APPENDIX C

Procedures and Drawing for Diameter Verification Pigging Procedure

- Midland Units 1 and 2 Pipe Sizing Operating Procedure Effectivity and Approval, Rev 0, dated October 26, 1981
- Diameter Verification Pigging Procedure, from Northwood's Constructor's, Inc.
- Drawing for Diameter Verification Pigging Operation, from Mears Engineering, Inc.

12/10/81

MIDLAND UNITS 1 AND 2 PIPE SIZING OPERATING PROCEDURE

EFFECTIVITY AND APPROVAL

Revision 0 of this procedure became effective on 10/27/81. This procedure consists of the pages and changes listed below:

Page No	Change	Date Effective
1	Rev 0	10/26/81
2	Rev 0	10/26/81
3	Rev 0	10/26/81
4	Rev 0	10/26/81
5	Rev 0	10/26/81

Approvals

William J Clouter 10/27/81
Written By Date

Donald E Sibbald 11/3/81
Technical Review Date

R C Dery 11/3/81
QA Review Date

MIDLAND UNITS 1 AND 2 PIPE SIZING OPERATING PROCEDURE

EFFECTIVITY AND APPROVAL

1.0 PURPOSE

This procedure provides a description of the activities necessary to verify minimum acceptable diameter of the designated (8") piping at the Midland Units 1 and 2 nuclear power plant.

2.0 SCOPE AND APPLICATION

2.1 This procedure is limited to the acquisition of relative out-of-roundness tolerances which may be used to determine the minimum acceptable pipe diameter of 8" piping systems located at the Midland Units 1 and 2 nuclear power plants.

2.2 This procedure is limited to the verification of acceptable tolerances of pipes at those designated locations. The work will be performed under the supervision of CP Co designated personnel.

2.3 Applicable Documents

The following documents are considered to form a part of this procedure as applicable:

1. Midland Project Quality Assurance Department Procedure F-8M, F-11M, E-1M, F-12M and F-2M

3.0 RESPONSIBILITY

1. The Manager, Midland Project Quality Assurance Department (MPQAD) shall be responsible for review and approval of this procedure.

MIDLAND UNITS 1 AND 2 PIPE SIZING OPERATING PROCEDURE

EFFECTIVITY AND APPROVAL

2. The Site Manager, Midland Project shall be responsible for the implementation of this procedure in accordance with the Midland Project QA Program.

3. The out-of-roundness tolerances shall be verified by an outside contractor. He will be technically qualified to perform this activity under supervision of CP Co designated personnel.

4.0 PERSONNEL REQUIREMENTS

Personnel performing verification of out-of-roundness tolerances shall demonstrate adequate proficiency in their assigned tasks as determined by Site Manager, Midland Project.

5.0 PROCEDURE REQUIREMENTS

1. This procedure shall be controlled in accordance with MPQAD Procedure F-11M and F-12M.

2. Deviations and nonconformances shall be reported in accordance with MPQAD Procedure F-2M. Compliance with 10 CFR 21 and 10 CFR 50.55(e) shall also be in accordance with MPQAD Procedure F-8M.

6.0 TEST CONDUCT

6.1 Witness

The Contractor shall keep the CP Co designated personnel informed of the approximate testing dates and times to the best of his ability.

MIDLAND UNITS 1 AND 2 PIPE SIZING OPERATING PROCEDURE

EFFECTIVITY AND APPROVAL

It shall be the responsibility of the CP Co designated personnel to notify any test witnesses and to establish hold points, if any. The Contractor shall abide by all hold points.

6.2 Test Environment

The inside area of the pipes are to be free of water puddles and any significant amount of rust or debris that may have accumulated in the bottom of the pipe.

6.3 Instruments

The out-of-roundness verification equipment to be used by the Contractor shall be used to measure the pipe tolerances. A description of the instrument used to make the measurements shall be included in the test data.

6.4 Calibration

Diameter (inches)	Percent Decrease in ID
7.781	2.5%
7.582	5.0%
7.343	8.0%

1. Verification Sizing Disk

   a. Check the sizing disk diameters and mark each disk with the percentage decrease from nominal ID according to the table given above.

