MIDLAND - UNDERGROUND PIPING J. Kane

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MIDLAND PROJECT
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UNDERGROUND PIPING CONCERNS
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ENCLOSURE: ANALYSIS OF BURIED PIPING FOR MIDLAND PLANT UNITS 1 AND 2

On October 6, 1981, we held a meeting with the NRC Staff to present to them a demonstration-type solution to resolve the Staff concerns on underground piping. As a result of our discussions it became clear that the Staff needed additional and more detailed background information to support our solution.

Since the October 6, 1981 meeting, we have continued to work with the Staff on resolving the open issues on underground piping. The enclosure entitled "Analysis of Buried Piping for Midland Plant Units 1 and 2" provides background information to support our proposed demonstration solution and additional engineering information. Included in this report is the following information:

1. Southwest Research Institute pipe profile and ovality measurements.

2. Seismic calculation results for the service water system (SWS).

3. Proposed future monitoring program for the SWS.

4. Inservice inspection plans for the SWS.

5. Proposed resolution for the borated water storage tank pipelines and small diameter safety grade piping.

6. Answers to general concerns regarding the underground piping.

We believe that this report demonstrates that the piping will be capable of performing its safety function throughout the lifetime of the plant. We are

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hopeful that a meeting can be held the first part of January 1982 to discuss the report.

G.S. Keeley for JW Cook

JWC/WJC/dsb

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J. Kane
12/18/81

ANALYSIS OF BURIED SAFETY-GRADE PIPING

FOR

MIDLAND PLANT UNITS 1 AND 2

Submitted by:
Consumers Power Company

12/10/81

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ANALYSIS OF BURIED SAFETY-GRADE PIPING

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MIDLAND PLANT UNITS 1 AND 2

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Previous questions on underground piping were Q.17 & Q45

I. SCOPE

This document addresses the open items identified in the draft Safety Evaluation Report(1) (SER) that are related to Seismic Category I piping buried in the plant fill. These items were identified in Section 3.7.3 (Page 3-7, Items 4 and 5), and Section 3.9.3 (Page 3-13, Items 1 through 6) of the draft SER. The discussion of the items identified in Section 3.9.3 will provide the information needed to resolve the items identified in Section 3.7.3.

The Seismic Category I buried piping systems included in this document are:

a. Service water system lines

b. Diesel fuel oil lines

c. Borated water storage tank lines

A complete list of the included pipe line numbers is included in Table I-1 and their location is shown in Figure I-1.

The control room pressurization lines are also Seismic Category I lines buried in the plant fill. They are not addressed in this document because they have recently been installed (May 1981) and therefore have not been subjected to settlement.

This document addresses open items identified in DRAFT SER (which GEs does not have copy of) but does not address itself to issues still outstanding in response to Q45 [particularly item 45(c)]

II. SUMMARY

II.A BACKGROUND

The NRC staff has expressed concerns for the adequacy of buried safety-related piping at the Midland nuclear plant due to settlement. These concerns were originally expressed in the NRC Requests Regarding Plant Fill[2], Questions 16 through 20. These requests will hereinafter be referred to as "50.54(f) Question(s) . . .". The concerns have been discussed by Consumers Power Company and the NRC staff throughout 1981; in January, May, and October meetings, and in numerous telephone conversations.

To resolve the NRC concerns, extensive measurements have been taken on piping location, elevation, and ovality, so that the current condition of the piping is well-defined. The as-built condition was, however, generally less well-defined. This has made it difficult to establish how much of the current profile was caused by settlement since installation and how much of it is due to as-built condition. Discussions have been continuing on methods for establishing the current stress condition in this piping. As an alternative, a "demonstration solution" has been proposed to establish that the pipe has sufficient dimensional stability to maintain its functional capability.

When will the continuing discussions reach a point that decides the "CURRENT STRESS" condition?

How does this alternate ("demonstration solution") affect the resolution of the "current stress" analysis?

II.B FUNCTIONAL CAPABILITY - PROPOSED SOLUTION

In a telephone conference on August 25, 1981, the NRC concern for maintenance of functional capability was expressed. The telephone conference was to provide the NRC staff response to the demonstration solution approach proposed by Consumers Power Company in an August 10, 1981, telephone conference.

The demonstration solution as proposed August 10, 1981, and changed and supplemented in the October 6, 1981, meeting consists of:

1. Passing a device through the pipelines to

    a) Establish that the pipe has not buckled, or

    b) Manually obtain ovality measurements in large lines

2. Performing periodic hydrostatic testing, including leakage measurement, to ensure pipe integrity

3. Performing periodic flow verification testing to ensure functional capability

The program is to demonstrate whether the pipe has retained sufficient dimensional stability to maintain the system's functional capability. Standard Review Plan SRP 3.9.3[3] recognizes the validity of this approach and provides guidance. This guidance includes, in part, the statement that, "Since . . . the treatment of functional capability, including collapse and deflection limits, is not adequately treated by the Code for all situations, such factors must be evaluated by designers and appropriate information developed . . ." (code requirements are discussed in more detail in Section III.A.4.d). This guidance indicates that an alternative to determining stresses is to demonstrate that the areas of discontinuity retain sufficient dimensional stability. Teledyne Engineering Services (TES) stated in a letter to Consumers Power Company, "Retaining sufficient dimensional stability is, in fact, the only basic question to be answered and is directly related to assuring functional capability of the piping" (see Appendix A).

The program has demonstrated acceptable current dimensional stability by inspecting the pipe to determine cross-sectional shape (ovality) which is directly related to stability. These results are discussed in Section III.A.1. Continued functional capability will be demonstrated by flow verification tests to be conducted during plant operations. An additional check of functional capability will be provided by the inservice inspection (ISI) program (see Section III.A.6). This type of testing will not explicitly show that no pipe deformation is occurring; rather, it demonstrates that deformation sufficient to reduce the flow below minimum requirements has not occurred.

To ensure that the system contains sufficient margin to prevent loss of functional capability, conservative acceptance limits have been established using code guidelines and standards for buried pipe in American Water Works Association (AWWA) Specification M11[4]. We have also based our acceptance limits on the general piping standards for bending nuclear pipe according to ASME Section III codes. Our confidence in these limits is supported by the results of various pipe experiments reported by E.C. Rodabaugh and S.E. Moore in NUREG/CR-0261[5].

The introduction to NUREG/CR-0261, under "Relevance to Functional Capability," states, "We do <u>not</u> have any test data in which large enough displacements were applied to produce significant reductions in flow area; e.g., 50% reduction of flow area. We would guess that to produce such a condition in straight pipe by application of a moment load, a rotation of 30° or more over a length of about 2 pipe diameters would be necessary." The NUREG discussion then states, "The moment to produce this 'kink' in the pipe might not be much greater than the 'limit moment'; the displacement would be far in excess of any normally-used criterion for defining a 'limit moment.' It is important to note that exceeding the deflection corresponding to a limit moment does not necessarily mean that functional capability will be significantly impaired." These conclusions indicate that it would take far more deflection than can conceivably occur in buried pipe due to settlement to significantly impair the pipe's functional capability.

The AWWA conclusions are discussed in Section III.A.2.

II.C ANALYTICAL SOLUTION DIFFICULTIES

The difficulty with analytical solution is separating the as-built condition of the piping (i.e., the local installation discontinuities) from the deflections due to settlement. The misalignments and discontinuities reflected in the field data are inherent in the fabrication process. Project quality records indicate that the piping was fabricated and installed within acceptable standards[6] (±5/32 inch, local mismatch; ±3/32 inch, overall mismatch; ±2 inches, overall location).

± 5/32 = 0.156"
± 3/32 = ±0.094"

The calculated stresses based on field deflection measurements cannot be relied upon because the measurements include installation discontinuities as well as soil settlement. For example, allowable angular mismatches of weld joints are magnified over a long length of pipe and can appear as "knees" along a straight line (see Figure II-1). Assuming that these knees are due to soil settlement results in concentrating the curvature at the knees, thereby significantly overestimating the stress levels. Deflections of this magnitude resulting from settlement would result in gross local deformations that would have been apparent during examination. Using the calculated stresses, these deflections would produce ovality well beyond 8%. ?

Ask T. Cappas

The analytical solution using empirical data is further complicated by the measuring tolerance. Measurement inaccuracies can cause apparent pipe oscillations to be overemphasized. In 1979 profiling was done to approximately ±1/4-inch accuracy, with measurements every 10 feet. A parametric study over a 20-foot span using worst case measurement errors (1/2-inch deflection) yielded a calculated elastic stress of 55 ksi. This stress alone is greater than the allowable stress. The latest reprofiling has been done to a tolerance of ±1/16-inch, but the number of survey points has also been increased, thus decreasing the flexibility and increasing the sensitivity to the measurement tolerances.

To develop a computer model of the piping, a rigid restraint in the vertical plane forces the pipe into the measured profile configuration at the survey locations along the pipelines. This does not allow the pipe to flex according to its geometric and material properties. These abrupt changes (knees) at the

question

survey locations concentrate the pipe curvature near these local discontinuities, resulting in artificially high local stresses. Thus, fitup and installation differences (discontinuities), assumed to be settlement, will result in erroneous, very high calculated stresses.

