existing Structure

a. Determine the cause

b. Remove the source of the problem

c. Repair the damage

CONFIRMATORY ISSUE 19

Provide summary submittal of specification or drawing notes to cover frequency for checking and adjusting jacking loads.

RESPONSE

The following note will be placed on the construction drawings:

> During the construction of Piers 1, 2, and 3, the jacks shall be monitored every day, including holidays and weekends, and adjusted if necessary. After attaining the initial jacking load for Piers 4 through 10, each jack shall be monitored and adjusted, if necessary, each working day.

During the adjustment of the jacking load from initial to final load, all jacks shall be monitored every day, including holidays and weekends. After the level of the final jacking load is attained, each jack shall be monitored and adjusted, if necessary, each working day until the wedges are driven and the jacks removed.

CONFIRMATORY ISSUE 20

Provide method to be followed for transfer of jacking load into permanent wall.

RESPONSE

Upon completion of Pier 10, the structure is fully supported by initial jacking loads. At this stage, the load is transferred from the initial to the final design jacking load. The final design jacking force shall be simultaneously applied to Piers 1 through 10 in three groups of jacks. Each group is connected to a separate hydraulic system. The increase in force to reach the level of the final jacking force shall be applied incrementally. The increments shall not exceed 25% of the additional force up to 75% of the increase. Thereafter, increments shall not exceed 10%, up to 100% (+5%, -0%) of the required additional force. The acceptance criteria for the final jacking force shall be when the rate of movement of the underpinning relative to the upper base slab (el 617') is less than 0.01 inch for 1 hour. This force shall be maintained, monitored every working day, and adjusted if necessary until the rate of settlement has reached a predetermined rate. At this stage, the wedges shall be tightly driven and the jacks removed.

check SWPS testimony - pg 50

CONFIRMATORY ISSUE 21

Provide decision on tunnel location prior to hearing.

RESPONSE

As indicated in a report, Summary of Soils-Related Issues at the Midland Nuclear Plant, dated April 19, 1982, access to the north and east side underpinning piers will be from the outside of the building by use of open excavation. Access for piers on the west wall will be provided by an access shaft from grade and a tunnel under the west side of the el 617' base slab.

Resolved

CONFIRMATORY ISSUE 22

Provide a report on crack repair.

SEB

RESPONSE

A report discussing the evaluation and repair requirement for cracks in structures for the Midland Nuclear Plant has been submitted to the NRC.

CONFIRMATORY ISSUE 23

Perform a limit analysis on a wall considering the effects of cracking.

SEB

RESPONSE

The following is a brief status report on the limit analysis being conducted by the Portland Cement Association, consultants to Consumers Power Company.

1. Background

In a previous report submitted to the NRC staff, cracks observed in the SWPS were described and their significance was evaluated. Cracks observed in the structure were primarily attributed to restrained volume changes that occur in concrete during curing and subsequent drying. No evidence of structural distress was observed. Although the possibility of settlement-related cracking could not be completely eliminated, crack patterns did not support the conclusion that settlement was a primary cause of cracking.

As a measure of significance of observed cracks relative to future integrity of the structure, the tensile stress that uncracked concrete may be assumed to carry was compared to available tensile capacity provided by structural reinforcement causing the cracks. This calculation was made for sections in the vicinity of cracks that had a measured width of 0.01 inch or greater. In the calculation, concrete is assumed to carry a principal tensile stress of $4\sqrt{f'_c}$ where f'_c is the specified concrete compressive strength.

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

After review of the report on evaluation of cracking in the SWPS, NRC staff members requested that a more detailed analysis be made to evaluate the capacity of the east and west walls of the SWPS. Therefore, the limit analysis described in this status report was initiated.

2. Methodology

The approach being used to evaluate the capacity of walls in the SWPS is to estimate forces that can be induced in the structure. This is being done by evaluating capacities at selected sections of wall members. Capacities are being calculated using representative stress versus strain relationships for material properties, and using section geometries determined from engineering design drawings. After sectional analyses are completed, the capability of the structure to resist hypothesized applied force distributions is calculated. These calculations will indicate the maximum level of shear force and moment that can be induced in the walls under idealized support conditions. Calculations will provide a "worst case" estimate of forces that the walls must resist. Once this estimate is known, the capacity of the walls to resist applied forces can be evaluated.

3. Expected Results

Results from limit analyses will provide an estimate of the maximum in-plane bending and shear forces that can be induced in walls of the SWPS for assumed force distributions. This will provide a conservative estimate of whether capacities of the walls are sufficient to resist applied forces. It is expected that the final calculation would support the hypothesis that the walls have sufficient capacity to resist the applied forces.

4. Current Status

As of April 22, 1982, analyses have been completed for vertical and horizontal sections through the SWPS. The north overhang of the building has been analyzed for the conservative assumption that it is unsupported by backfill or underpinning. Calculations are in progress to evaluate horizontal shear forces that could be induced in the structure. In addition, a report on limit analyses is in progress. It is estimated that the calculations will be completed and the report will be submitted to the NRC by the first week of May 1982.

CONFIRMATORY ISSUE 24

Provide a commitment for monitoring fines from construction wells in Q-listed areas using a 0.005 mm filter and for monitoring the performance of the construction dewatering system.

RESPONSE

The construction dewatering system is a temporary system and, therefore, is not subject to as rigorous a criterion (0.005 mm particle size in well discharge water) as would be applied for a permanent well installation. Consequently, the system operation test procedure for the construction dewatering system will be based on a 0.050 mm filter media. If the quantity of soil particles retained on a 0.050 mm filter is greater than 10 ppm during well operation, the well will be retested and removed from the system if retests confirm that the criterion is exceeded.

GEI

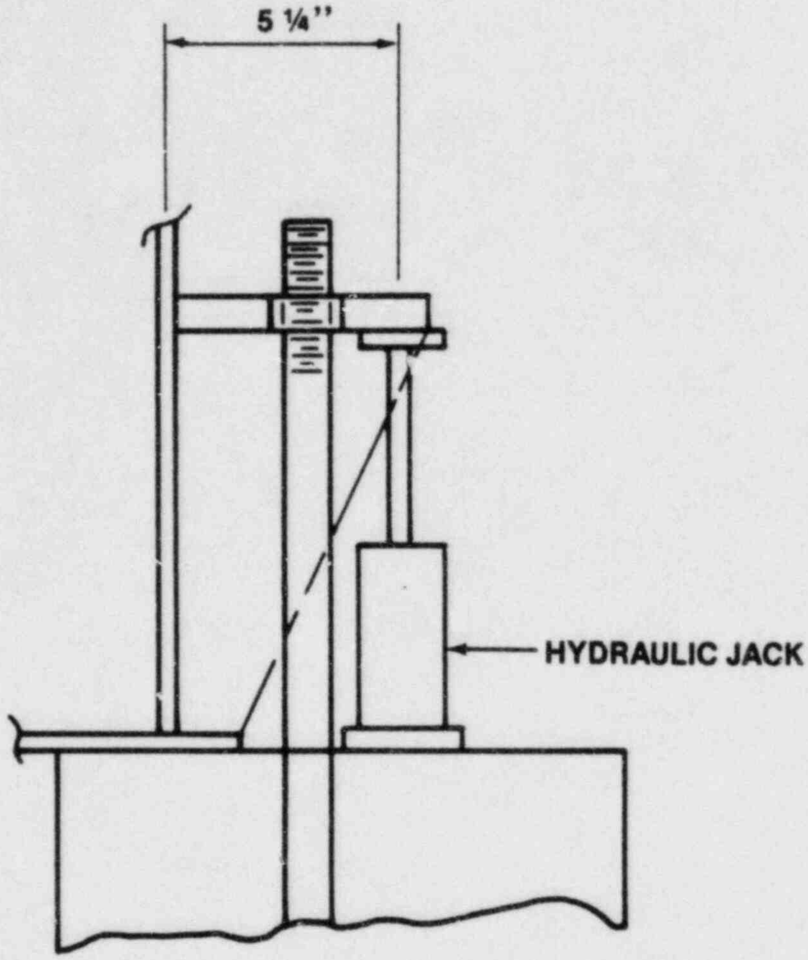
The well discharge will be monitored for the 0.005 mm size for information. If the amount of soil particles retained on a 0.005 mm filter is greater than 10 ppm, an engineering evaluation will be made based on actual pumping rates, etc, to determine the significance of the condition. The NRC will be made aware of any such situation.

The dewatering system will be monitored by a series of observation wells. The wells will consist of 1/2-inch diameter slotted polyvinylchloride well screens with 1/2-inch diameter riser pipes. The wells will monitor the groundwater levels in the fill and the undisturbed natural soil. The bottoms of wells used to monitor the water level in the fill will be installed at an elevation no lower than approximately 1 foot above the undisturbed natural soil.

The bottoms of wells used to monitor the water level in the undisturbed natural soil will be installed such that the bottom of the well screen is at approximately el 570' and the bottom of the bentonite seal will be approximately 2 feet below the interface of the fill and the undisturbed natural soil.

The construction dewatering system will be installed so that the excavation and construction operations can be performed under stable soil conditions. The resident geotechnical engineer will observe groundwater conditions as part of his responsibilities. The wells will be installed so that the water level in the fill will be drawn down to approximately the interface of the fill and natural soil. If the natural soil at the dewatering well location is cohesionless, the wells will be installed such that the water level in the natural cohesionless material will be drawn down below the depth of any excavation made in that material. Where such situations occur, the groundwater level will be lowered to approximately 2 feet below the base of the excavation, provided relatively pervious soil exists below the excavation. If the material below the base of the excavation is cohesive and relatively impervious, the water in any cohesionless material may be drawn down by localized dewatering techniques such as sumping.

interface variable
El. 605 Boring PD-27A
El. 601 PD-27
Below El. 583 CH-2
El. 588 CH-4
589 CH-3