MIDLAND UNITS 1 AND 2 PIPE SIZING OPERATING PROCEDURE

EFFECTIVITY AND APPROVAL

b. Markings shall be done with an indelible marker.

c. Mark one disk of each size (2.5%, 5.0%, 8.0%) with a pipeline designation number as follows:

8-1HBC-310
8-1HBC-311
8-2HBC-81
8-2HBC-82

6.5 Test Procedure

1. Sizing Assembly

a. Assemble the sizing assembly with either single or multiple sizing disk according to the technical representatives' recommendation.

b. Check the sizing disks markings in Section 6.4C to match the pipeline to be tested.

2. Receiver Cushion

a. At the branch connections into 26"-OHBC-53 or 26"-OHBC-54, place a soft material receiving cushion to catch the sizing pig as it exists from the tested 8" pipeline.

3. Assembly Mounting Flange

a. Place the sizing assembly into the mounting flange.

MIDLAND UNITS 1 AND 2 PIPE SIZING OPERATING PROCEDURE

EFFECTIVITY AND APPROVAL

b. Cover this mounting flange with a blind flange and connect the compressed air supply.

4. Sizing Assembly Propulsion

a. Throttle the air supply valve to force the sizing assembly through the pipeline.

b. Retrieve the sizing assembly from the receiver cushion.

c. Mark each target disk used with a "T" to indicate it as a tested disk.

5. Recording Results

a. Summarize the results by examining each disk for dented indications. All results shall be documented.

7.0 ACCEPTABILITY OF MEASUREMENTS

1. The Contractor or the CP Co designated personnel may void or repeat any set of tests which has doubtful validity.

8.0 TEST RESULTS

1. The test results shall be summarized as described in Section 6.5.5 or repeat any set of tests which has doubtful validity.

2. Permanent documents generated in accordance with this procedure shall be stored and retained by the utility.

DIAMETER VERIFICATION PIGGING PROCEDURE
FOR
CONSUMERS POWER COMPANY
AT
NUCLEAR FACILITY - MIDLAND, MICHIGAN

On October 28, 1981, Mr. J. W. Fluharty, Northwood's Constructors, and Mr. H. L. Fluharty, Mears Engineering, conducted diameter verification pigging operations on four (4) 8.00" I.D. pipelines at the above mentioned facility. The purpose of the test was to determine that the four pipelines had not been flattened due to heavy loads transported across the ground surface above them.

The pipelines were equipped with 150# ANSI flanges at one end and connected to a large diameter pipeline at the other end. Two (2) of the pipelines each had two (2) - 90° elbows and the other two (2) each had one (1) - 90° elbow and one (1) - 45° elbow.

A sizing pig constructed as shown on the attached drawing was run through each pipeline equipped with aluminum sizing discs as shown. The procedure followed for each pipeline is as follows:

1. Pig launcher (as shown on attached drawing) is bolted to the pipeline flange utilizing 4 bolts only.
2. Lubricant is applied to the wide opening of the pig launcher for ease of installing sizing pig.
3. Sizing pig is placed in launcher and driven into 8" pipeline past the face of flange.
4. Launcher is removed and Pressure Assembly is securely bolted to pipeline flange utilizing all eight (8) bolts.
5. Pressure was applied to pig by means of compressed air fed through a 3/4" MUELLER LOCK WING valve and monitored by a pressure gauge on end of pressure assembly.
6. Each pipeline was pigged with less than 20 psi of pressure applied for a duration of 3 minutes to 13 minutes.

The results indicated that each pipeline was of a diameter greater than 7.781 inches and had no obstructions. Upon observation of each disc it was noted that the edge of the discs were slightly beveled. This is attributed to the lead edge of each disc coming in contact with the elbows when forced through the radius. There were no other markings that would indicate an area of diameter change.