Structural Mechanics Associates[7] performed calculations by modeling the pipe as a beam on elastic foundation to determine the soil loading necessary to cause the measured deformations. This study showed that soil loadings as much as three times the conservative estimate of the soil capacity would have been needed (see Figure II-2). The limited information available about presettlement as-built conditions proves that we have an unrealistic calculational solution. The modeling technique was further refined to include nonlinear aspects of the pipe and soil parameters. The computer results would not converge on the measured pipe configuration. This demonstrates again that, in certain locations, the measured pipe profiles could not occur due to soil settlement alone.

The problem of developing an accurate analytical model is complicated by the presence of the soil around the pipe and the soil/pipe interaction. The soil characteristics such as friction and soil support mechanisms are very difficult to approximate. As the pipe tries to deform (ovalize), pressure develops between the pipe and the soil which counteracts the ovalization and maintains the pipe geometry and, thus, functional capability.

The basic analytical problem is how to separate the as-built condition of the piping from the deflections due purely to settlement. We have concluded that the profile data cannot be used in a traditional flexibility analysis unless an agreement can be reached on a method to accomplish this separation.

To understand this calculation & comparison it would be helpful to know the actual pipeline involved, where the distances are measured along that pipeline (i.e. in the surcharged area?), elevation of 0 datum for settlement, properties (i.e. moment of inertia) of the pipe. It is also unclear how the "conservative soil capacity estimates" shown on Fig. II-2 were determined and if they should properly be compared to the soil loading calculated by modeling the pipe as a beam on an elastic foundation.

III. DETAILED DISCUSSION

A. SERVICE WATER SYSTEM

1. Profile and Ovality

In 1979, a profile of one line in each trench was done. The profiling was done to approximately ± 1/4-inch accuracy with measurements every 10 feet.

App. B gives results for 6 of the 22 SWS pipelines

In August 1981 new profile and ovality measurements were started in all service water system piping. This was to obtain more accurate information and to profile the condition of all lines which had not been measured. Reprofiling and ovality measurements of the service water supply and return lines were completed in October 1981 (see results in Appendix B).

The 1981 profiles involved cleaning the interior surface and marking it at a minimum of 5-foot increments for measurement. Measurements at some locations, particularly in elbows, were as close as 1.5 feet apart. Measurements were also taken 2-1/2 inches on either side of pipe welds. The tolerance on the measurements was estimated to be ±1/16 inch. (See Section II.C for discussion of the effect of these tolerances.)

To do the 1981 profiles, a unique apparatus was developed by Southwest Research Institute (SwRI). The pipe elevation profile measurement system developed by SwRI for this effort is shown in Figure III-1. The device uses a pressure transducer moved within the pipe and positioned on the pipe bottom (as determined using a bubble level on the transducer). It measures the differential pressure between a reference water column and a water column ending at the transducer. The system used in 1979 was similar, but involved a visual measurement rather than sensed differential pressure.

Ovality is measured at the same locations as elevation with another SwRI instrument (See Figure III-2). The device uses rotating arms to obtain both maximum and minimum diameters. Their azimuth orientation is recorded with the azimuth location of the longitudinal fabrication weld. Fittings were measured using the same measurement arm; however, this required removing it from the rolling platform (dolly) which was used in straight pipe sections for accurate positioning.

The ovality measurements for both straight pipe and fittings have been plotted and are shown along with the profile data in Appendix B. They generally were less than 2% as compared to the manufacturing tolerances of 1% for straight pipe (ASME SA155) and 1.76% for fittings (ANSI B16.9).

Some piping fabricator catalogs (NAVCO, in particular) include a note that ovality may change due to handling. They indicate that for pipe manufactured to a 1% tolerance, experience shows that 2% or more ovality is normal for pipe installed in a trench ready for backfill.

For the ovalities measured at Midland, there is no way to determine how much is due to settlement, but in any case the ovalities measured are within the range considered normal for newly installed pipe.

III.A.2 Ovality/Buckling

The bending stresses induced in the buried pipe by settlement are similar to fabrication bending stresses because the support provided by the surrounding soil is similar to the radial support provided by a bending mandrel. The acceptance criteria for ovality that we propose to use is 8% as stated in ASME Section III codes (NC-4223.2 and NC-3642.1) as the tolerance for installation and fabricated bends.

Most codes that discuss ovality relate it to the fabrication of bends. Most of the codes limit the ovality in the bend area to be a maximum of 8% (ovality defined by $(D_{max} - D_{min})/D_o$). The bending/forming requirements in ASME Section III, ANSI B31.1, B31.3, and PFI ES-3 all incorporate this limit.

Some of these codes imply that this limit is a "good practice" tolerance rather than a limitation imposed because of material ductility considerations. For example, ASME SA155 fabrication requires forming to a cylinder and joining with a full penetration weld. This indicates the pipe material can take a permanent set in a manufacturing process substantially in excess of the 8% limit without sustaining damage. Likewise, ASME SA106 requires flattening a section of pipe between parallel plates to a diameter approximately one third of the original diameter without any evidence of damage. ANSI B31.1, Paragraph 104.2.1.c and ASME III, NB-4223.2, provide for flattening greater than 8%.

It is evident from the above discussion that the codes indicate that considerable deformation can be sustained without damaging the integrity of the pipe, and that restricting the ovality to 8% is conservative when the actual ductility of the pipe is considered. It should be noted that the existence of ovality does not in itself imply a structural failure of the pipe.

It should also be noted that the codes, and hence the code considerations of bending and ovality, are based on an assumed failure where the moment carrying capability of the pipe is a maximum. This presumes that after the instability point is reached, the conditions which caused the instability continue to prevail as in a load-controlled situation and that deformation will increase without limit. Settlement, however, is a deflection-controlled condition where the settlement induced secondary stresses may cause localized yielding, but are not self-driving to failure.

In the letter from TES (Appendix A), the applicability of the current ASME III Code requirements were discussed in the following manner.

> For the piping systems we are addressing here it is important to recognize that the entire buried pipe was subjected to soil settlement. This is really a different situation than that addressed in current Section III criteria (NC-3611.2(f)) for non-repeated <u>anchor</u> movements. Many of the reasons for this difference have been discussed above and demonstrate the important variations between non-repeated anchor motions (building settlement for a non-buried pipe) and general soil settlement.

NUREG/CR-0261[5] provides an experimental relationship between moment and ovality just before buckling. The experimental results of Reference 12 of the NUREG defines flattening as the decrease in the diameter in the plane of the moment divided by the original diameter $(D_o - D_{min})/D_o$. This formula has been verified by telephone conference between J.F. Sorensen, author of Reference 12, and W.J. Cloutier of Consumers Power Company.

This definition of flattening is different from the definition of ovality used throughout this document, which is based on ASME $(D_{max} - D_{min})/D_o$. The difference results in the flattening comprising half of the ovality. The NUREG states that the flattening is a function of the diameter-thickness ratio (D/t), and is shown to be 4.5% for D/t = 100 for small-scale tests and 5.5% for large-scale tests. This represents the flattening at the maximum load just before buckling.

This is Ref. 12 in NUREG-2161

The underground service water system piping D/t varies between 69 and 96. Considering the calculation method of Reference 12, the ovality reported in the experiment would be 9 to 11%.

All analyses/experiments discussed thus far reflect analytical models or experimental conditions which conservatively neglect stabilizing influences present under actual site conditions. These influences include the following.

a) The assumption of an infinitely long pipe neglects the restraint provided by adjacent cross sections undergoing a smaller degree of ovalization.

b) The minimum specified yield stress values used in the analyses/experiments neglect the extra capacity indicated by the stress-strain data from the actual pipe material used at Midland.

c) The increase in the predicted buckling resistance of the pipe due to the service pressure was neglected.

d) The confinement and cross-sectional support provided by compacted fill surrounding the pipe was neglected.

The cumulative conservatism represented by these four stabilizing influences is sufficient to raise our confidence about the appropriateness of the 8% acceptance criteria established to determine a pipe's worthiness as safety-grade piping.

The code most directly applicable to steel pipe buried in fill is AWWA M11[4]. In Chapter 8, Earth Loads on Steel Pipe, the following excerpts discuss the mechanism by which buried steel pipe support loads.

Although the maximum load-carrying capacity of flexible pipe depends to some extent on the wall thickness and its section modulus, the pipe, by deflecting, is able to make full use of the load-carrying ability of the earth surrounding it. As the pipe may change shape without failure, it transfers part of the vertical load into a horizontal or radial thrust which is resisted by the passive pressure of the earth at its sides as these move outward. When the wall itself is rigid, this movement may not occur. It follows that the rigid pipe must carry the whole load itself, whereas the flexible pipe divides the load with the earth enclosing it. Therein lies the inherent difference between rigid and flexible behavior and the explanation of why the classical bending-moment formulas apply to the analysis of rigid pipe but not to the analysis of flexible pipe.

At this point, when deflection is mentioned, the engineer accustomed to thinking in terms of flex ure or bending-moment formulas in rigid construction is likely to contend that permanent deflection can occur only after the yield point has been passed and that, therefore, a pipe so stressed has failed structurally and is dangerous. The simplest rebuttal to this argument is to recognize that the steel in a finished pipe has, in the manufacturing process, been cold coiled, uncoiled, bent, curved, or twisted a number of times and has been stressed beyond the yield point each time; yet, after all these operations have been completed, the finished steel pipe is used for all manner of high-pressure work without fear or hesitation.