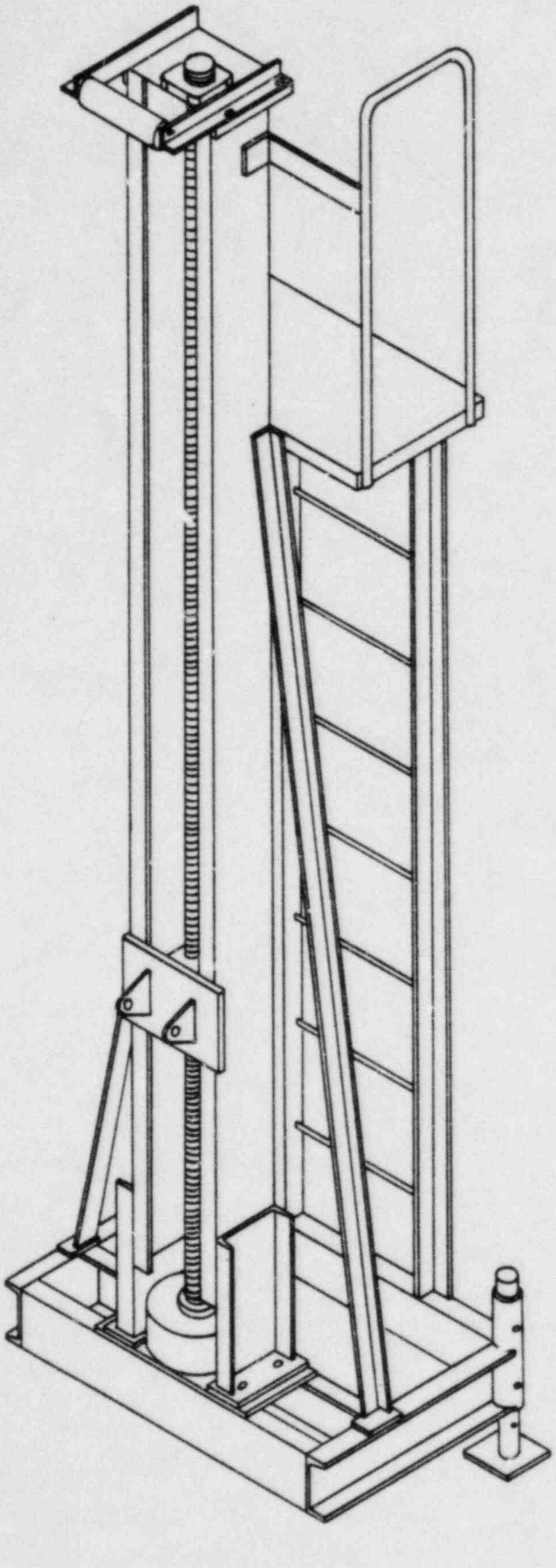


CONSUMERS POWER COMPANY
MIDLAND UNITS 1 AND 2

PLACEMENT OF
HYDRAULIC JACK

FIGURE BWST-1

G-1986-50

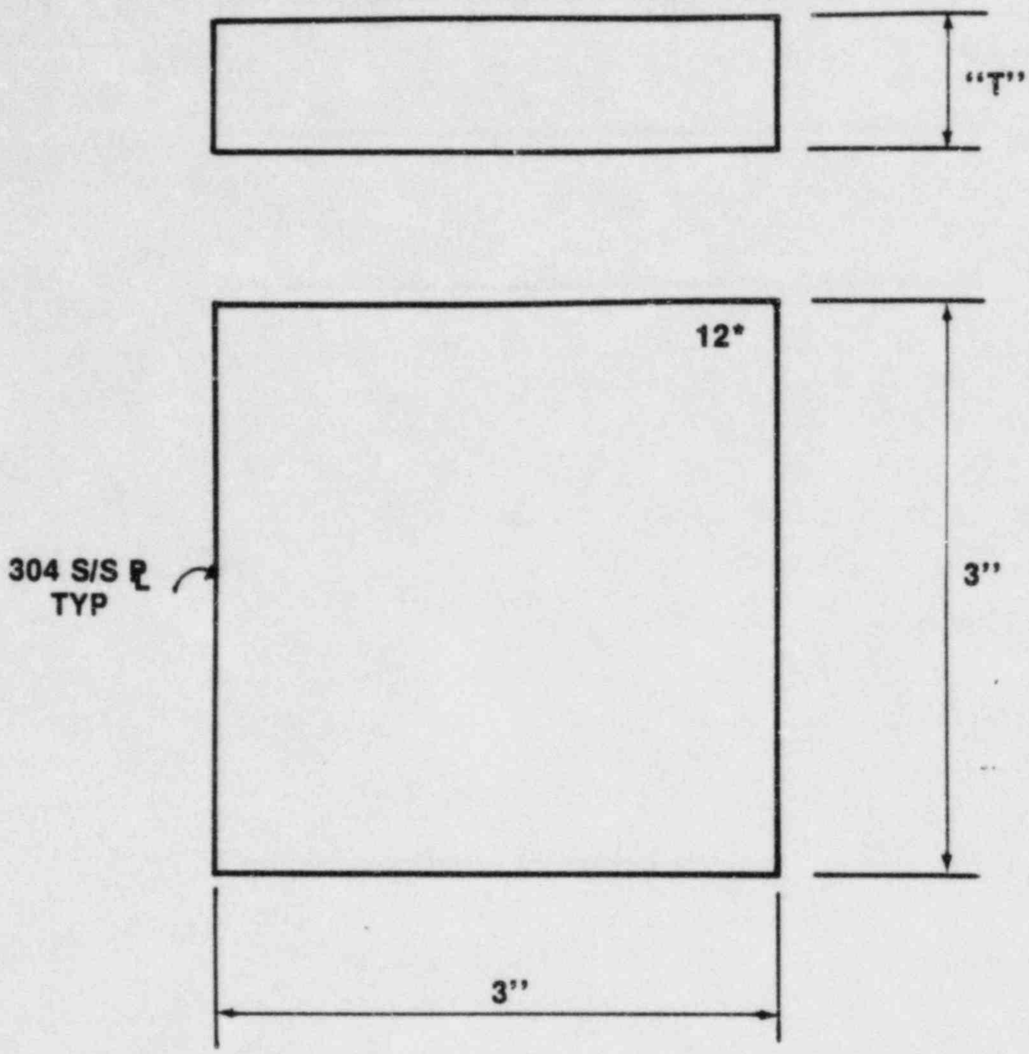


CONSUMERS POWER COMPANY
MIDLAND UNITS 1 AND 2

FIGURE I-2
ELECTROMECHANICAL JACK

FIGURE BWST-2

G-1986-85



"T"
1/32"
1/16"
1/8"
1/4"
1/2"
1 1/2"

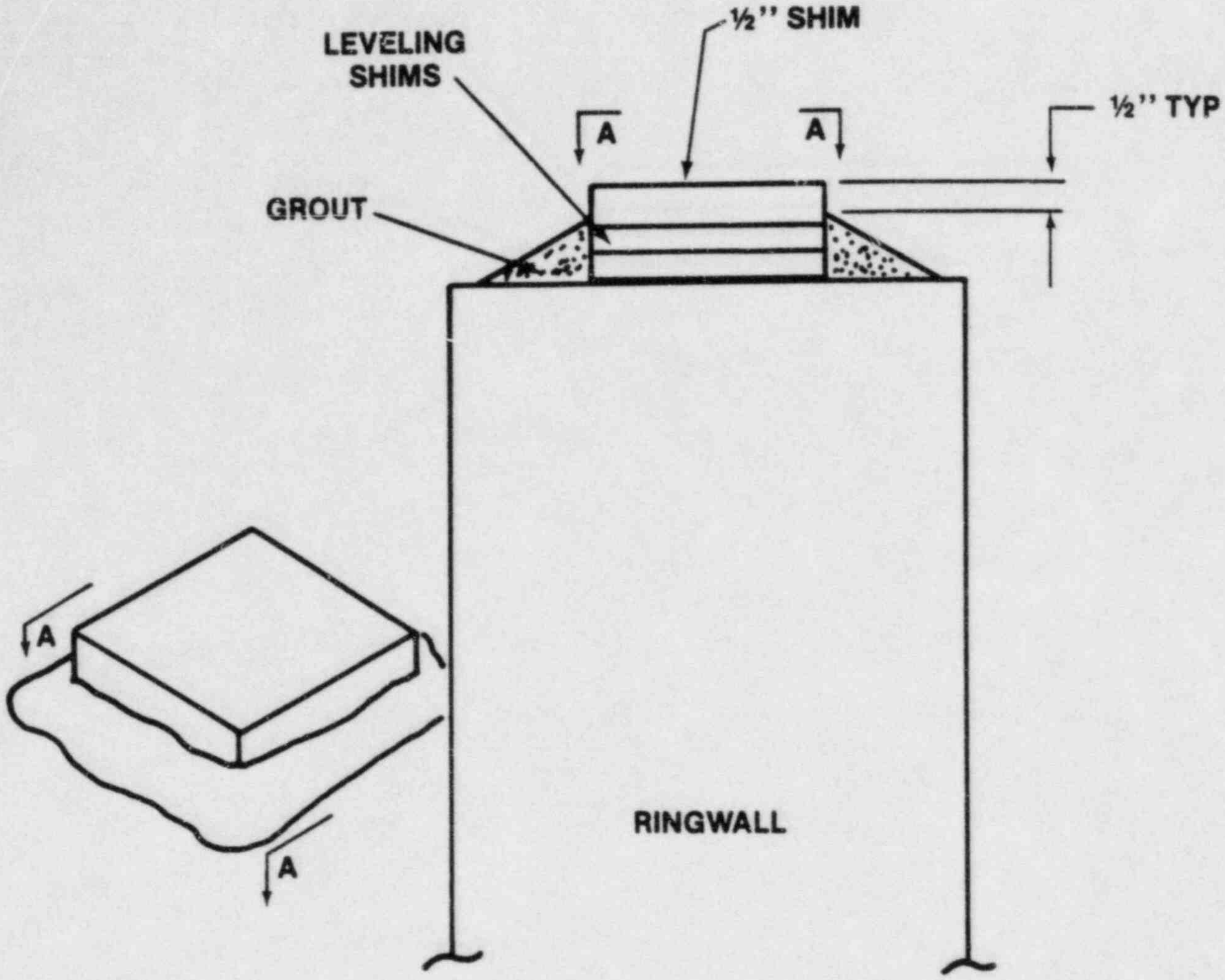
*Each 1-1/2" thick shim shall be identified with a numerical stamping sequentially from #1 through #42.