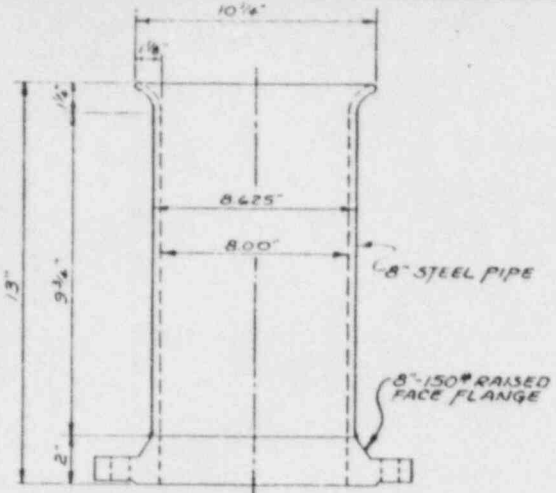
Respectfully submitted

NORTHWOOD'S CONSTRUCTOR'S, INC.

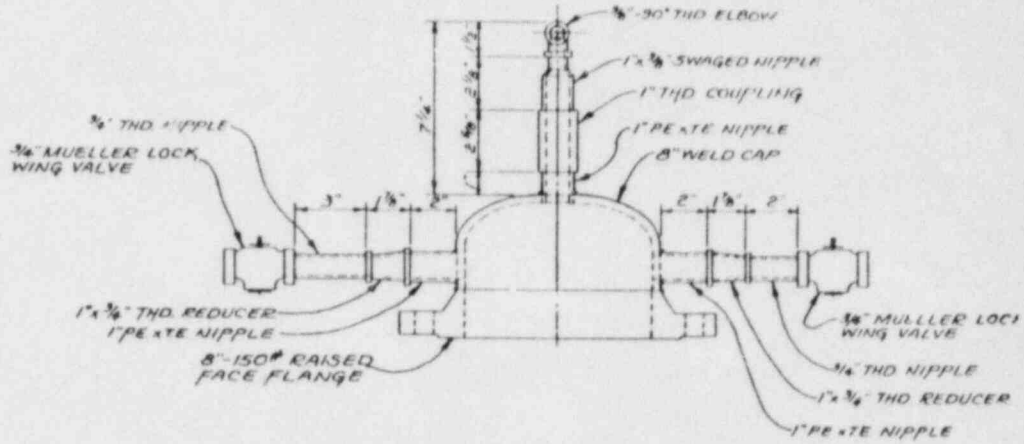
J. W. Fluharty

J. W. Fluharty, President

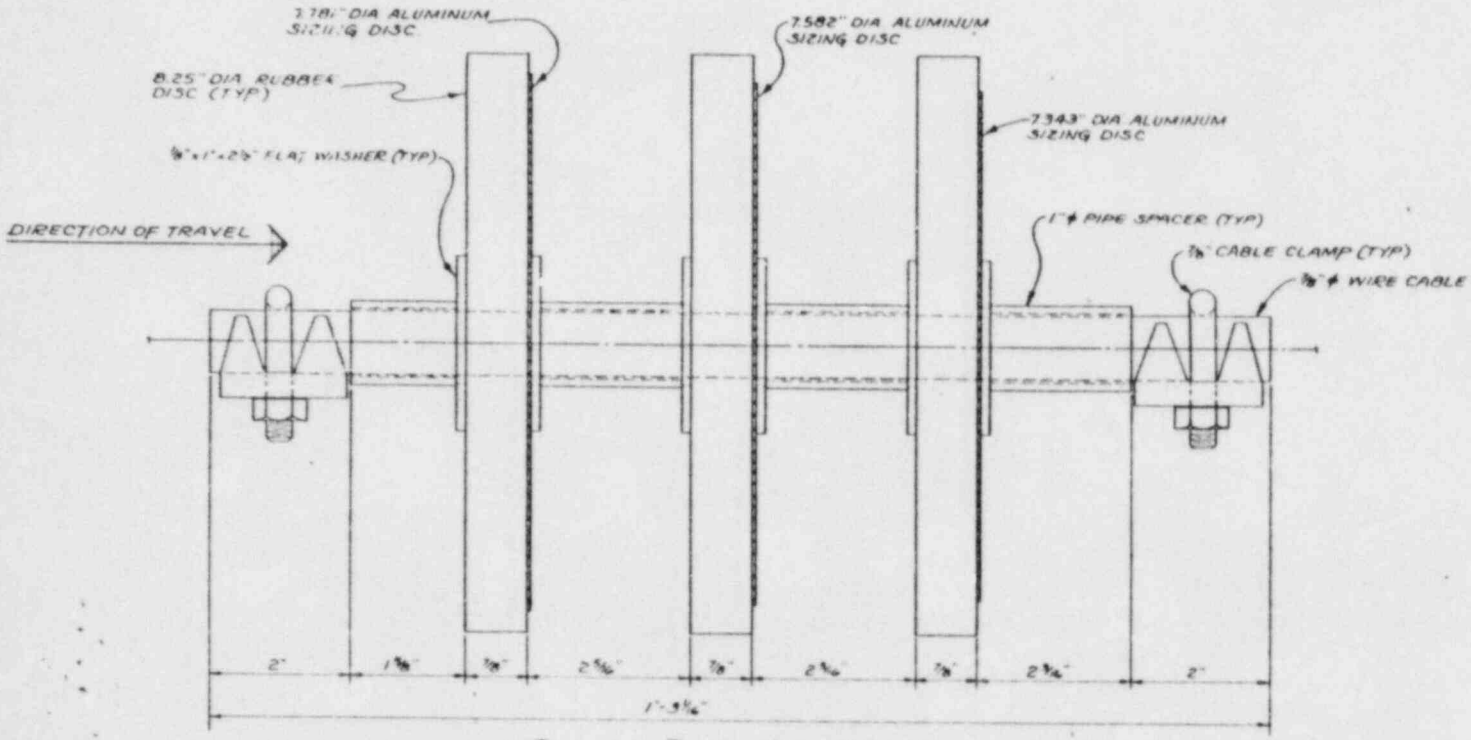
JWF/mls



PIG LAUNCHER
SCALE: 1/4" = 1"



PRESSURE ASSEMBLY
SCALE: 1/4" = 1"



SIZING PIG - SIDE VIEW
SCALE: 3/4" = 1"

DIAMETER VERIFICATION
PIGGING OPERATION
BY
NORTHWOODS CONSTRUCTORS, INC.
3995 EAST ROSEBUSH ROAD
ROSEBUSH, MICHIGAN 48878

CONSUMERS POWER COMPANY
NUCLEAR FACILITY
MIDLAND, MICHIGAN

SCALE: AS NOTED
DATE: 11-1-81

MEARS ENGINEERING, INC.
PO BOX 66, 4500 N MISSION ROAD
ROSEBUSH, MICHIGAN 48878
PH (517) 433-2981

APPENDIX D

Response to Question 49 of NRC Requests
Regarding Plant Fill

The portion of the response which addresses Question 49, Part c2 (Pages 49-3 to 49-7) is included

12/10/81

Reponse (Question 49, Part c)

The measured distance (x) is 325 feet as shown in Figures 24-1 and 24-5, not 240 feet as stated in the Question. The 325 feet is the shortest distance between the critical structures and the recharge source.

Response (Question 49, Part c1)

The analysis given in response to Question 24(a) is based on actual observations of the groundwater level rise in piezometers located at the diesel generator building as compared to records of filling the cooling pond from el 621.8' to 627.4' (Figures 24-3 and 24-4). The calculated apparent permeability of 11 feet per day was confirmed as a representative value by long-term aquifer pumping tests PD-5C, PD-15A, and PD-20 [see response to Question 24(b)]. In summary, it is not necessary to revise the recharge analysis presented in Question 24(a) because the values used are correct. This analysis will be verified by the full-scale construction dewatering test discussed in the response to Question 47(1c).

It should be noted that the permeability values presented and discussed in this response, and the response to Question 24, are expressed in units of feet per day. Feet per second, as cited in the above question, were not used in any calculations or presentations.