> If the engineer still is hesitant to restress a part of the finished pipe wall beyond the yield point by slightly deflecting it underground, let him consider what happens to the test specimen by which the pipe strength is measured according to specification. Usually it is sliced as a ring from the end of a finished pipe, cut at one side, uncurled from the circle into a flat piece, and then put in a tensile-testing machine which proceeds to show that after once more passing the elastic limit, the steel still possesses the specified strength. In a way, the deflection underground is simply a finished forming operation.
>
> Therefore, where steel pipe such as is here discussed is concerned, the word "failure" must define a state of falling short of satisfactory performance and not a state in which localized stresses appear to pass the yield point of the material as judged by the results of bending-moment formula analysis.

These excerpts support the provision in SRP 3.9.3 that the pipe is acceptable as long as it retains sufficient dimensional stability to ensure functional capability.

More specific to ovality tolerance, AWWA M11, Section 8.23.1 states, "Deflection of unlined pipe, or of pipe lined after installation, may safely reach 5 per cent of nominal diameter." This deflection is nominally equivalent to 10% by the formula $(D_{max}-D_{min})/D_o$ used by ASME and is based on failure of the coating, not any limitation of the pipe.

AWWA M11, Chapter 8 also states, "Real collapse failure of steel pipe does not occur under earth loads until a condition is reached where the vertical diameter has been decreased about 20 per cent of the nominal diameter and the horizontal diameter has been increased a similar amount."

From the foregoing guidance based on research, experimental results, and years of experience, we feel that applying the 8% ovality criteria recommended by ASME is a very conversative acceptance criteria for ovality due to settlement.

III.A.3 Future Settlement

a) Predicted values

The responses to 50.54(f) Questions[2] 4 and 27 contain a discussion of the methods used to estimate future settlement. The response to Question 27 includes the following description of the two settlement components (Figure 27-1 is attached as Figure III-3):

> The distinction between [primary] consolidation and secondary compression settlement is made on the basis of the physical processes which control the time rate of settlement. In primary consolidation settlement, the time rate of settlement is controlled by the rate at which water can be expelled from the voids. In the case of secondary compression settlement, the speed of settlement is controlled largely by the rate at which the soil skeleton itself yields and compresses. The transition time between these two processes is conveniently identified as that time when excess pore water pressure becomes essentially zero. This time, denoted as t_{100} is shown in Figure 27-1.
>
> It has been observed in many laboratory and field measurements that the relationship between the magnitude of secondary compression and time is approximately a straight line on a semilogarithmic plot after the primary

consolidation has been completed, as shown in Figure 27-1. Thus, the settlement ΔH can be expressed approximately as:

$$\Delta H = -C_\alpha \log t_2/t_1$$

where t_2 and t_1 are two specific time periods on the extrapolated secondary compression line and C_α is the settlement per log cycle of time during secondary compression.

The response to Question 27 contains a much more extensive discussion of settlement, prediction method, and the basis for conservatism and accuracy than is presented here in these excerpts. Supplemental Figures 27-51 through 27-198 show settlement vs log time plots for the diesel generator building. They show that the fill is in the secondary compression settlement phase.

what about those portions of pipe on fill that was not surcharged

In March 1980 a preliminary settlement estimate was provided for calculating future pipe stresses. The estimated settlement envelope was determined based on measured time-settlement data from Borros anchors buried in the plant fill. This estimate resulted in a settlement envelope of 0 to 3 inches for the 40-year plant life to be used in analyzing the piping buried in the plant fill. This estimate did not include settlement that occurred prior to March 5, 1980, nor has settlement since March 5, 1980, been used to adjust the predicted value of future settlement.

Check response to Q 27 — How is this 0 to 3" prediction used in evaluating pipe stresses

III.A.3.b) Monitoring program for PIPE SETTLEMENT
— what about lines for BWST?

The service water system (SWS) future settlement shall be monitored at the terminal ends, before the first anchor point of each pipe as it enters the buildings. The first pipe anchor inside the building is the most rigid anchor in the system, compared to the soil bedding outside the building. Therefore, it is most susceptible to high stress.

The settlement to be monitored will be the differential settlement between the pipe anchor and the underground piping. This settlement limit shall be established from the amount of settlement (Δy) the piping can tolerate before it reaches the ASME III code criteria for nonrepeated anchor movements ($3S_c$). A representative cantilevered length of piping shall be used to calculate this limit. The settlement limit must be corrected for any settlement which has already occurred and has induced anchor stress because settlement occurring before the piping was fitted to the pipe anchor does not cause pipe stress at this location.

basis? Why isn't it the distance betw. anchor point & point measured as it enters the building?

Statement is unclear & procedure for making correction is not explained

The technical specifications shall require a report to the NRC when the settlement reaches 75% of the maximum allowable settlement limit. Upon reaching the 75% settlement point, the monitoring frequency shall be monthly, and engineering evaluations will begin.

The monitoring frequency for the monitoring points shall be similar to the monitoring frequency established and implemented for monitoring of structure settlement points throughout the plant. The monitoring points shall be surveyed at 90-day intervals for the first 5 years of operation, and on a yearly basis for the remainder of operating life. The anchor points as well as a point on the piping as it enters the wall penetration shall be monitored. If the differential settlement between the anchor and the designated piping point reaches the 75% reporting limit, we would decrease the official monitoring interval to 30 days, to better assess the settlement rate and severity.

III.A.4 Seismic

a) Seismic analysis

The seismic analysis performed on the buried pipe uses the theory and technique presented in Section 6.0 of BC-TOP-4A[8] to calculate forces and stresses at connections, tees, and bends in buried pipe. These stresses are summarized in Table III-1.

The analysis is based on the equations for beams on elastic foundation. The soil subgrade modulus for each case is calculated based on the soil and pipe properties. The method considers the effect of soil strain on the subgrade modulus and uses the variation of shear modulus with shear strain for sand as developed by Seed and Idriss (Reference 8, Figure 2-5).

The analysis calculates forces and the corresponding stresses due to seismic movements of the surrounding soil and connecting structures. It is a static analysis based on the maximum soil strain which is, in turn, based on the magnitude of the earthquake and the propagation velocities of the various seismic waves.

The flexibility and stress intensification factors for welded elbows or pipe bends and welded tees are determined in accordance with ASME Boiler and Pressure Vessel Code, Section III, Division 1, Subsection NC, Table NC-3673.2(b)-1.

For each case with a bend, elbow, or tee, the analysis considers earthquake motion in two directions (i.e., parallel to each leg). For each case with a connection to a building or component, the analysis considers earthquake motion in three directions.

In the case of a bend, the transverse leg is assumed to deform as a beam on an elastic foundation due to the axial force in the longitudinal leg (parallel to the earthquake motion). The displacement of the bend is defined by the overall spring constant at the bend. The spring constant of the bend depends on the stiffness of the longitudinal and transverse legs as well as the degree of fixity at the bend and at the far ends of the legs. Tees and connections are analyzed in a similar manner.

III.A.4.b) Variable soil properties

The analysis considers the following soil properties:

Poisson's ratio
Unit weight
Coefficient of friction (soil/structure)
Shear modulus
Shear wave velocity
Compression wave velocity
Surface wave velocity
Maximum particle velocity
Maximum particle acceleration
Maximum soil strain

The soil subgrade modulus is calculated for each case, based on the soil and pipe properties. The values used for these soil properties were those determined from the investigation work at the jobsite. The soil modulus of elasticity was varied ±50%. The maximum particle acceleration was increased 50% above the SSE value as a margin for the site-specific response spectra.

III.A.4.c) Effect of pipe deformation on seismic forces

The pipe deformation will affect the seismic forces in two ways. The pipe settlement will cause the idealized straight pipe to bend in the vertical direction, and the bending will cause the pipe cross-section to deform from the idealized circular cross-section to an ovalled cross-section.

To analyze the difference in the seismic loads on the idealized straight pipe and the actual settled pipe with a curved profile, a series of analyses was done on 26-inch diameter pipe configurations which varied from straight to a 5-degree bend. The bend radius was varied from 1 pipe diameter to 100 pipe diameters (2,600 inches).

Figures III-4 and III-5 were used to determine what degree of bend and radius are representative of the existing condition of pipe at the Midland jobsite. Figure III-4 shows the measured profile for line 20"-2HCD-169 and focuses on the segment of greatest bend. Figure III-5 shows the method used to determine the bend radius and degree of bend. This method establishes that the cross-section of this profiled line would be a sag with approximately 4 degrees of bend and a radius of 1,800 inches (90 pipe diameters).

The analyses showed that, for a straight pipe, the axial stress was approximately 5.5 ksi. For a 5-degree bend, the axial stress decreased to 5.3 ksi and 5.4 ksi for 1- and 100-pipe diameter bends respectively.

For a straight pipe the bending stress is zero. For a 5-degree bend the bending stress was 5.5 ksi and 1.3 ksi for 1- and 100-pipe diameter bends respectively.

With the configuration established in Figures III-4 and III-5 (4-degree bend, 90 diameter bend radius) considered representative of existing conditions, both the axial stress (5.4 ksi) and the bending stress (1.1 ksi) were found to vary insignificantly from the straight pipe analysis.

The seismic forces transverse to the axis of the pipe are so small that to distinguish between the forces on the theoretical cross-section and the forces on the ovalled cross-section is beyond the sensitivity of the methods used in the seismic analysis.