CONSUMERS POWER COMPANY
MIDLAND UNITS 1 AND 2

STAINLESS STEEL SHIMS

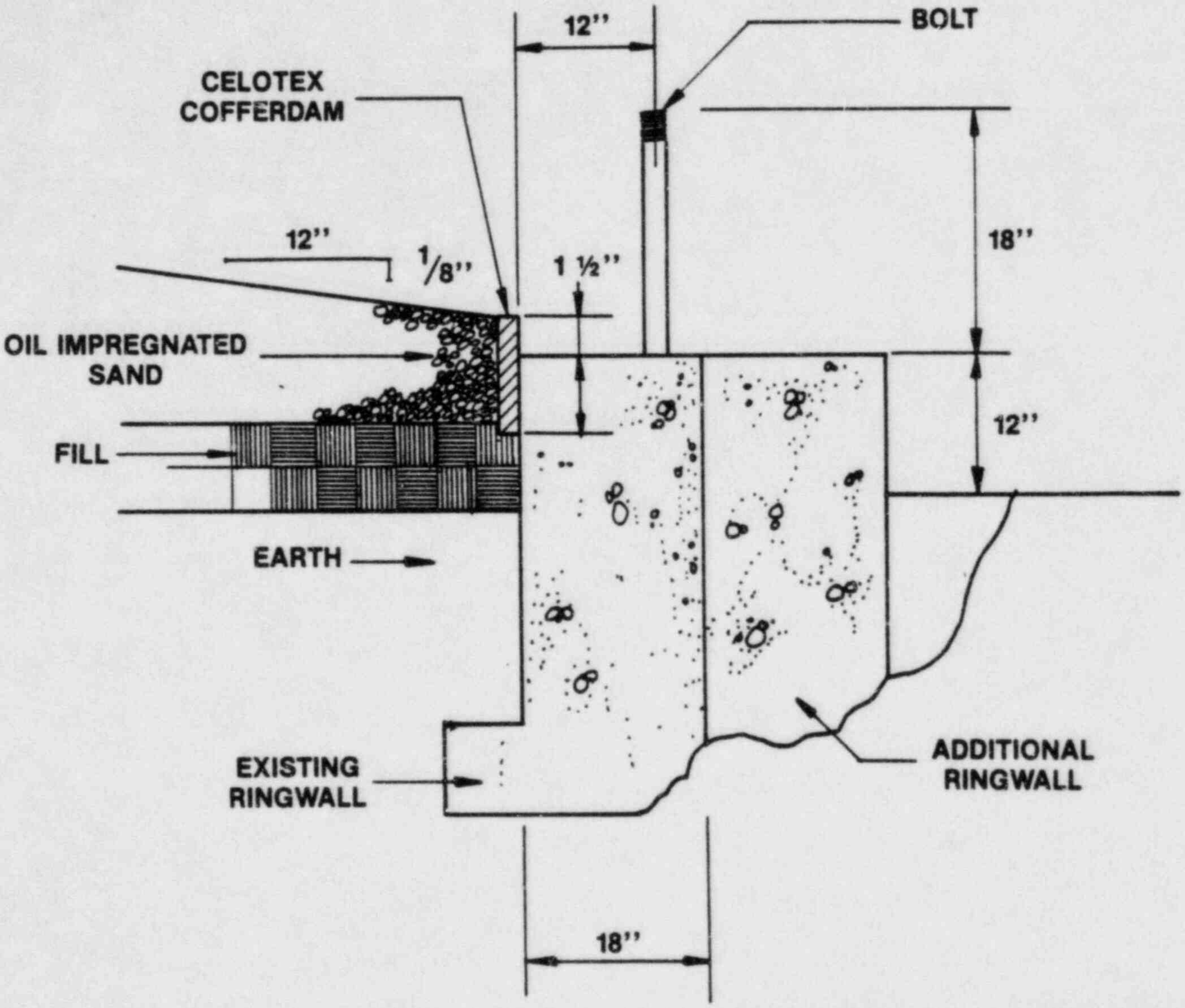
FIGURE BWST-3

G-1986-51



CONSUMERS POWER COMPANY MIDLAND UNITS 1 AND 2
DETAIL OF SHIM STACK FIXED BY GROUTING
FIGURE BWST-4

G-1986-52

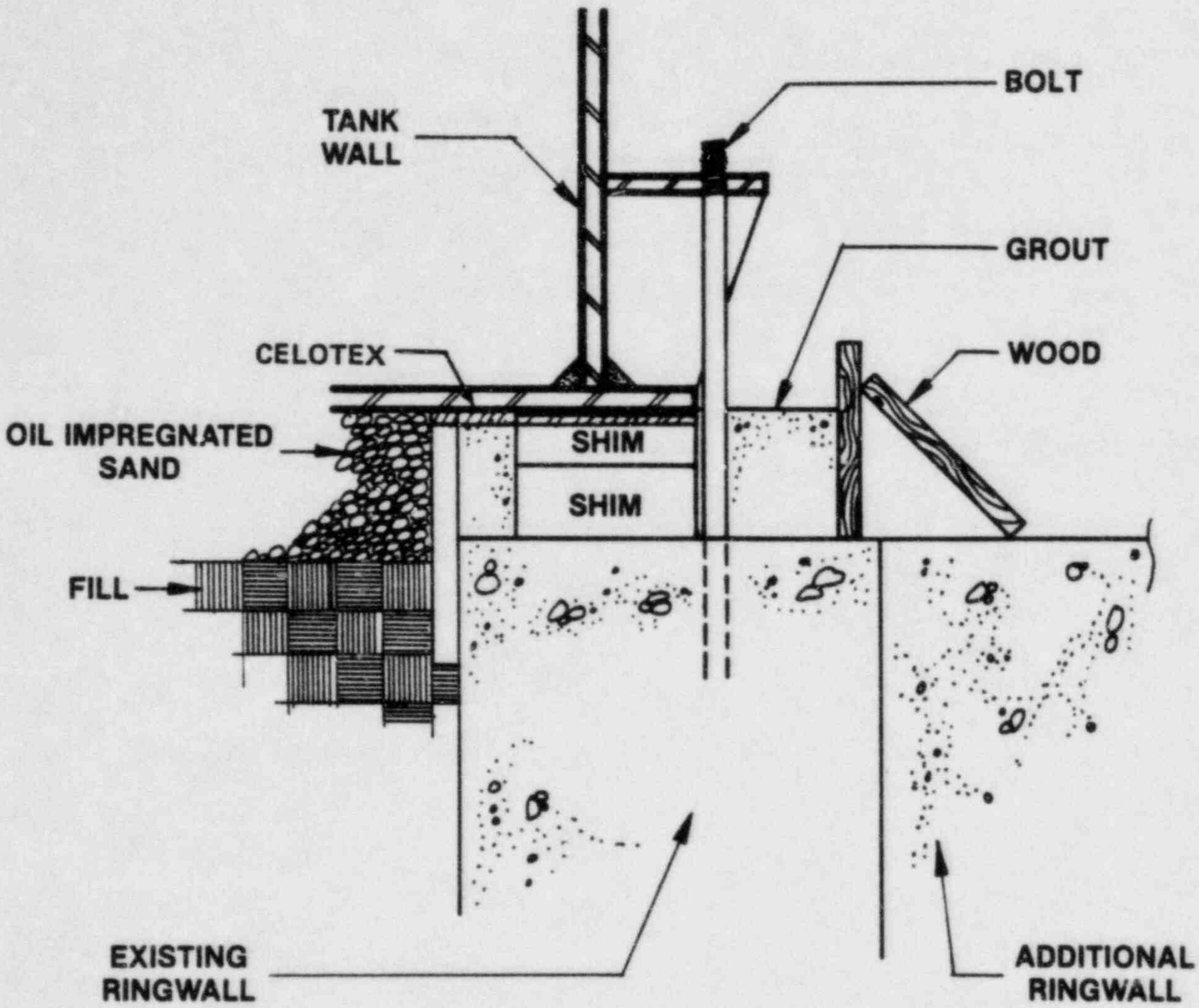


CONSUMERS POWER COMPANY
MIDLAND UNITS 1 AND 2

COFFERDAM AND OIL
IMPREGNATED SAND PAD

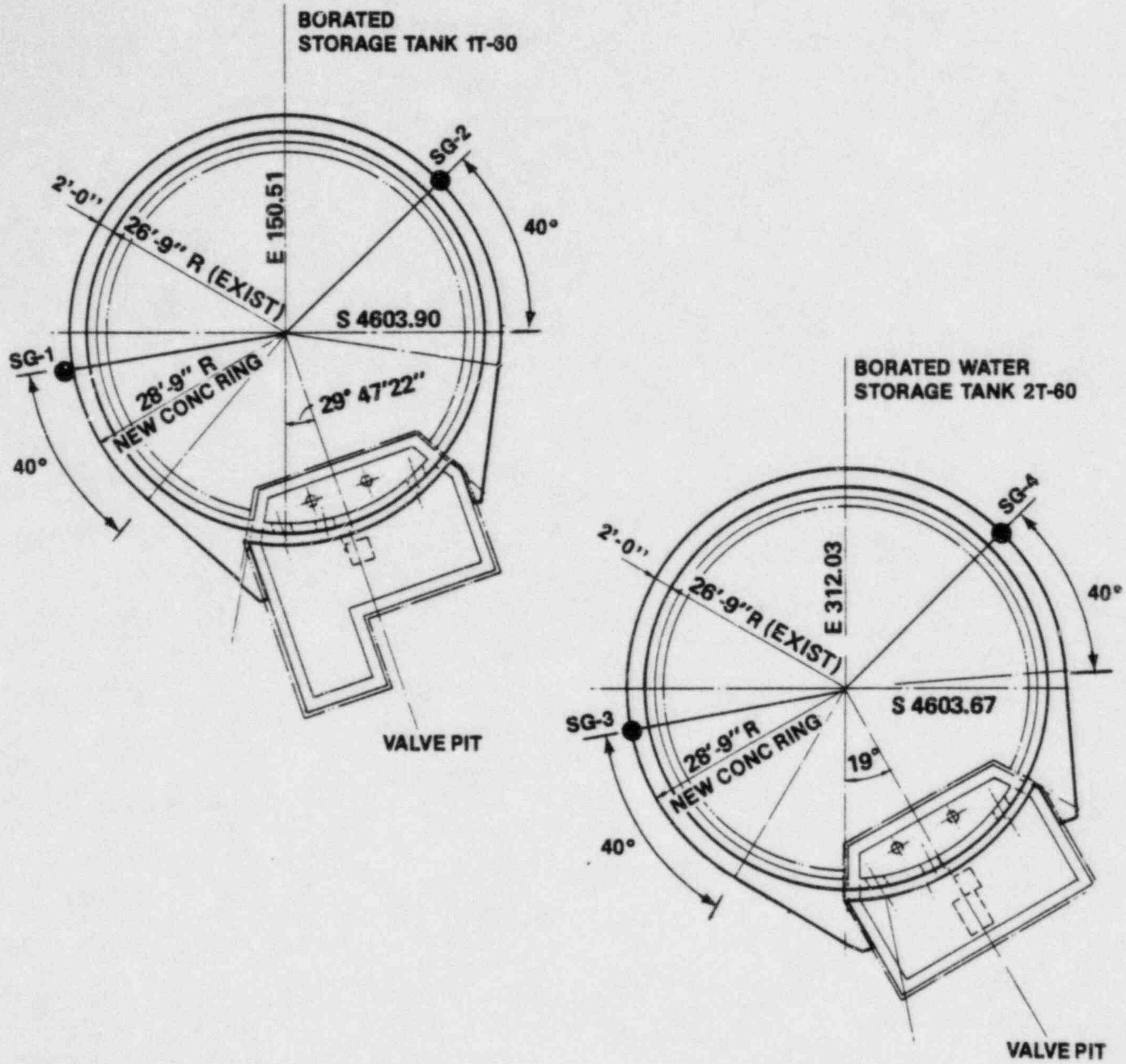
FIGURE BWST-5

G-1986-58



CONSUMERS POWER COMPANY MIDLAND UNITS 1 AND 2
TANK AND FOUNDATION AFTER POURING OF GROUT
FIGURE BWST-6

G-1986-57

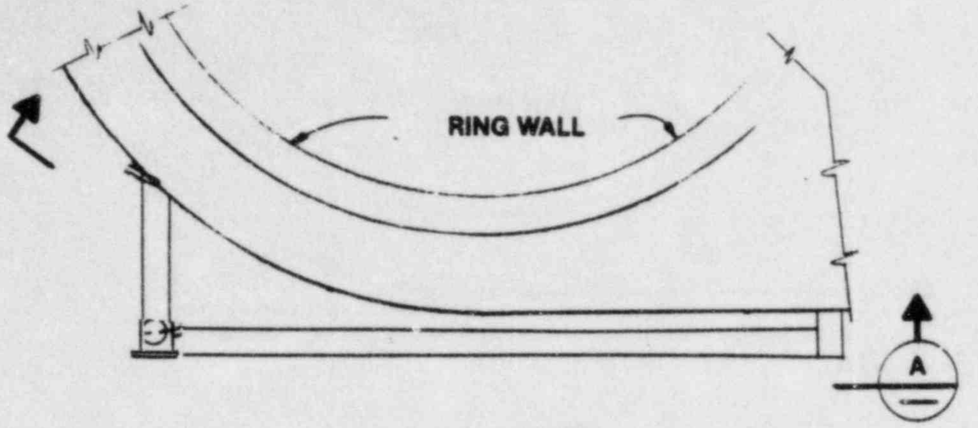


CONSUMERS POWER COMPANY
MIDLAND UNITS 1 AND 2

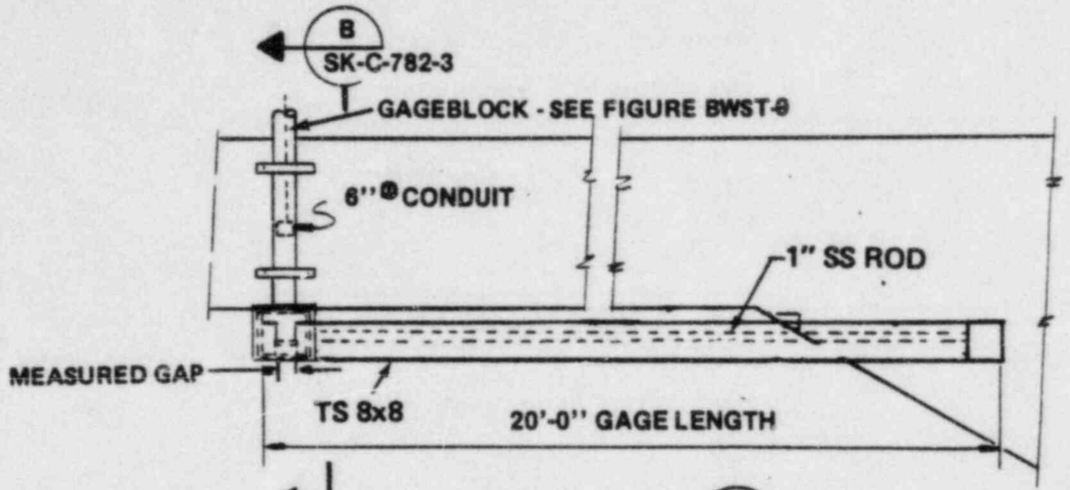
BORATED WATER STORAGE TANKS RING BEAM STRAIN MONITORING PROGRAM

FIGURE BWST-7

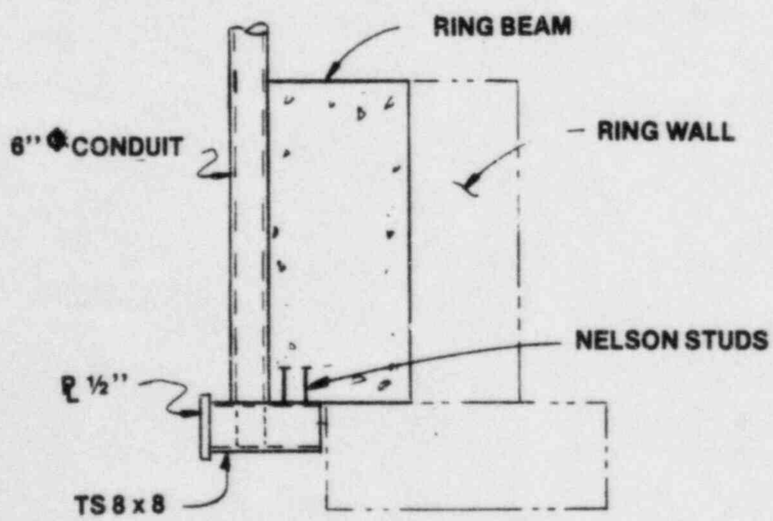
G-1986-59



PLAN VIEW



SECTION A



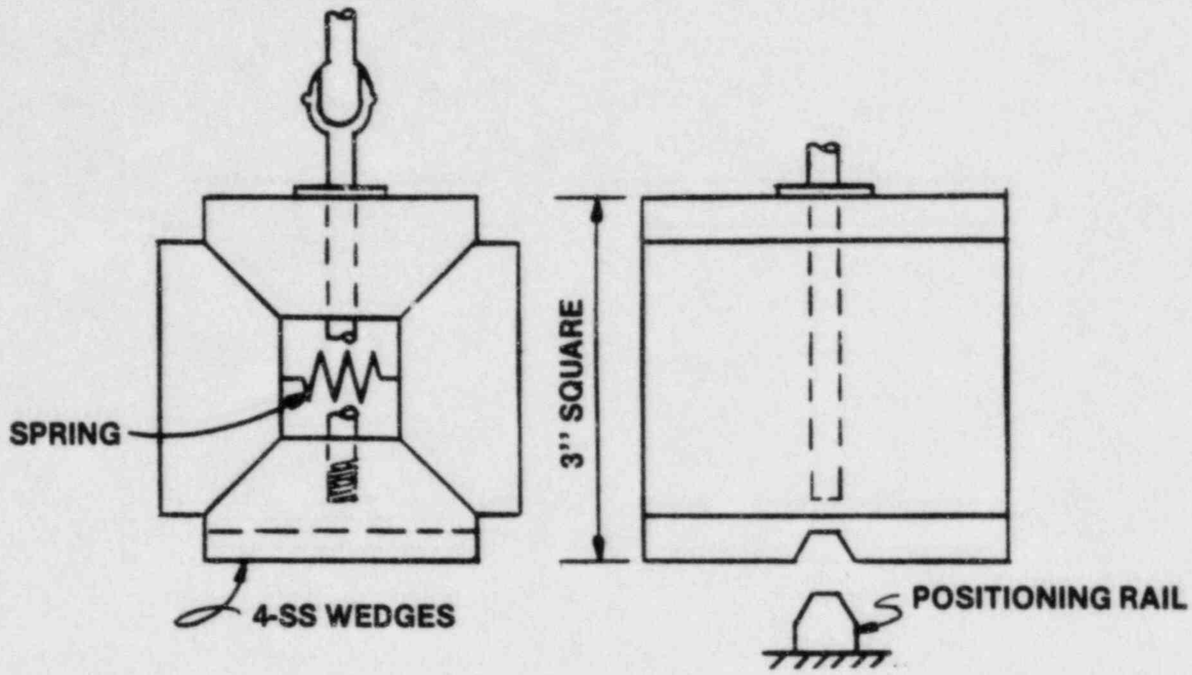
SECTION B
SK-C-782-3

CONSUMERS POWER COMPANY
MIDLAND UNITS 1 AND 2

BORATED WATER STORAGE TANK

FIGURE BWST-8

G-1986-62



EXPANDING GAGE BLOCK

CONSUMERS POWER COMPANY
MIDLAND UNITS 1 AND 2

BORATED WATER
STORAGE TANK

FIGURE BWST-9

G-1986-60

TABLE SWPS-1

MAXIMUM REINFORCING BAR STRESSES

Building Component	Max Reinforcing Bar Stress (ksi)
South wall	49.007
North wall (main structure)	23.356
North wall (overhang)	20.378