Response (Question 49, Part c2) 10

The response to Question 24(c) discussed failure of a dewatering system header line, the concrete pipe pond blowdown line, or the concrete pipe cooling tower line. To respond to this question, we have postulated a nonmechanistic failure of a Unit 2 circulating water discharge pipe near the diesel generator building because it is the largest pipe near a critical structure (Figure 49-1). Potential hazards resulting from this failure were assessed by determining the length of time necessary for the rise in water level to activate a permanent area dewatering well, and the height which the water level would attain at the edge of the critical structure at that time. It was determined that groundwater levels would be significantly below the critical elevation (el 610') when the permanent area dewatering wells would be activated.

Analysis of the water level rise along the eastern side of the diesel generator building assumes the following.

1. The high-level switch in the permanent dewatering well would be activated due to a water level rise of 0.10 feet above el 595'.

2. The change in water level (caused by the pipe failure) to initiate flow to the well is 1.0 foot and is applied instantaneously.

3. The effective porosity of the backfill is 0.30 (Davis and DeWeist, 1966).

4. The failure would occur at the location closest to the structure, yet at the farthest distance from any permanent dewatering well (60 feet).

5. The average depth of flow is 5.5 feet. This depth is the average of the saturated thickness of sand at the well (5 feet) and the saturated thickness at the failure (6 feet).

6. The permeability of the backfill is 11 ft/day. (Refer to PD-20 pumping test, Table 24-1.)

The length of time before the high-level switch on the permanent area dewatering well would be activated due to a water level rise of 0.10 foot can be calculated from the solution to the linearized form of the Boussinesq equation (adapted from Bear, 1972). When the difference in head is small with respect to the average depth of flow, the equation may be solved for the boundary conditions:

$$h = H \qquad X = 0 \qquad t > 0$$

$$h = 0 \qquad X > 0 \qquad t = 0$$

10

The solution adapted from Bear, 1972, is:

$$h = H\left(1-\mathrm{erf}\,\frac{x}{\sqrt{4K\bar{h}t/n_e}}\right)$$

where

h = water level rise at x (L)

H = water level rise at x = 0 (L)

n_e = effective porosity

t = time since initial water level rise at x = 0 (T)

x = distance (L)

$\bar{h}$ = average depth of flow (L)

K = permeability (L/T)

erf = error function

Solving the equation for time shows that it would take 3.3 days before a water level rise of 0.10 feet above el 595' would be detected at the closest permanent area dewatering well. At that time, the area dewatering well pump would be actuated and begin to lower the water level (see response to Question 51).

The height of the groundwater mound along the eastern edge of the structure can be calculated using the following.

1. The pipe consists of welded carbon steel having an internal coating for corrosion protection.

2. The pipe is low pressure (10 psi).

3. The pipe is located 5 feet east of the diesel generator building.

4. The top of the pipe is at el 610' and the bottom at el 602'.

5. The entire cross-sectional area of the pipe is open to the backfill sand (96-inch diameter).

6. The bottom of natural sand is at el 590' (Figure 24-12).

7. The groundwater level at the time of the pipe break is at el 595'. 10

8. The length of the flowpath from the pipe break to the groundwater table is 7 feet.

9. The maximum allowable height of water beneath the Seismic Category I structure is el 610'.

The quantity of water flowing from the pipe into the backfill sand (assuming steady-state conditions occur instantaneously) can be calculated using Darcy's law:

$$Q = KA \frac{h}{L}$$

where

Q = flowrate from pipe (L^3/T)

K = permeablity of backfill sand (L/T)

A = area of flow (cross-sectional area of pipe) (L^2)

h = total head drop between the pipe and the water table (L)

L = distance from pipe bottom to water table (L)

The total head drop between the pipe and the water table is composed of the pressure head (23.1 feet) and elevation head (15 feet) for a total head of 38.1 feet. The calculation shows a total inflow to the backfill sand of 3,011 ft^3/day.