An indication of the effect of deformation on the transverse seismic forces can be obtained from determining the change in ovality resulting from seismic strains. Assuming 2.5% ovality to be conservatively representative of pipe at the Midland job-site, the deformation resulting from a 2.5% ovality in a 36-inch diameter pipe is:

$$\% \text{ ovality} = 100 \times (D_{max} - D_{min})/D_o$$

$$2.5 = 100 \times (D_{max} - D_{min})/36$$

$$D_{max} - D_{min} = 0.9 \text{ inches}$$

$$\therefore D_{max} - D_o = D_o - D_{min} = 0.45 \text{ inches}$$

The additional deformation due to seismic strain transverse to the pipe axis is:

Maximum soil strain = 0.000185 in/in - Basis?

Assume soil strain results directly in equal strain in the pipe.

Therefore, the seismic strain induced in the pipe (ϵ_{sse}), is:

$$\epsilon_{SSE} = 36 \times 0.000185 = 0.00666 \text{ in} \cong 0.007 \text{ in.}$$

Assume this strain reduces the minimum diameter and increases the maximum diameter by the same amount.

$$D_{max} - D_{min} = 2(D_{max} - D_o)$$
$$= 2(D_o - D_{min})$$

Adding the seismic strain results in:

$$D_{max} - D_{min} = 2(D_o - D_{min}) + \epsilon_{SSE}$$

$$= 2(0.45 + 0.007)$$

$$= 0.914$$

$$\% \text{ ovality} = 100 \times (D_{max} - D_{min})/D_o$$

$$= 100 \times (0.914)/36$$

$$= 2.539\%$$

Thus, the effect of the seismic loads on an ovalled pipe would be to increase the ovality from 2.5% to 2.539%, which is still within allowable limits.

The preceding discussions indicate that the seismic analysis of the deformed piping, considering the deformation of the piping, would result in axial and bending stresses virtually unchanged from those for a straight pipe, and an increase in ovality from 2.5 to 2.539%.

III.A.4.d) Code requirements

There has been discussion with the staff on the treatment of seismic stresses and settlement stresses. The staff's concern is that if our settlement stress calculations do not meet the $3S_c$ limit as specified for single-anchor point movements in ASME Code Section III, these stresses must be combined with the primary stresses in Equation 10 of Paragraph NC 3652.2. The stress effect of any single nonrepeated anchor movement is compared to a separate allowable ($3S_c$) in Equation 12 of Paragraph NC 3652.3.

Our position, based on settlement stress calculations, is that most of the piping is not overstressed above the code allowable ($3S_c$); and in the local areas where analysis indicates an apparent overstress, it is mainly due to the analytical difficulties in treating the profile data.

These difficulties were discussed in Section II.C. Furthermore, if we do combine settlement stress with seismic stress, it would not be clear from an ASME code viewpoint which code allowable to use for comparison with the calculated stress.

III.A.5 Rebedding

a) Size verification of 8-inch lines

On October 28, 1981, diameter verification pigging operations were conducted on four 8-inch diameter piplines. The specific lines were 8"-1HBC-310, 8"-1HBC-311, 8"-2HBC-81, and 8"-2HBC-82.

The results indicated that each pipeline was greater than 7.781 inches in diameter and was not obstructed. This indicates that none of the pipes has been flattened due to bending or heavy loads and they currently meet the 8% acceptance criteria for ovality.

The pigging operation was conducted in accordance with Appendix C and provided a go, no-go test to check ovality. The results are described in Appendix C.

b) Rebedding of 8- and 10-inch service water lines

Lines 8"-1HBC-81, 8"-1HBC-82, and 10"-0HBC-28 were previously rebedded. Service water lines 8"-2HBC-311, 8"-2HBC-310, and 10"-0HBC-27, near the east side of the diesel generator building, which have not previously been rebedded, will be rebedded to conform to a straight unstressed condition. These lines are identified in the detail section of Drawing SK-C-745 (shown as Figure I-1).

III.A.6 Verification

As discussed in Section II.B and briefly mentioned in Section III.A.2.b, it is necessary to demonstrate that the pipe has sufficient dimensional stability to maintain its functional capability. This will be accomplished by a program of preservice and inservice checks, tests, and inspections.

a) Preservice

Current dimensional stability has been established by inspecting the pipe to determine cross-sectional shape (ovality). Section III.A.1 discusses the equipment and technique for determining ovality, and provides the results of these surveys.

A construction hydrostatic test (ASME III. NC-6221, NC-6129) will be done as follows:

- Test pressure of 1.25 x system design pressure
- Hold interval of 1 hour to test inaccessible weld joints
- Monitoring test pump leakage to establish future leakage criteria

III.A.6.b) Inservice

Inservice inspection will be performed in accordance with ASME Section XI as committed in Midland Preservice and First Ten Year Interval Inspection Plan for NDE and System Pressure Tests - Volume II. The ISI program consists of inservice tests and hydrostatic tests to ensure pressure boundary integrity. The inservice tests are described in Figure III-6 and the hydrostatic tests are described in Figure III-7. The leakage acceptance criteria for these tests are shown in Figure III-8. The ISI will be done with one unit at power during the test. The remaining SWS train will supply cooling water to both units by crossover piping in the auxiliary building and turbine building. Rapid restoration of the tested SWS train is possible because normal isolation valves will be used during these tests.

The flow verification tests to be conducted during plant operations are outlined in Figure III-9, and Tables III-2 and III-3 show minimum required flows and the number and location of flow measurement elements. The requirements are proposed for inclusion in the technical specifications. The monitoring program performs a trending evaluation of the test data to detect any decrease in flow, although acceptance criteria are met.

III. RESOLUTION

B. DIESEL FUEL OIL LINES

1. Profile

The diesel fuel oil lines were installed in June 1980 after the diesel generator building surcharge program was completed. The as-built elevations of those lines were surveyed approximately every 20 feet. These elevations are shown in Drawing MPY-138Q (Figure III-10). This drawing also shows piping support details. It is mounted on Unistrut sections embedded in concrete at intervals along the pipe length. The piping was then covered by approximately 2 feet of compacted soil.

III.B.2 Future Settlement

50.54(f) Questions[2] 17 and 20 discuss the stresses induced in buried pipe due to settlement. Both responses (Table 17-1, Note 6 and Page 20-2, third paragraph) indicate that the fuel oil lines are of such small diameter (1-1/2" and 2") that they have enough flexibility to withstand the predicted settlement without exceeding allowable stresses or affecting their structural integrity.

To substantiate this judgment, an analysis was done to evaluate stresses in the following diesel fuel oil lines due to predicted future settlement:

1-1/2"-1HBC-3, 1-1/2"-2HBC-3,
1-1/2"-1HBC-4, 1-1/2"-2HBC-4,
2"-1HBC-497, 2"-2HBC-497,
2"-1HBC-498, 2"-2HBC-498

This analysis assumed:

1. Three inches of settlement was proportioned over a 40-foot pipe span with the 3 inches occurring at midspan.

2. Simplified beam equations were used for buried piping continuously supported by soil.

The analysis indicated the highest stress value, including stress intensification factors, was 18 ksi in a 2-inch diameter line. This is well within the allowable stress of 45 ksi ($3S_c$) for these lines and substantiates the claim of flexibility made in the responses to 50.54(f) Questions 17 and 20.

III. RESOLUTION

C. BORATED WATER STORAGE TANK LINES

1. Rebedding

In the October 6 and 7, 1981, meeting with the NRC staff, Consumers Power Company committed to rebed the 18-inch BWST line from the tank valve pit to the tank farm dike. These pipelines are identified as 18"-1HBC-1, 18"-1HBC-2, 18"-2HBC-1, and 18"-2HBC-2. This commitment was made because this piping is in the area to be surcharged as part of the remedial fixes on the tank foundations. The measured profile data taken in 1979 on pipelines 18"-2HBC-1 and 18"-1HBC-2 show maximum deflections of 1.92 inches and 0.96 inch, respectively. These measurements are within the construction tolerance of ±2 inches for installation of piping and it may be assumed that soils settlement has not adversely affected this piping.

2. Future Settlement

Borated water storage tank lines have been cut loose at the valve pit to isolate them from the settlement caused by the surcharge of the valve pit.

The existing program which monitors the settlement of the BWST and the auxiliary building will provide data on the future settlement of these lines. These monitoring points will indicate whether the piping is overstressed due to settlement.

III. RESOLUTION

D. MISCELLANEOUS GENERIC SUBJECTS

There are several subjects pertinent to most of the buried pipe. Rather than discuss each subject several times as it relates to each piping system, this section will discuss each subject, including how it affects each piping system. The subjects considered generic to all buried pipe and which are discussed in this section are:

- Anchor and component loads
- Effects of rupture of nonsafety-related piping on safety-related piping, components, and structures (referred to herein as "II under I")
- Overburden loads

1. Anchor and Component Loads

The loads induced into anchors and components by settlement of the underground piping are being analyzed to determine acceptable settlement used limits. These limits will be in conjunction with the monitoring program discussed in Section III.A.3.6.

The settlement limit shall be established from the amount of settlement (Δy) the piping can tolerate before it reaches the ASME Code Section III criteria for nonrepeated anchor movements ($3S_c$). The limit will be the lesser of the settlement which causes the limiting stress or the settlement which causes contact with the penetration through the building. The settlement limit will be corrected for any settlement which has already occurred and has induced pipe stress at the anchor point.

III.D.2 II Under I

In the draft SER[1] the NRC expressed a concern for the effects of the rupture of nonsafety-related piping on safety-related piping, components, and structures. This concern is referred to herein as "II under I."