West wall	38.348
East wall	47.551
Lower basemat	*
Upper basemat	15.552
Slab at el 634.5'	45.713
Roof slab	40.789
Underpinning walls	42.752

*A yield line analysis of the lower base slab was performed. The ultimate capacity of the slab provides a factor of safety of 2.5.

Building component stresses are from FSAR load combinations, except for the underpinning wall, which is from an ACI 349 load combination.

5.020 ↕ T.	4.213 ↕ T.	4.081 ↔ T.	6.409 ↔ T.	8.419 ↔ T.	10.647 ↔ T.	8.609 ↔ B.	8.226 ↔ B.	7.950 ↔ T.	4.600 ↔ T.	4.225 ↕ T.	5.021 ↕ T.
5.852 ↕ T.	3.412 ↔ T.	4.971 ↔ T.	6.178 ↔ T.	6.886 ↔ T.	7.313 ↔ T.	5.707 ↔ B.	5.729 ↔ B.	6.371 ↔ T.	5.690 ↔ T.	3.902 ↔ T.	5.960 ↕ T.
8.108 ↕ T.	3.700 ↕ T.	4.833 ↔ T.	5.363 ↔ T.	4.801 ↔ T.	3.564 ↔ T.	4.154 ↔ B.	4.585 ↔ B.	4.832 ↔ T.	5.322 ↔ T.	3.638 ↕ T.	8.517 ↕ T.
10.370 ↕ T.	4.576 ↕ B.	4.291 ↔ T.	4.355 ↔ T.	3.555 ↕ B.	3.329 ↕ B.	2.959 ↔ B.	3.569 ↔ B.	3.938 ↔ T.	4.694 ↔ T.	4.638 ↕ B.	11.486 ↕ T.
14.233 ↕ B.	3.338 ↔ B.	3.036 ↔ T.	3.505 ↔ T.	3.252 ↕ T.	3.368 ↕ T.	2.908 ↕ T.	3.168 ↕ T.	3.344 ↔ T.	3.255 ↔ T.	2.653 ↔ B.	15.552 ↕ B.

N

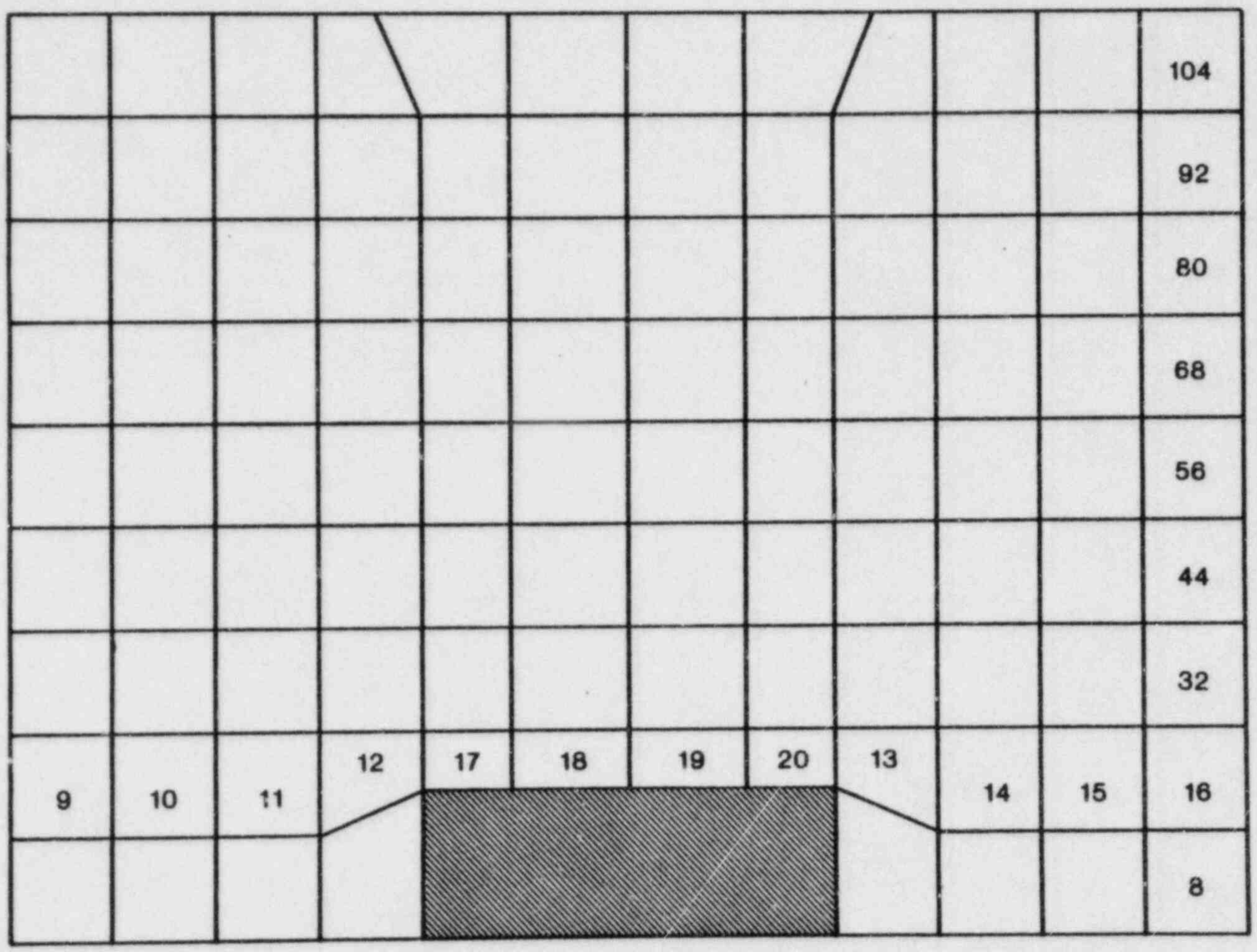
NOTE: ARROWS DENOTE DIRECTION OF MAXIMUM STRESS
FOR THE GIVEN LOAD CASE
NUMBERS INDICATE REBAR STRESSES IN KSP
0.9 F_Y 5 54KSP
T. DENOTES TOP BAR STRESS
B. DENOTES BOTTOM BAR STRESS