The water level rise along the eastern side of the diesel generator building, 3.3 days after the failure, can be calculated for a vertically downward uniform rate of recharge from an assumed rectangular area, as developed by Walton (1970) from Hantush (1967):

$$h_m^2 - h_i^2 = \frac{W_m \bar{m} t}{15 S_y} \left\{ W^*\left[1.37\ (b_m + x)\sqrt{\frac{S_y}{Tt}},\ 1.37\ (a_m + y)\sqrt{\frac{S_y}{Tt}}\right] \right.$$

$$+ W^*\left[1.37\ (b_m + x)\sqrt{\frac{S_y}{Tt}},\ 1.37\ (a_m - y)\sqrt{\frac{S_y}{Tt}}\right]$$

$$+ W^*\left[1.37\ (b_m - x)\sqrt{\frac{S_y}{Tt}},\ 1.37\ (a_m + y)\sqrt{\frac{S_y}{Tt}}\right]$$

$$\left. + W^*\left[1.37\ (b_m - x)\sqrt{\frac{S_y}{Tt}},\ 1.37\ (a_m - y)\sqrt{\frac{S_y}{Tt}}\right] \right\}$$

where

h_i = initial height of water table above bottom of natural sand (L) 10

h_m = height of water table with recharge above bottom of natural sand (L)

W_m = recharge rate ($L^3/T/L^2$)

$\bar{m}$ = 0.5 ($h_i + h_m$) (L)

t = time after recharge starts (T)

S_y = specific yield of aquifer

$$W^*(\alpha, \beta) = \int_0^1 \mathrm{erf}\left(\frac{\alpha_m}{\sqrt{\tau_m}}\right) \mathrm{erf}\left(\frac{\beta_m}{\sqrt{\tau_m}}\right) d\tau_m$$

b_m = one-half width of recharge area (L)

x, y = coordinates at observation point in relation to center of recharge area (L)

T = coefficient of transmissibility ($L^3/T/L$)

a_m = one-half length of recharge area (L)

To solve for h_m, the following values were used:

h_i = 5 feet

W_m = 351.9 gallons per day per square foot

$$3{,}011\ ft^3/day \times 7.48\ gal/ft^3 \times \frac{1}{8\ ft \times 8\ ft}$$

$\bar{m} = 0.5\ (5 + h_m)$

t = 3.3 days

$S_y = 0.30\ (S_y \approx n_e)$

$\{W^* \ldots\} = 0.094$

b_m = 4 feet

x = 9 feet

y = 0 feet

T = 411.4 gallons per day per foot (11 ft/day x 5 ft x 7.48 gal/ft^3)

a_m = 4 feet 10

Substituting these values into the equation and solving quadratically, the height of water level rise (h_m) is 12.1 feet (el 607.1') along the eastern side of the diesel generator building 3.3 days after the failure.

Therefore, in the unlikely event of a nonmechanistic failure of a circulating water discharge pipe, there is sufficient time for the permanent area dewatering wells in the diesel generator building area to detect and begin removing water before the levels would rise above el 610' beneath the structure.

Response (Question 49, Part c3)

In the unlikely event that the interceptor wells and the backup interceptor wells cannot be repaired, sufficient time exists to replace the system before groundwater levels exceed el 610' beneath critical structures. To demonstrate that sufficient time exists to install a replacement system, a full-scale test will be conducted with the construction dewatering system [see response to Question 47(1c)].

APPENDIX E

Response to Question 34 of NRC Requests
Regarding Plant Fill

12/10/81

QUESTION 34

Supplement your response to question 16 to address how underground seismic Category I piping and conduit are protected from excessive stress due to railroad tracks, construction cranes, and other such heavy vehicles during construction and operation.

RESPONSE

The Seismic Category I piping (conduit) systems are protected against excessive stresses due to construction vehicular traffic, railroad traffic, etc, by using appropriate design and installation techniques. Select granular bedding material is placed and compacted all around the pipe to an elevation approximately 1 foot above the top of the pipe. In areas where it is impractical to use granular bedding material, concrete with a minimum strength of 2,000 psi is substituted.