This concern is a classic II/I question, brought up during a discussion of underground piping settlement at Midland, but not peculiar to the Midland soils issue and not unique to Midland.

Pipe break encompasses not only whip and jet impingement, but also the related hazards of steam and liquid flooding, excess pressure, differential pressure, and temperature.

Of the foregoing effects of a pipe break, liquid flooding is the single item requiring evaluation for buried piping. Analysis of flooding is treated on a case-by-case and individual system basis. The possible result of flooding would be a washout/loss of support.

A review was done to identify where non-Seismic Category I pipe passes beneath a Seismic Category I pipe or structure. A break in the non-Seismic Category I pipe was assumed to cause a washout extending to the surface, thus causing a loss of support for any Seismic Category I system above it. The unsupported length was determined using a side slope of 45 degrees, the vertical separation, and the angle of crossing of the two systems.

The review indicated that for all non-Seismic Category I pipes passing beneath a Seismic Category I pipe, the maximum stress induced in the overlying Seismic Category I pipe was approximately 3 ksi for line 1-1/2"-2HBC-498.

The effect of a non-Seismic Category I pipe break on structures is considered to be encompassed by the break discussed in the Response to 50.54(f) Question 49[2], Part c2. The pertinent portion of this response is included as Appendix D.

II.D.3 Overburden Loads

This section discusses the effects of overburden loads such as soil dead weight, heavy equipment, etc on the buried piping. The Response to 50.54(f) Question[2] 34 addressed this question. The Response to Question 34 is attached as Appendix E.

The Response to Question 34 refers to the effect, at a depth of 6 feet, of a Cooper E-80 railroad load. A review of the depth of cover (distance below ground surface) of all Seismic Category I lines indicated that 6 feet is the approximate depth of cover on all lines except the diesel fuel oil lines. The results, indicated in Appendix E, concluded that the 26-inch and 36-inch buried Seismic Category I pipes are adequate to withstand external loads, and stresses in pipes smaller than 26-inch diameter will be relatively low and are not critical.

The diesel fuel oil lines have a minimum cover of approximately 2.2 feet. AWWA M11[4] includes a graph (see Figure III-11) showing the relationship between load (expressed as height of cover) and the diameter of steel pipe. This graph shows that for diameters less than 20 inches, the amount of load needed to cause a 1% deflection increases almost infinitely. According to this graph, a 1-1/2 inch to 2 inch diameter pipe would be virtually uncrushable when buried in the fill.

IV. LIST OF REFERENCES

		Referenced
1.	Safety Evaluation Report (draft), Sections 3.6.2, 3.7.3, and 3.9, transmitted by R.L. Tedesco's September 23, 1981, letter	1, 32
2.	NRC Requests Regarding Plant Fill (referred to as 50.54(f) Questions)	2, 15, 29, 32, 33
3.	Standard Review Plan SRP 3.9.3, ASME Code Class 1, 2, and 3 Components, Component Supports, and Core Support Structures, Rev. 1, July 1981	3
4.	AWWA Manual M11, Steel Pipe Design and Installation, American Water Works Association, 1964	4, 11, 33
5.	NUREG/CR-0261, Evaluation of the Plastic Characteristics of Piping Products in Relation to ASME Code Criteria, July 1978	4, 10
6.	Specification 7220-M-214, Piping System Erection Fitup Control Requirements	5
7.	Evaluation of Pipe Behavior Due to Soil Settlement for a Typical Buried Line for the Midland Nuclear Power Plant, Structural Mechanics Associates, May 1981	6
8.	BC-TOP-4A, Seismic Analyses of Structures and Equipment for Nuclear Power Plants, Rev. 4	18
9.	ICS Civil Engineer's Handbook, page 136; edited by Archibald DeGroot, International Textbook Company, 1956	Fig. III-5

V. LIST OF TABLES

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I-1	Seismic Category I Lines to be Addressed	1
III-1	Stress Summary for Buried SW Piping	18
III-2	Minimum Required Flows	27
III-3	Flow Measurement	27

P= Settlement Profile Provided by CPC Either in past or as of 1/28/82

SEISMIC CATEGORY I LINES TO BE ADDRESSED

A. Service Water System (SWS)

26-2JBD-1 } Profiled Aug 81 (App B-Rpt.1)
26-2JBD-2 } for elevation - not ovality

8"-1HBC-310 - Pigging Operation Cat'e.
P 8"-2HBC-81 - Pigging Operation Cat'e.
8"-1HBC-81 - Rebedding completed
8"-2HBC-310 To be rebedded
P 8"-1HBC-311 - Pigging Operation Cat'e.
P 8"-2HBC-82 - Pigging Operation Cat'e.
8"-1HBC-82 - Rebedding completed
8"-2HBC-311 To be rebedded
P 10"-OHBC-27 - To be rebedded
10"-OHBC-28 - Rebedding completed

P 26"-OHBC-53
P 26"-OHBC-54
for elevation P 26"-OHBC-55 - Profiled Aug 81 } App B-Rpt. 1
& ovality P 26"-OHBC-56 - Profiled Aug 81 }
P 26"-OHBC-15
P 26"-OHBC-16
P 26"-OHBC-19 } Profiled Sept 81 for elevation
P 26"-OHBC-20 } & ovality (App B, Rpt. No 2
P 36"-OHBC-15
P 36"-OHBC-16
P 36"-OHBC-19 } Profiled Sept. 81 for elevation
P 36"-OHBC-20 } & ovality (App. B, Rpt. No 2

B. Diesel Fuel Oil Lines (Fuel Oil)

Analyzed & shown not to be a problem

1-1/2"-1HBC-3 ✓
1-1/2"-1HBC-4 ✓
1-1/2"-2HBC-3 ✓
1-1/2"-2HBC-4 ✓
2"-1HBC-497 ✓
2"-1HBC-498 ✓
2"-2HBC-497 ✓
2"-2HBC-498 ✓

C. Borated Water Storage Tank (BWST)

18"-1HBC-1 }
18"-1HBC-2 } Are to be rebedded (Sep 82)
18"-2HBC-1 }
18"-2HBC-2 }

Profiles Presented

P 20"-1HCD-169
P 26"-2JBD-1
P 26 - 1JBD-2

TABLE I-1 12/10/81

* Analyses are being completed

ASME CODE CHECK - STRESS SUMMARY FOR BURIED S.W. PIPING.

STRESSES IN PSI.

LINE #	DESCRIPTION	NORMAL EQ. 8		UPSET EQ. 9		FAULTED CODE CASE 1606		THERMAL EQ. 10		EQ. 11	
		ACTUAL STRESS	ALLOWABLE STRESS	ACTUAL STRESS	ALLOWABLE STRESS	ACTUAL STRESS	ALLOWABLE STRESS	ACTUAL STRESS	ALLOWABLE STRESS	ACTUAL STRESS	ALLOWABLE STRESS
36/26-0HBC-15	S.W. SUPPLY	2442	17,500	6997	21,000	12,996	42,000	5214	26,250	—	—
36/26-0HBC-16	S.W. RETURN	2442	17,500	8380	21,000	30,277	42,000	10,420	26,250	—	—
36/26-0HBC-19	S.W. SUPPLY	2442	17,500	9965	21,000	32,212	42,000	10,814	26,250	—	—
36/26-0HBC-20	S.W. RETURN	2442	17,500	9965	21,000	32,612	42,000	21,613	26,250	—	—
26-0HBC-53	S.W. SUPPLY	1742	17,500	6121	21,000	18,187	42,000	12,513	26,250	—	—
26"-0HBC-54	S.W. RETURN	1742	17,500	5862	21,000	24,842	42,000	25,009	26,250	—	—
26"-0HBC-55	S.W. SUPPLY	1742	17,500	9265	21,000	31,512	42,000	13,857	26,250	—	—
26"-0HBC-56	S.W. RETURN	1742	17,500	6150	21,000	17,177	42,000	—	—	29,437	43,750
10"-0HBC-27 8"-1HBC-81 8"-2HBC-310	S.W. SUPPLY	695	15,000	7019	18,000	18,713	36,000	← NOT ANALYZED* →			
8"-1HBC-310	S.W. SUPPLY	625	15,000	2434	18,000	5159	36,000	← NOT ANALYZED* →			
8"-1HBC-311	S.W. RETURN	625	15,000	1274	18,000	3975	36,000	← NOT ANALYZED* →			
8"-2HBC-81	S.W. SUPPLY	625	15,000	2434	18,000	5159	36,000	← NOT ANALYZED* →			
8"-2HBC-82	S.W. RETURN	625	15,000	1511	18,000	6228	36,000	← NOT ANALYZED* →			
10"-0HBC-28 8"-1HBC-82 8"-2HBC-311	S.W. RETURN	695	15,000	4416	18,000	11,050	36,000	← NOT ANALYZED* →			

AAO-820 8/76

TABLE III-1

TABLE III-2

MINIMUM REQUIRED FLOWS

Line	Description	Required Flow (gpm)
8"-1HBC-310	DG 1A Supply	1,600
8"-2HBC-81	DG 2A Supply	1,600
8"-1HBC-81	DG 1B Supply	1,600
8"-2HBC-310	DG 2B Supply	1,600
8"-1HBC-311	DG 1A Return	1,600
8"-2HBC-82	DG 2A Return	1,600
8"-1HBC-82	DG 1B Return	1,600
8"-2HBC-311	DG 2B Return	1,600
10"-0HBC-27	DG 1B/2B Supply	3,200
10"-0HBC-28	DG 1B/2B Return	3,200
26"-0HBC-53	DG 1A/2A + TB Supply	9,225
26"-0HBC-54	DG 1A/2A + TB Return	9,225
26"-0HBC-55	DG 1B/2B + TB Supply	9,225
26"-0HBC-56	DG 1B/2B + TB Return	9,225
26"-0HBC-15	Aux Bldg A Supply	15,894
26"-0HBC-16	Aux Bldg A Return	15,894
26"-0HBC-19	Aux Bldg B Supply	15,894
26"-0HBC-20	Aux Bldg B Return	15,894
36"-0HBC-15	A Supply	25,119
36"-0HBC-16	A Return	25,119
36"-0HBC-19	B Supply	25,119
36"-0HBC-20	B Return	25,119

Required flows are based on FSAR tables 9.2-1 and 9.2-2. Worst-case values for each line were determined from the six operation modes and the ESF mode in those tables. Turbine building flows are based on potential flow under accident conditions (Mode 6).