CONSUMERS POWER COMPANY
MIDLAND UNITS 1 AND 2

REBAR STRESSES
LOAD CASE
U = 1.0(D + F + L + PL + E')

FIGURE SWPS-1

G-1986-63

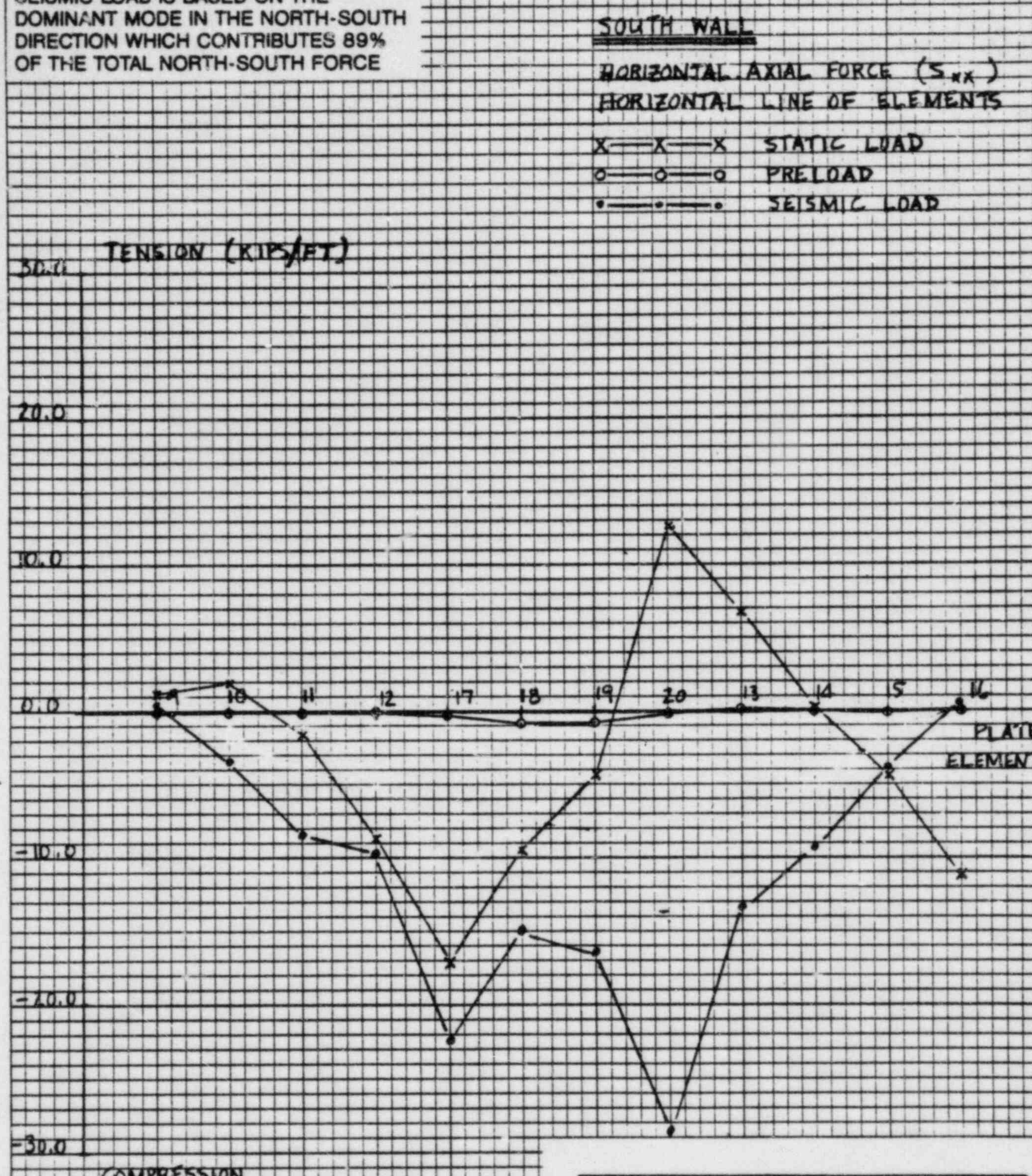


CONSUMERS POWER COMPANY
MIDLAND UNITS 1 AND 2

SERVICE WATER
PUMP STRUCTURE
SOUTH WALL

FIGURE SWPS-2

SEISMIC LOAD IS BASED ON THE DOMINANT MODE IN THE NORTH-SOUTH DIRECTION WHICH CONTRIBUTES 89% OF THE TOTAL NORTH-SOUTH FORCE