The buried Seismic Category I piping in the yard includes service water lines, borated water lines, and diesel oil fuel lines. The wall thicknesses for these pipes are primarily based on internal pressure to meet the appropriate ASME code requirements and are considered sound and conservative[2]. 5

The buried pipes are also checked for ring deflection (ovalling) caused by earth loads and superimposed loads such as construction vehicular traffic, railroads, cranes, etc. A ring deflection of 5% of the pipe diameter for externally coated pipes is considered an acceptable limit[1,2]. Ring deflection calculations are performed using a soil density of 120 lb/cu ft for dead loads and Cooper's E-80* railroad loads for live loads. A soil modulus value of 1,900 psi was used in the calculations and resulted in a ring deflection of less than 2% of the pipe diameter. A soil modulus of 1,900 psi corresponds to 85%** compaction determined in accordance with AASHO T-99 specification[1]. Ring deflections for bare steel pipes up to 10% are considered safe[1,4]. The amount of deflection to cause collapse of flexible pipe is about 20% of the nominal diameter[4]. The ring deflection calculations are based on Spangler's method[1]. The soil modulus was treated as a selective constant. The soil modulus is a measure of the passive resistance of the earth at the sides of the pipe on an elastic basis.

The bending resistance of pipes under an external load is relatively unimportant[2]. Reference 4 discusses the design of buried piping and states, in part:

> Satisfactory performances of steel pipe for over a century have proven that the principal function of a structure is to resist loads and that apparent bending stresses based on elastic theory are not of importance in themselves when the ductility of the material in the shell permits deformation without service failure.

Structural calculations have been performed to determine the stresses in the pipe wall for illustrative purposes. The calculations considered Spangler's method for determining the lateral soil pressures on the pipes using a soil modulus of 1,900 psi[2,3]. The results of this analysis are indicated on Table 34-1. This table shows the stresses in 36-inch and 26-inch diameter service water lines. It should be noted that the stresses in pipes smaller than 26-inch diameter will be relatively low and are not critical. Since the stresses due to internal pressure are minimal (about 8% and 5% for 36-inch and 26-inch diameter, respectively), the wall thicknesses of the buried Category I pipes are adequate to withstand the external loads. 5

Seismic Category I conduit used for electrical cables is embedded in concrete duct banks. These duct banks behave differently from buried pipes. The dead load from soil and live load from vehicular traffic (e.g.. railroad, construction cranes, etc) are transferred directly to the subsoil below the duct bank. These loadings only impose insignificant compressive stresses on the concrete.

NOTES

*Cooper's E-80 railroad load, with an impact factor of 1.5, produces a load of approximately 2,000 lb/sq ft at a depth of 6 feet below grade. This is the maximum vehicle load, enveloping the spent fuel cask, the heaviest construction crane (Manitowac-4100W load of about 1,000 lb/sq ft), and the HS-20 truck loadings (200 lb/sq ft) at 6 feet below the grade.

**85% compaction in accordance with AASHO T-99 corresponds to 82% compaction according to ASTM D-1557-66T modified to obtain 20,000 foot-pounds of compactive energy per cubic foot of soil.

REFERENCES

1. Steel Plate Engineering Data, Volume 3, American Iron and Steel Institute (AISI), 1977

2. Steel Pipe Design and Installation, American Waterworks Association, Manual M-11, 1964

3. Spangler, Merlin G. and Richard L. Handy, Soil Engineering, 1973

4. "Design and Deflection Control of Buried Steel Pipe Supporting Earth Loads and Live Loads," Proceedings, American Society for Testing and Materials (ASTM), 57:1233, 1957

5

TABLE 34-1

STRESS IN BURIED PIPES DUE TO DEAD LOAD OF SOIL AND LIVE LOAD FROM COOPER'S E-80 RAILROAD LOADING

	Soil Modulus E'=1,900 psi (85% Compaction AASHO T-99 Specification)	
Pipe Diameter	36 in.	26 in.
Wall Thickness	3/8 in.	3/8 in.
Yield Stress (ksi)	38	38
Stress (ksi)		
Internal pressure (uniform)	+3.1	+2.2
External loads (maximum)	-0.7	-0.4
Ring Bending	±26.9	±20.5
Vertical Displacement (% of Diameter)	1.4%	1.1%