TABLE III-3

FLOW MEASUREMENT

Line	Description	Flow Element	Location
8"-1HBC-310	DG 1A Supply	1 FE 1841	Cooler Outlet
8"-2HBC-81	DG 2A Supply	2FE 1851	Cooler Outlet
8"-1HBC-81	DG 1B Supply	1FE 1846	Cooler Outlet
8"-2HBC-310	DG 2B Supply	2FE 1855	Cooler Outlet
8"-1HBC-311	DG 1A Return	1FE 1841	Cooler Outlet
8"-2HBC-82	DG 2A Return	2FE 1851	Cooler Outlet
8"-1HBC-82	DG 1B Return	1FE 1846	Cooler Outlet
8"-2HBC-311	DG 2B Return	2FE 1855	Cooler Outlet
10"-0HBC-27	DG 1B/2B Supply	1FE 1846 + 2FE 1855	Cooler Outlet Cooler Outlet
10"-0HBC-28	DG 1B/2B Return	1FE 1846 + 2FE 1855	Cooler Outlet Cooler Outlet
26"-0HBC-53	DG 1A/2A + TB1 Supply	1FE 1876	Supply Line - Metering Pit
26"-0HBC-54	DG 1A/2A + TB1 Return	1FE 1876	Supply Line - Metering Pit
26"-0HBC-55	DG 1B/2B + TB2 Supply	2FE 1876	Supply Line - Metering Pit
26"-0HBC-56	DG 1B/2B + TB2 Return	2FE 1876	Supply Line - Metering Pit
26"-0HBC-15	Aux Bldg A Supply	0FE 1995A + 1FE 1914A + 1FE 1990A + 2FE 1990A	Aux Bldg A - Supply Line Booster Pump Discharge Chiller Outlet Chiller Outlet
26"-0HBC-16	Aux Bldg A Return	0FE 1995A + 1FE 1914A + 1FE 1990A + 2FE 1990A	Aux Bldg A - Supply Line Booster Pump Discharge Chiller Outlet Chiller Outlet
26"-0HBC-19	Aux Bldg B Supply	0FE 1995B	Aux Bldg B - Return Line
26"-0HBC-20	Aux Bldg B Return	0FE 1995B	Aux Bldg B - Return Line
36"-0HBC-15	A Supply	1FE 1876 + 0FE 1995A + 1FE 1914A + 2FE 1990A 2FE 1990A	Supply Line - Metering Pit Aux Bldg A - Supply Line Booster Pump Discharge Chiller Outlet Chiller Outlet
36"-0HBC-16	A Return	1FE 1876 + 0FE 1995A + 1FE 1914A + 1FE 1990A + 2FE 1990A	Supply Line - Metering Pit Aux Bldg A - Supply Line Booster Pump Discharge Chiller Outlet Chiller Outlet
36"-0HBC-19	B Supply	2FE 1876 + 0FE 1995B	Supply Line - Metering Pit Aux Bldg B - Return Line
36"-0HBC-20	B Return	2FE 1876 + 0FE 1995B	Supply Line - Metering Pit Aux Bldg B - Return Line

(This list confirms capability to measure flows in buried service water system piping using installed instrumentation. In some areas, additional measurement devices are installed that may be considered preferable alternatives.)

VI. LIST OF FIGURES

		Referenced
I-1	Plan of Buried Q-Listed Pipe Locations, Bechtel Drawing SK-C-745	1, 25
II-1	Initial Discontinuities in Installed Pipe, Figure 6 from Southwest Research Institute Report, "Structural Analysis of Buried Pipeline," October 16, 1981	5
II-2	Linear Elastic Analysis Results for Upper Bound Soil Properties, Figure 4-2 from Structural Mechanics Associates report, Evaluation of Pipe Behavior Due to Soil Settlement for a Typical Buried Line for the Midland Nuclear Power Plant, SMA 13701.02, May 1981	6
III-1	Schematic-Pipe Elevation Profile Measurement System, Page 8 of Southwest Research Institute Report No. 1, Pipe Profile Measurements at Midland, August 1981	7
III-2	Sketch-SwRI Out-of-Roundness Measurement, Instrument	8
III-3	Typical Laboratory Time-Settlement Behavior Under Constant Pressure	15
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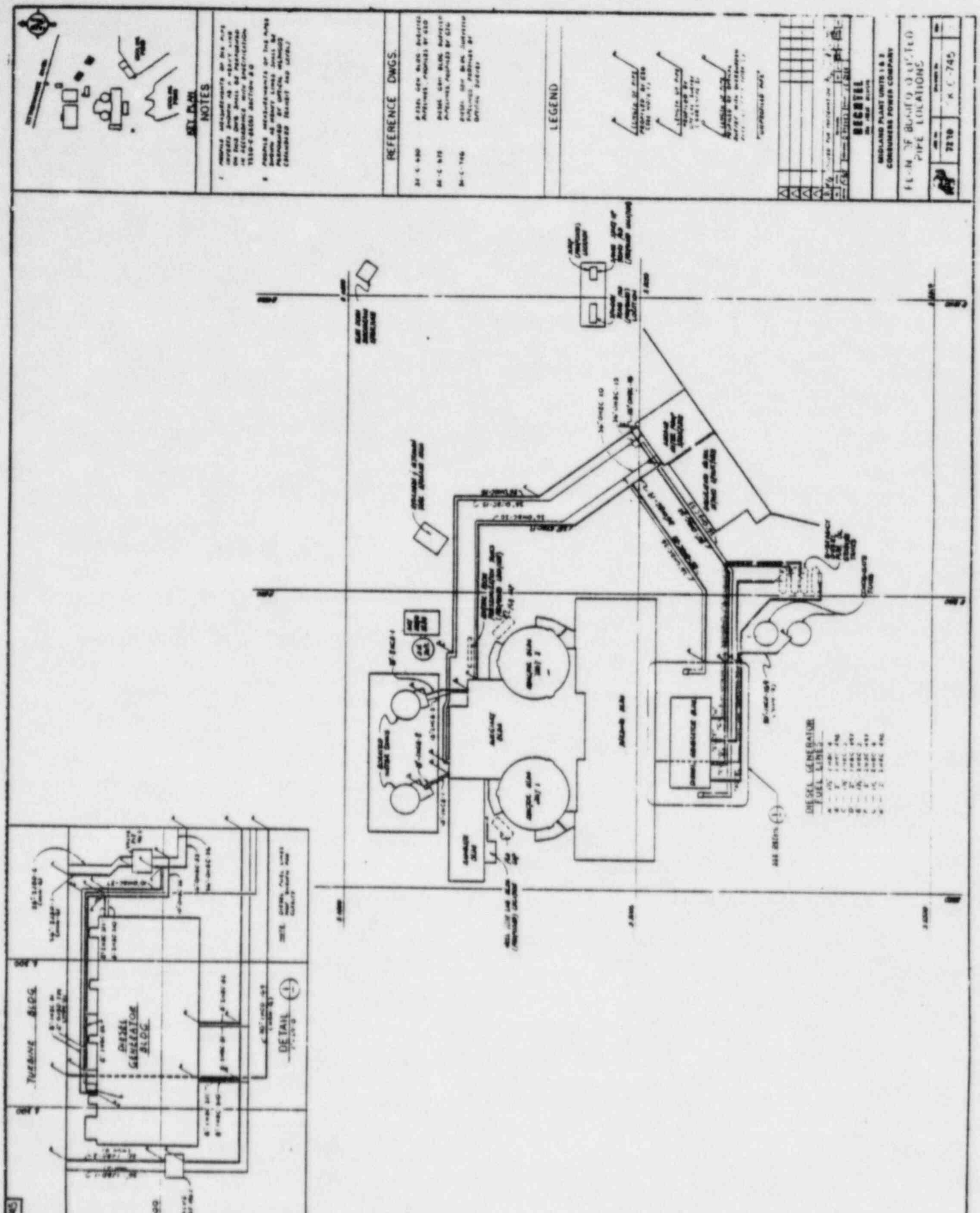
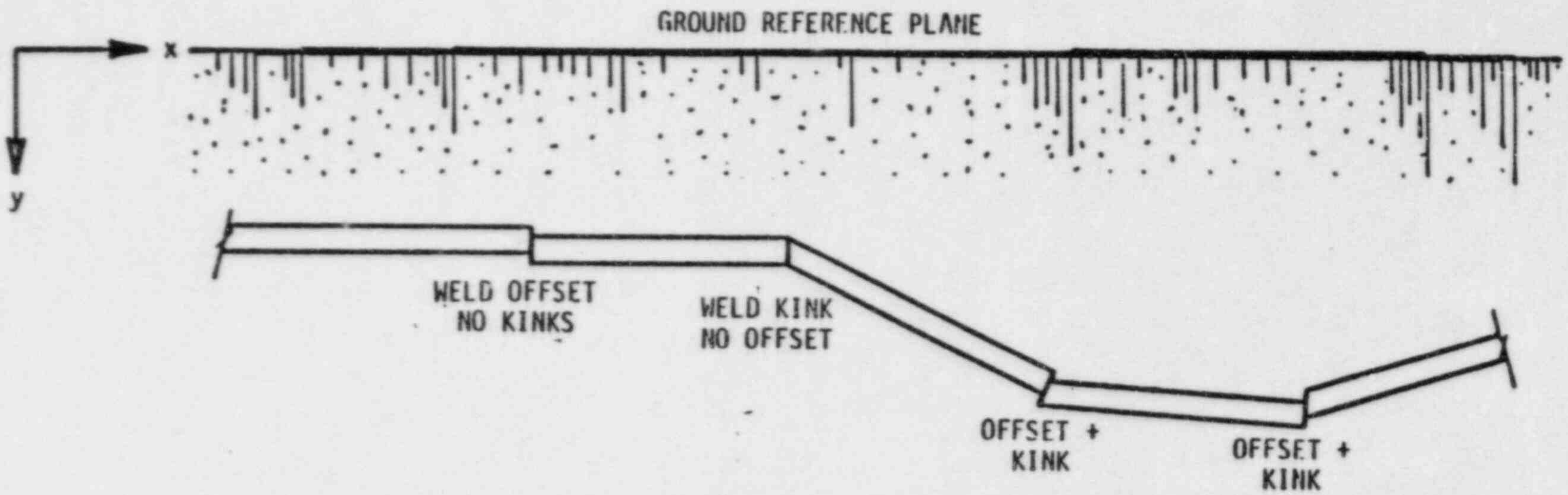