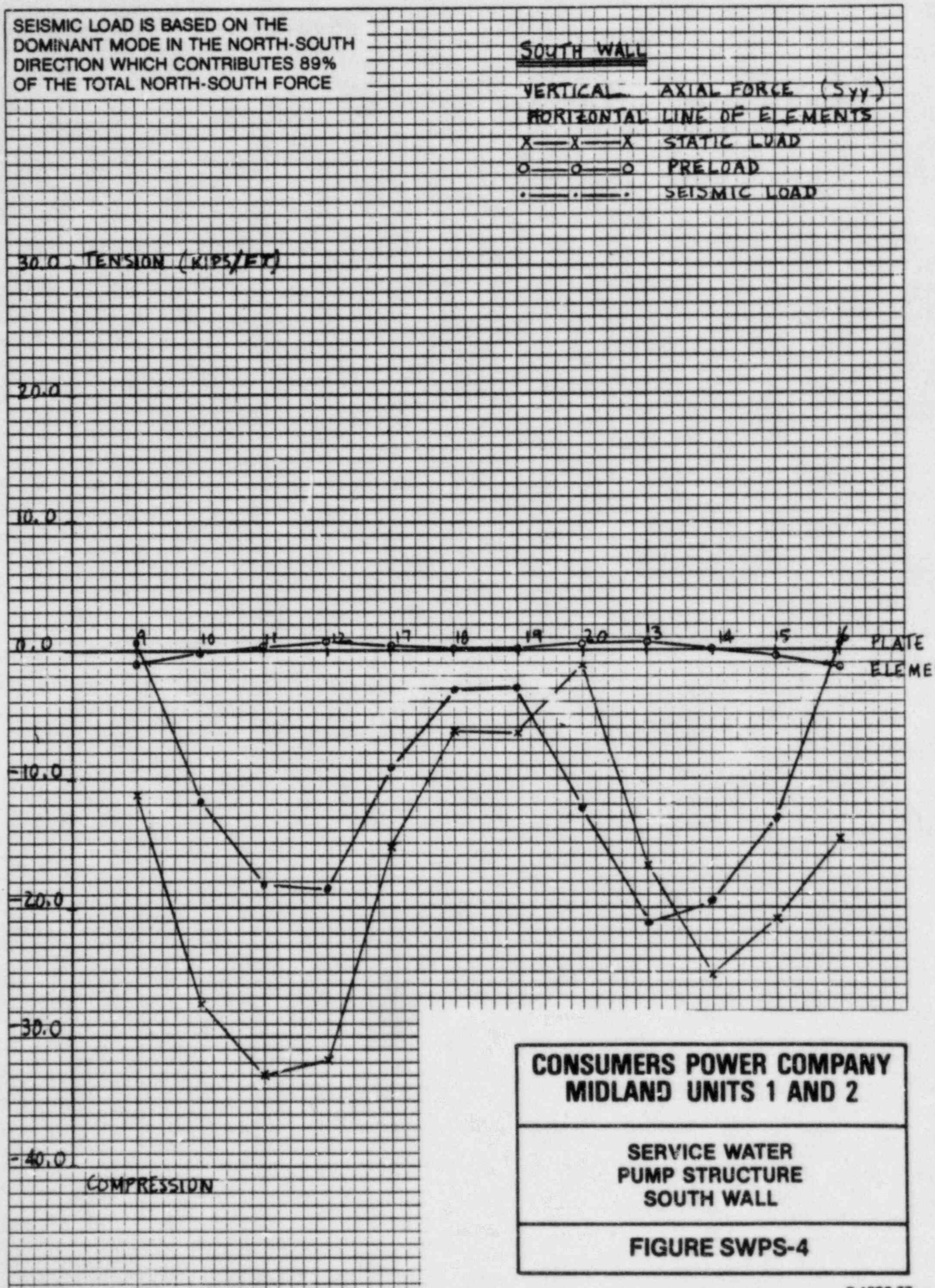


CONSUMERS POWER COMPANY
MIDLAND UNITS 1 AND 2

SERVICE WATER PUMP STRUCTURE SOUTH WALL

FIGURE SWPS-3

G-1986-76

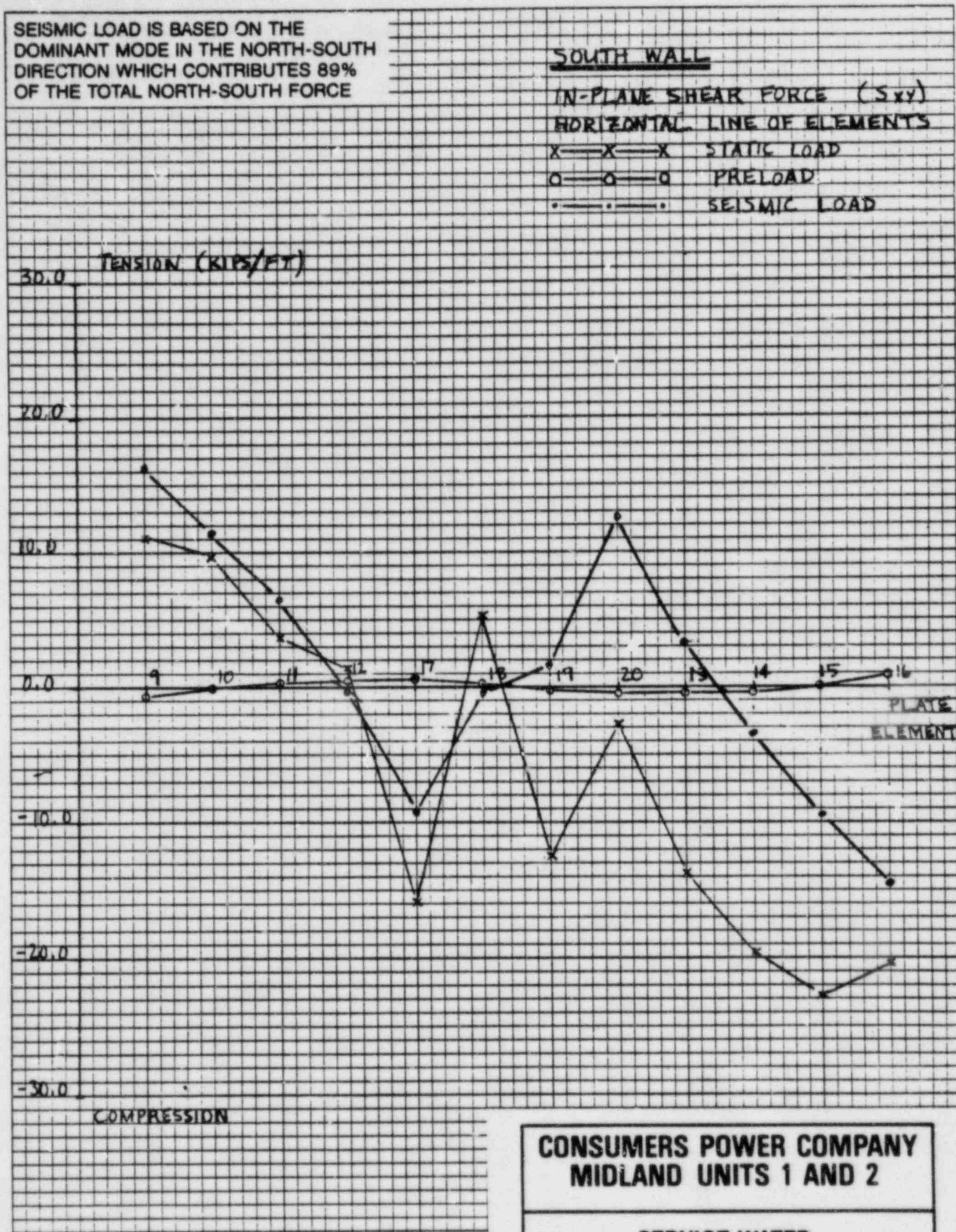


CONSUMERS POWER COMPANY
MIDLAND UNITS 1 AND 2

SERVICE WATER PUMP STRUCTURE SOUTH WALL

FIGURE SWPS-4

G-1986-77

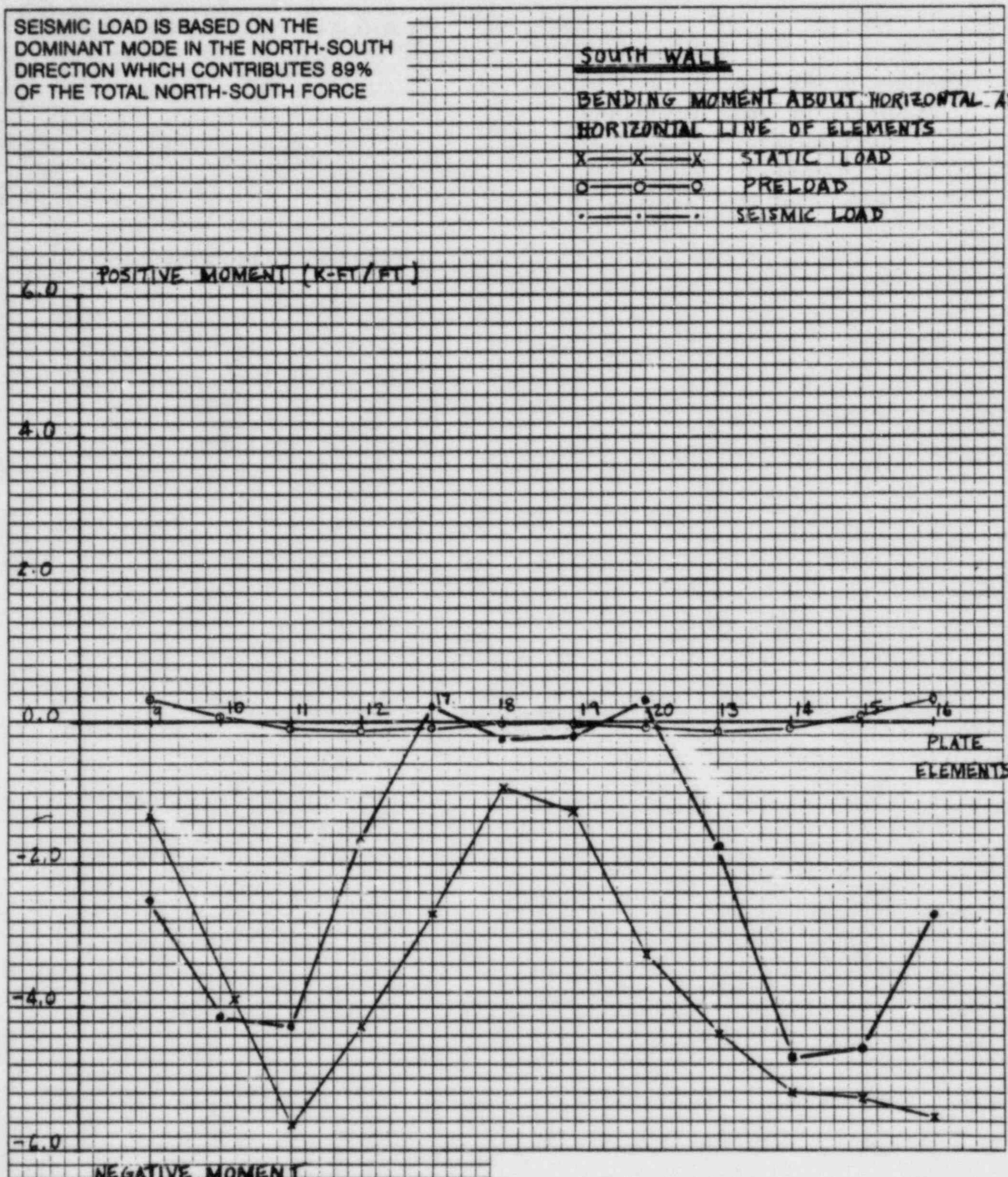


CONSUMERS POWER COMPANY
MIDLAND UNITS 1 AND 2

SERVICE WATER PUMP STRUCTURE SOUTH WALL

FIGURE SWPS-5

G-1986-78



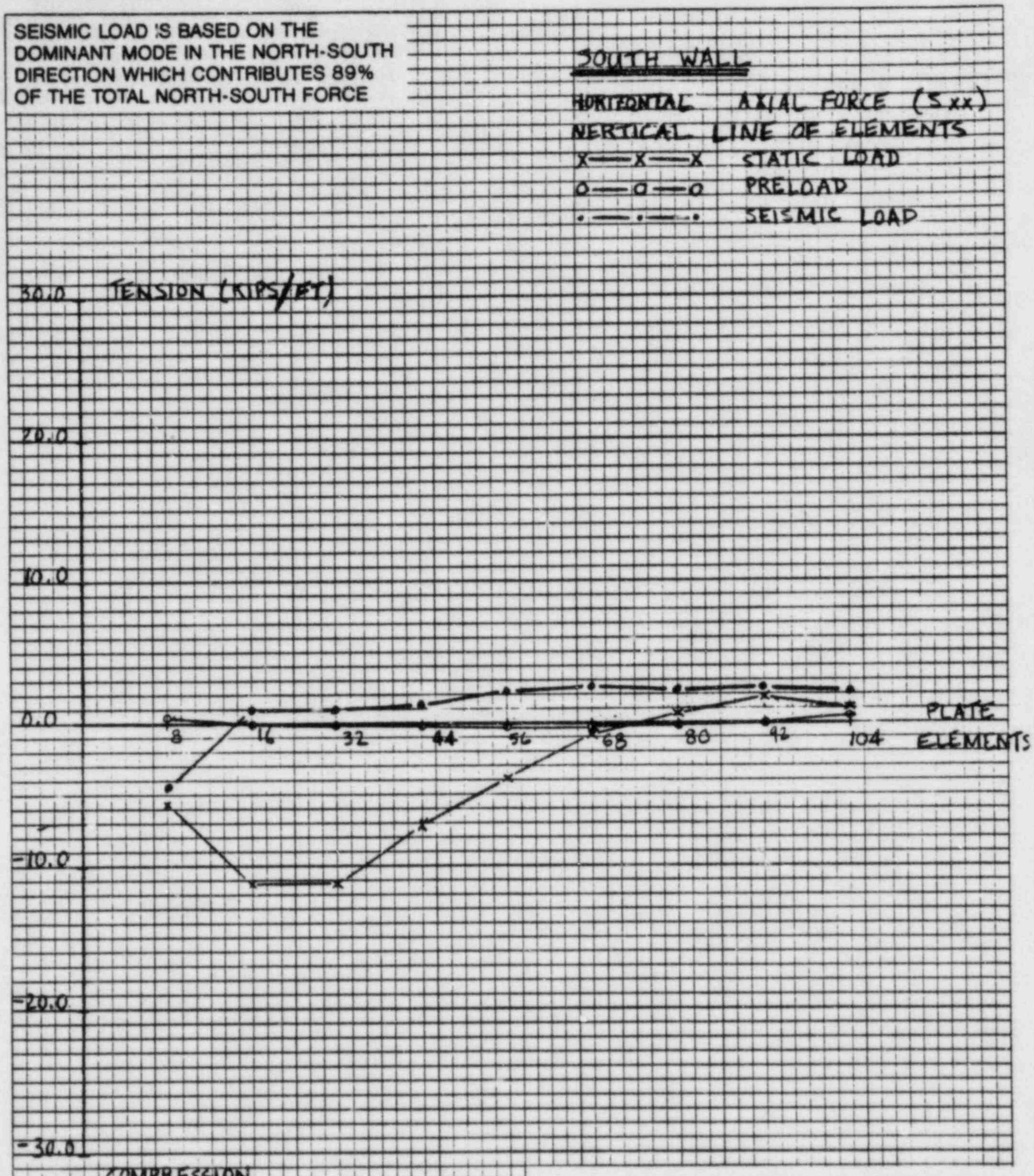
CONSUMERS POWER COMPANY
MIDLAND UNITS 1 AND 2

SERVICE WATER
PUMP STRUCTURE
SOUTH WALL

FIGURE SWPS-6

G-1986-81

SEISMIC LOAD IS BASED ON THE DOMINANT MODE IN THE NORTH-SOUTH DIRECTION WHICH CONTRIBUTES 89% OF THE TOTAL NORTH-SOUTH FORCE

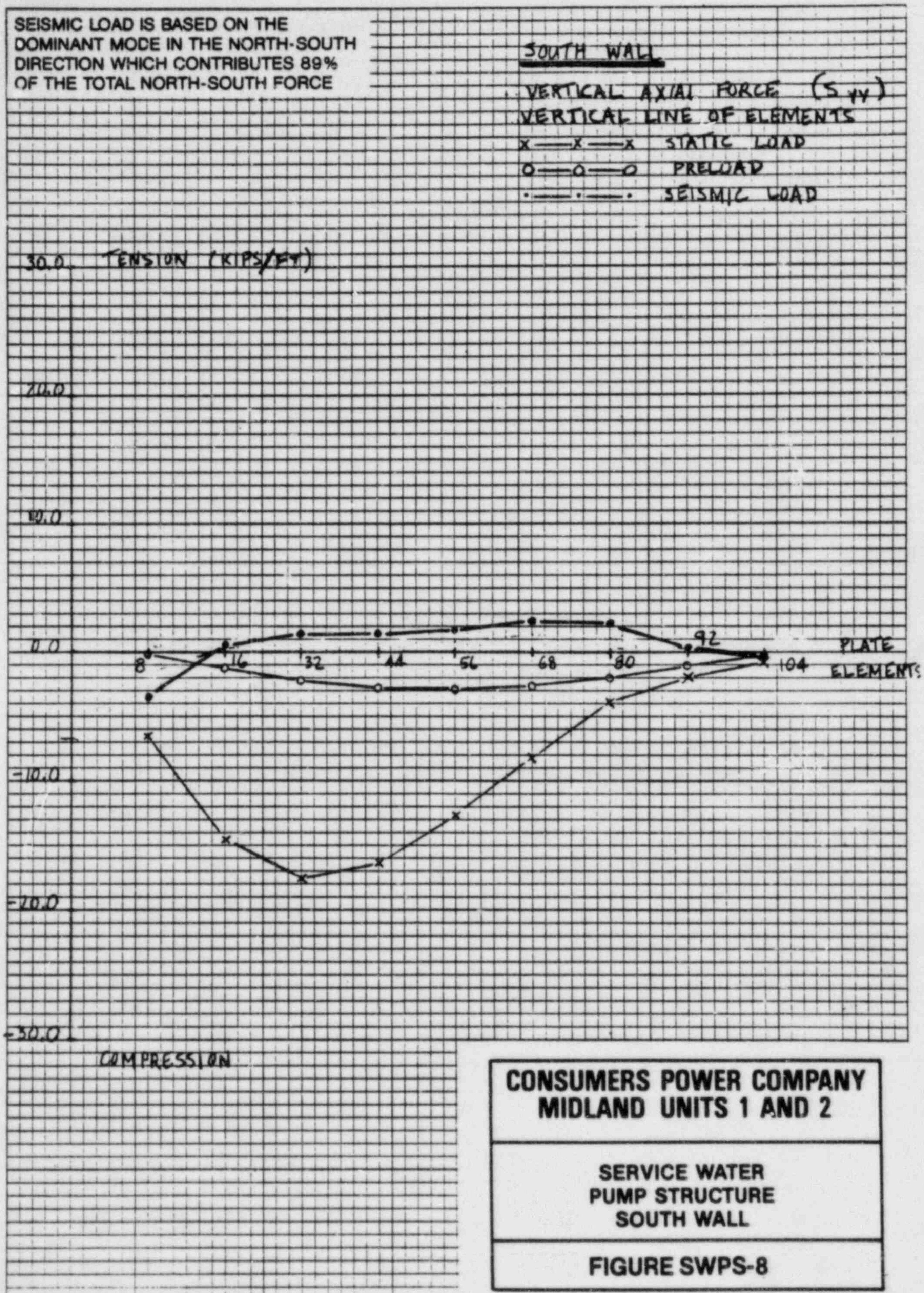


CONSUMERS POWER COMPANY
MIDLAND UNITS 1 AND 2

SERVICE WATER PUMP STRUCTURE SOUTH WALL

FIGURE SWPS-7

G-1986-75



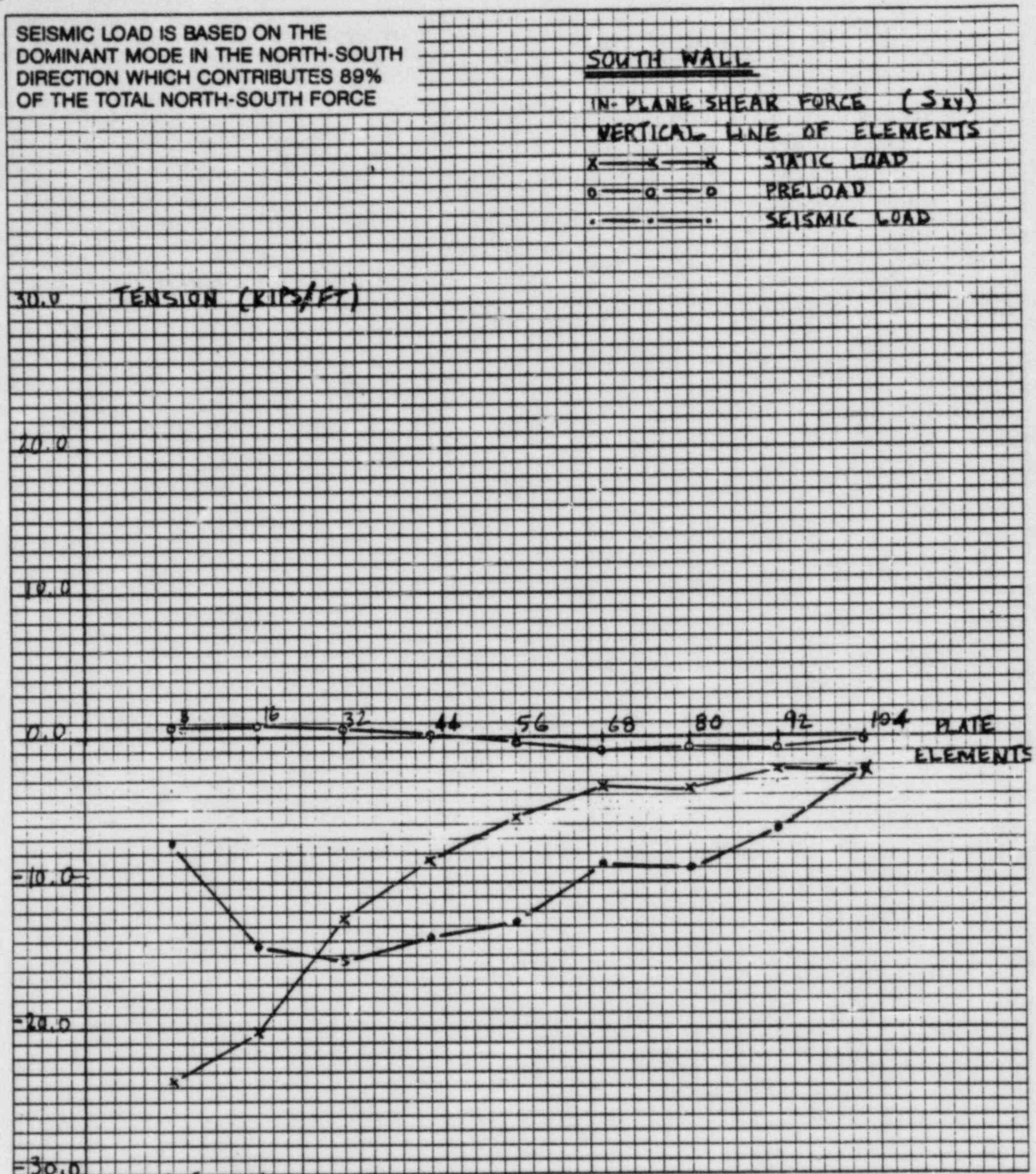
CONSUMERS POWER COMPANY
MIDLAND UNITS 1 AND 2

SERVICE WATER PUMP STRUCTURE SOUTH WALL

FIGURE SWPS-8

G-1986-79

SEISMIC LOAD IS BASED ON THE DOMINANT MODE IN THE NORTH-SOUTH DIRECTION WHICH CONTRIBUTES 89% OF THE TOTAL NORTH-SOUTH FORCE