KEY PLAN
NOTES
REFERENCE DWGS.
LEGEND
BECHTEL
MIDLAND PLANT UNITS 1 & 2
CONSUMERS POWER COMPANY
TURBINE BLDG
DIESEL GENERATOR BLDG
DETAIL
DIESEL GENERATOR FUEL LINES

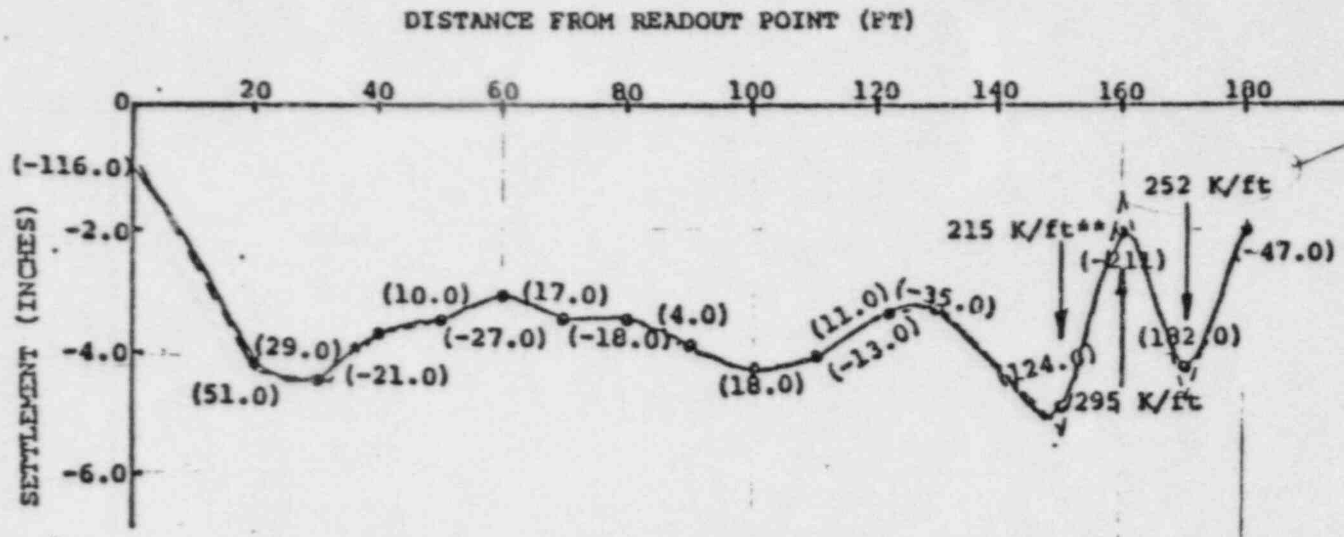
FIGURE I-1

FIGURE II-1



INITIAL DISCONTINUITIES IN INSTALLED PIPE

FIGURE II-2



● Indicates Pipe Position at Survey Points

—— Pipe Displacement Profile

--- Soil Settlement Profile

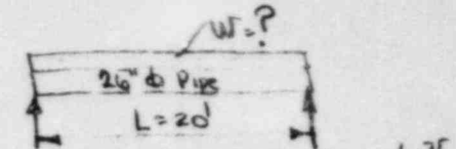
* Pipe bending stress in ksi at measurement point (typical)

** Soil spring forces (typical)

Conservative Soil Capacity Estimates

Uplift = 10 K/Ft
Bearing = 75 K/Ft
Lateral = 28 K/Ft

Not clear how calculated



Δ_{max} = deflection @ center $= \dfrac{5wL^4}{384EI}$ Pg. 6-25 C.E. Handbook

$\Delta_{scaled} = 2.1"$

$\therefore 2.1" = \dfrac{5 \cdot w \cdot (20' \times 12"/ft)^4}{384 \times 30{,}000\frac{K}{in^2} \times \frac{\pi(26^4 - 25^4)\,in^4}{64}}$

where $I = \dfrac{\pi(d^4 - d'^4)}{64} = 3257\,in^4$

Pg. 6-20 C.E. Handbook
values of 26", 25" were assumed

$2.1\,inch = \dfrac{5 \cdot w \cdot 331776000\,in^4}{384 \times 30{,}000\frac{K}{in^2} \times 3257\,in^4}$