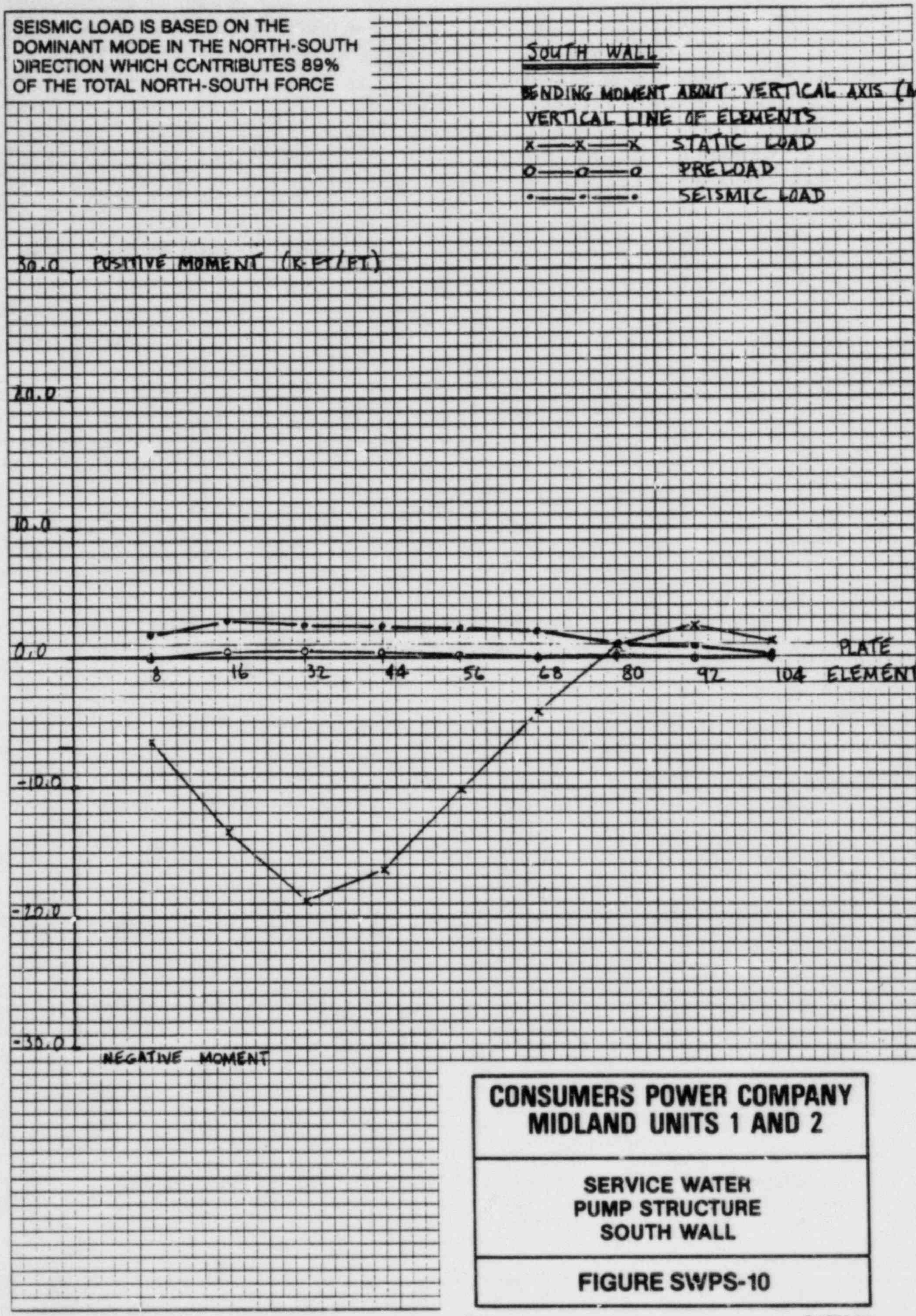


CONSUMERS POWER COMPANY
MIDLAND UNITS 1 AND 2

SERVICE WATER PUMP STRUCTURE SOUTH WALL

FIGURE SWPS-9

G-1986-80

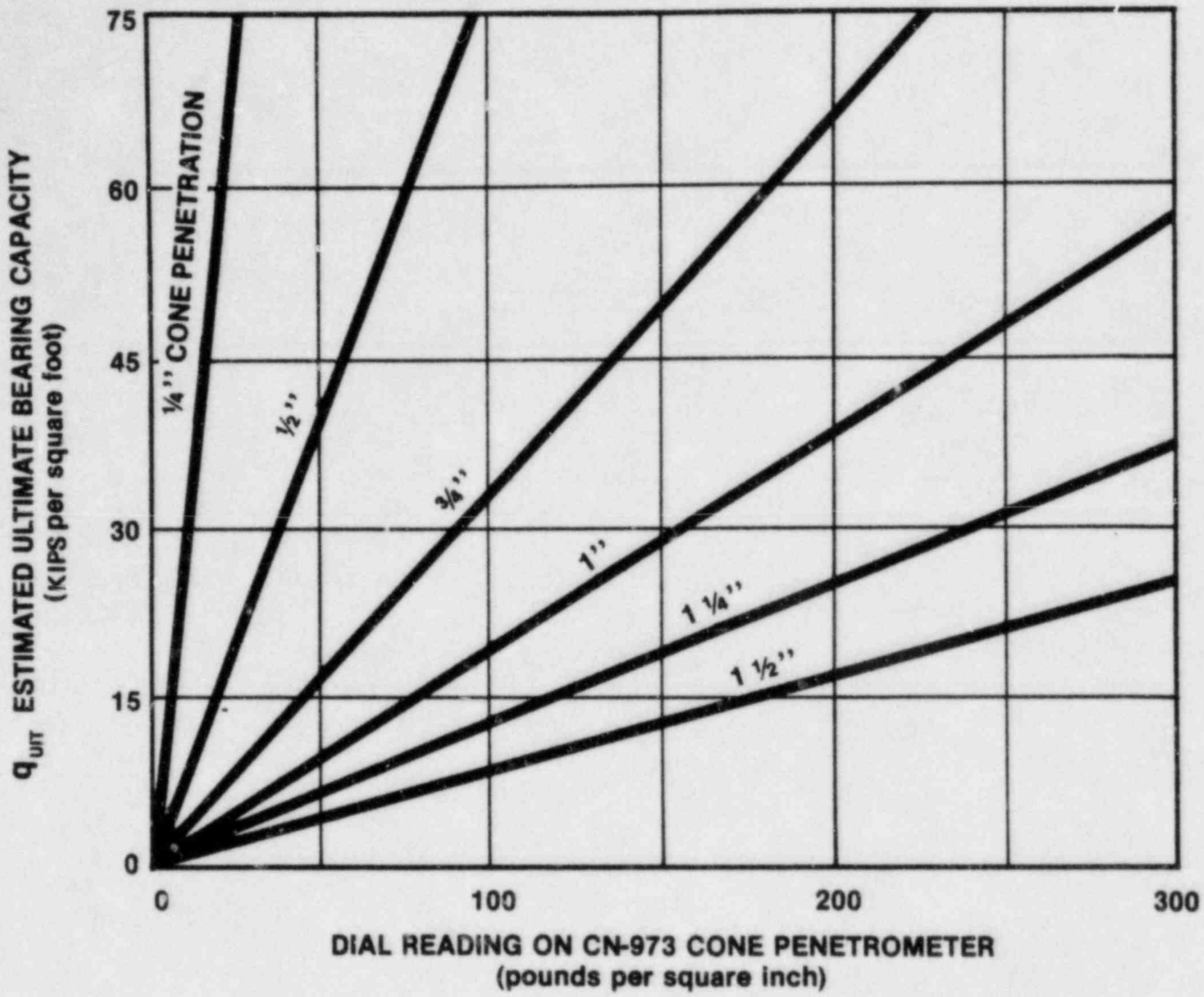


CONSUMERS POWER COMPANY
MIDLAND UNITS 1 AND 2

SERVICE WATER PUMP STRUCTURE SOUTH WALL

FIGURE SWPS-10

G-1986-82

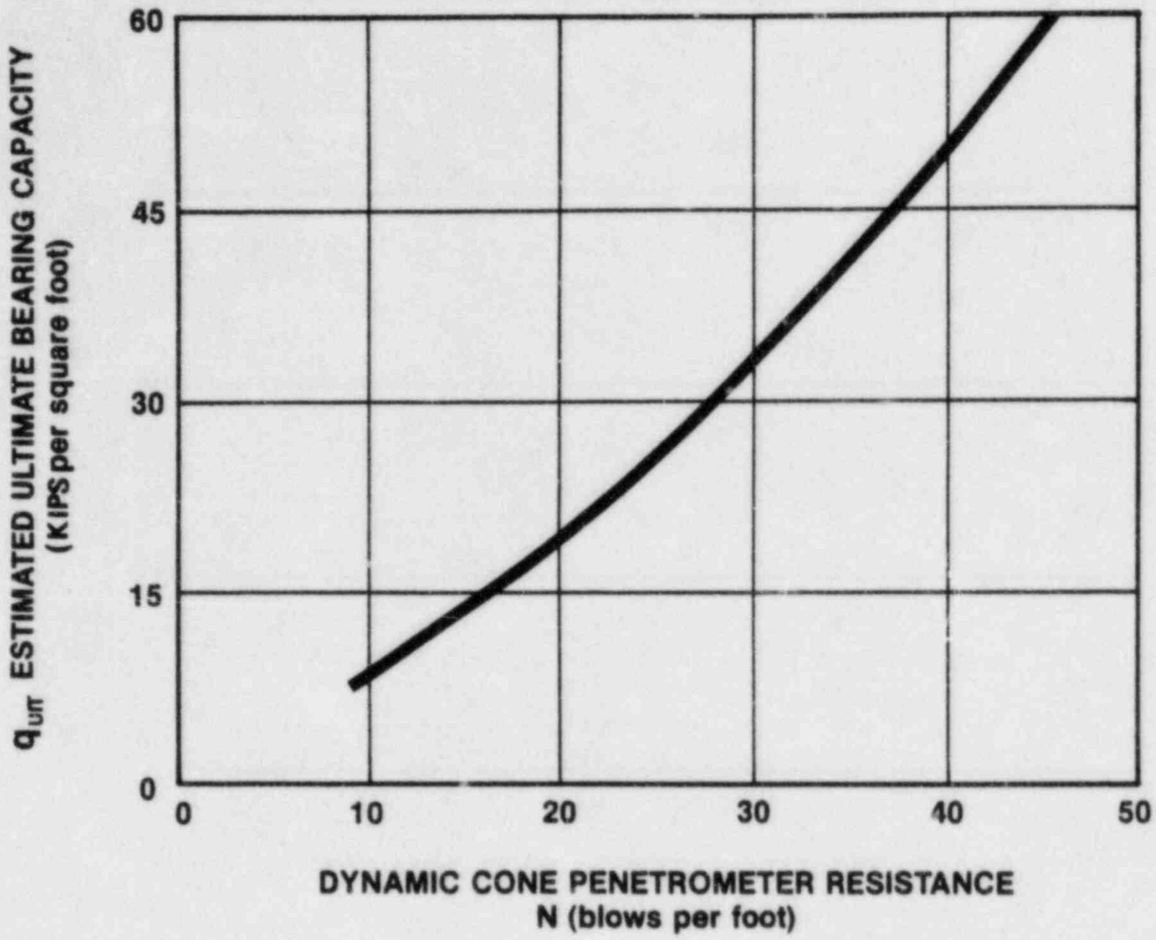


CONSUMERS POWER COMPANY
MIDLAND UNITS 1 AND 2

RELATIONSHIP BETWEEN DIAL READING, CONE PENETRATION AND ESTIMATED ULTIMATE BEARING CAPACITY FOR VICKSBURG CN-973 STATIC CONE PENETROMETER

FIGURE SWPS-11

G-1986-53

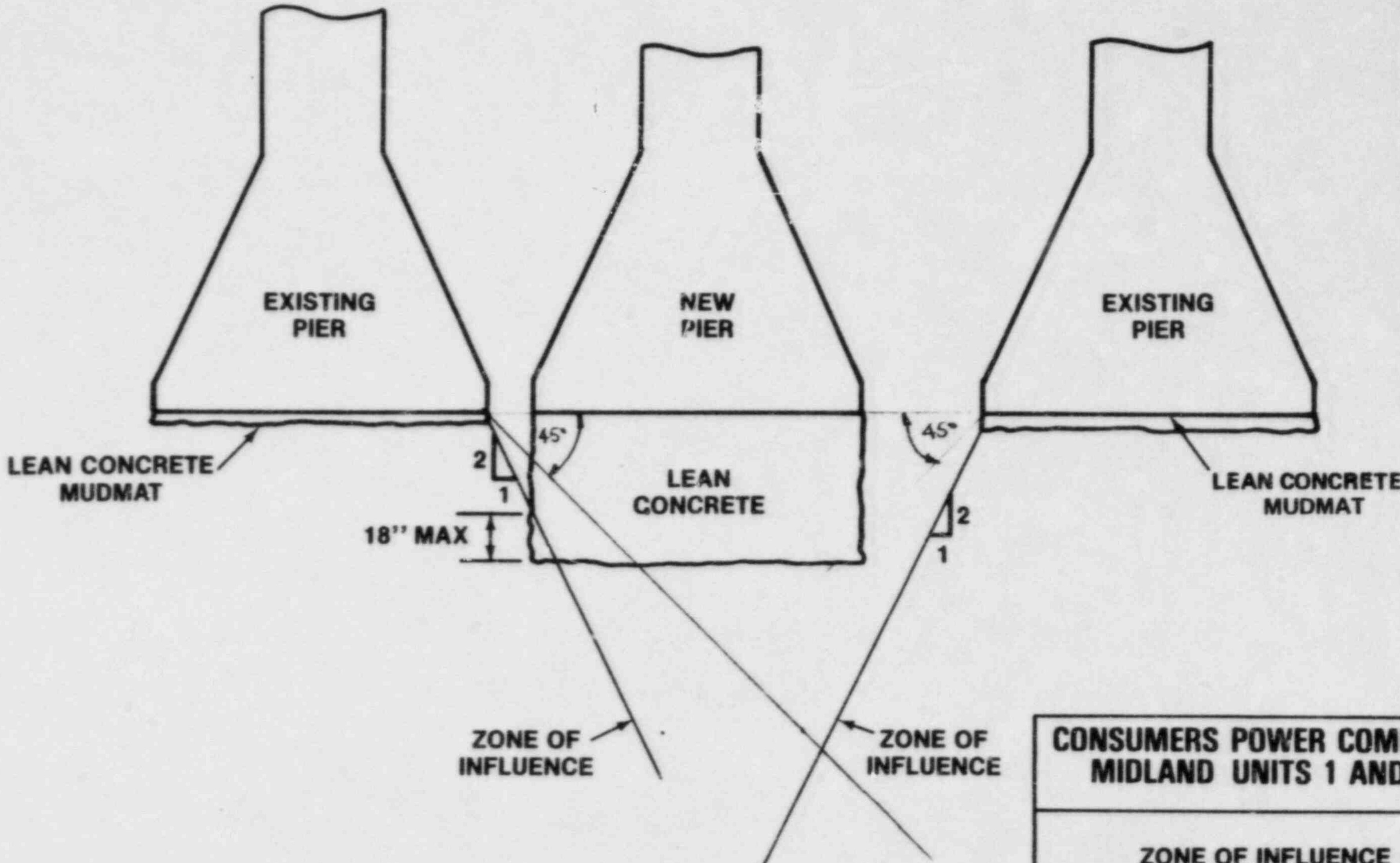


CONSUMERS POWER COMPANY
MIDLAND UNITS 1 AND 2

RELATIONSHIP
BETWEEN ESTIMATED ULTIMATE
BEARING CAPACITY AND DYNAMIC
CONE PENETROMETER
RESISTANCE FOR A 6-FOOT WIDE
STRIP FOOTING

FIGURE SWPS-12

G-1986-54



CONSUMERS POWER COMPANY
MIDLAND UNITS 1 AND 2

ZONE OF INFLUENCE
OF ADJACENT PIERS

FIGURE SWPS-13

G-1988-55