$2.1\,inch = .442\,\frac{in^2}{K} \cdot w$

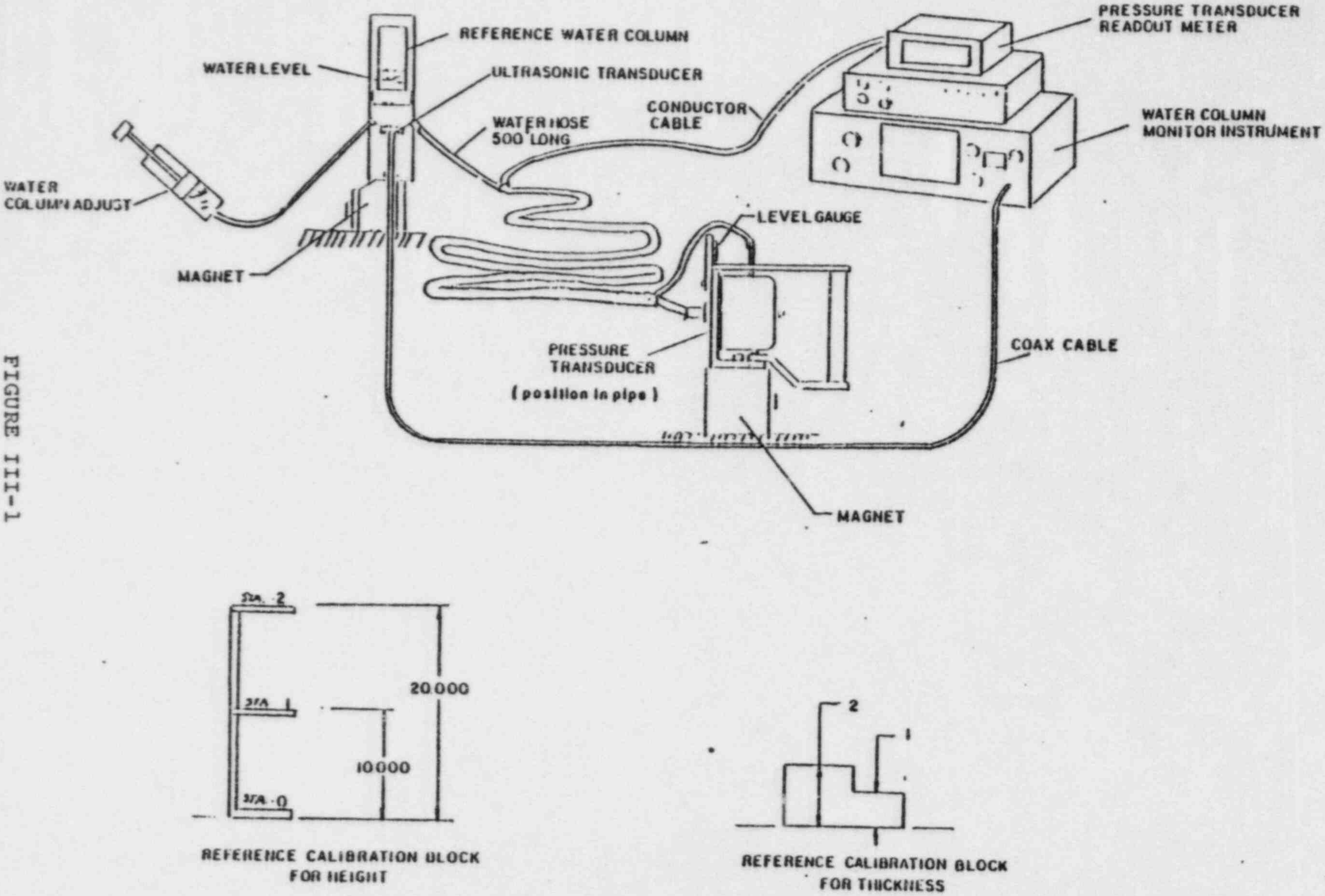
$w = \dfrac{2.1\,inch}{.442\,inch^2/K}$

$w = 4.75\,K/in$

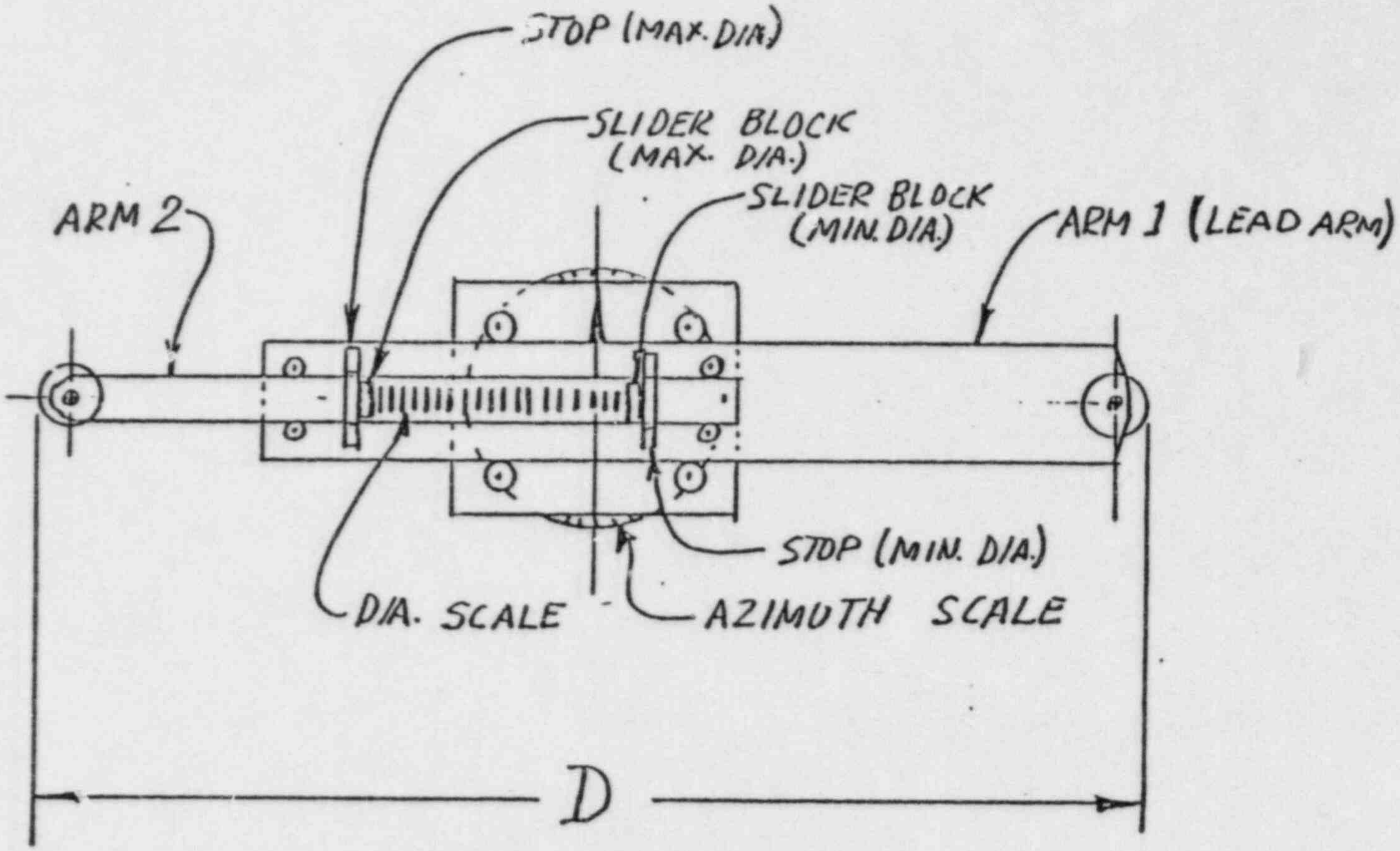
$w = 57\,K/ft$

LINEAR ELASTIC ANALYSIS RESULTS FOR UPPER
BOUND SOIL PROPERTIES

FIGURE III-1

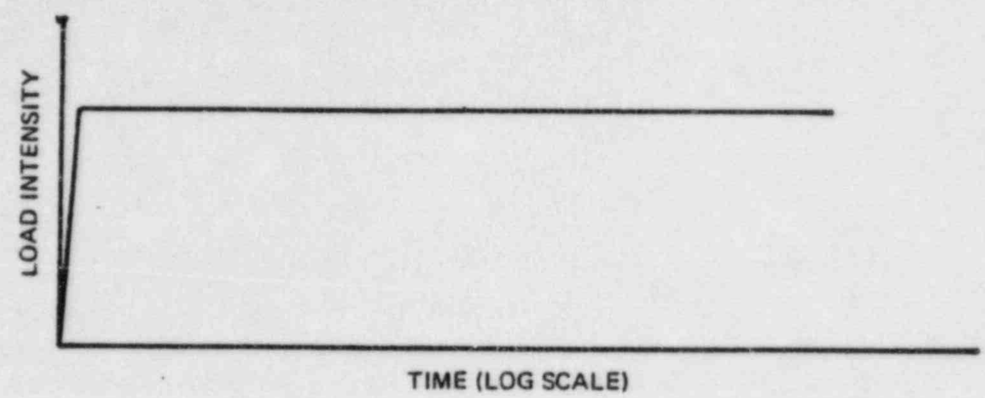


SCHEMATIC- PIPE ELEVATION PROFILE MEASURMENT SYSTEM

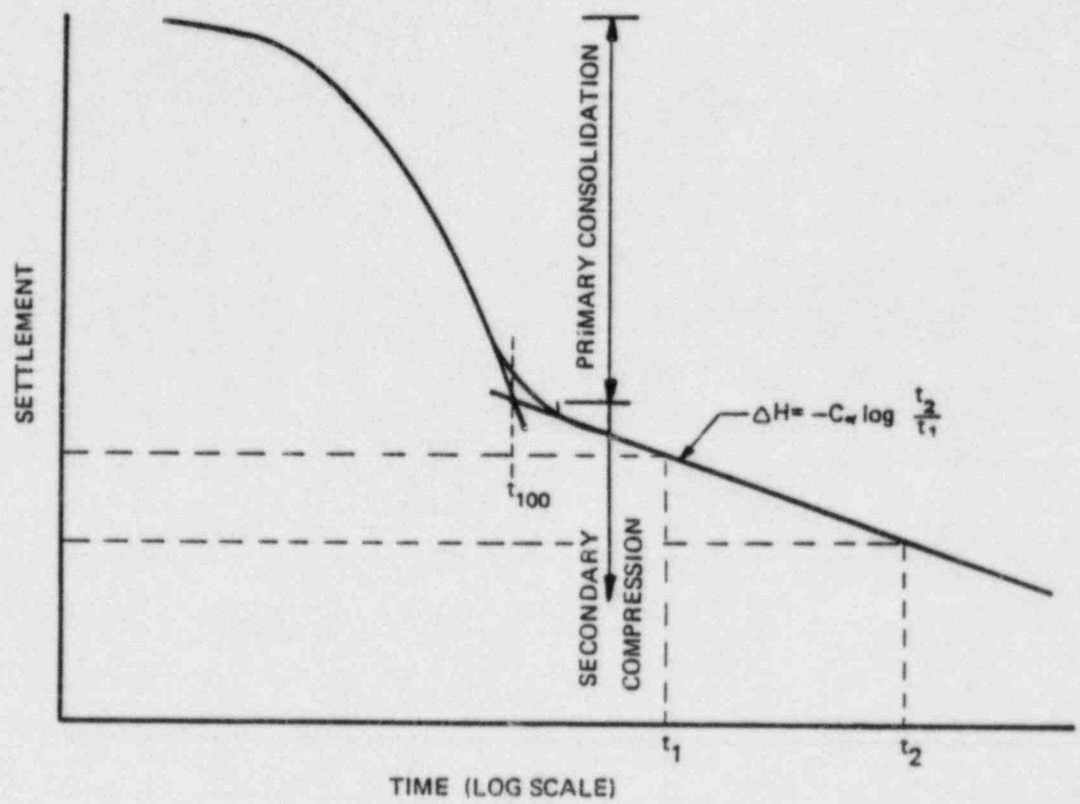


SKETCH- SwRI OUT-OF-ROUNDNESS MEASUREMENT INSTRUMENT

FIGURE III-2



(A) LOAD HISTORY



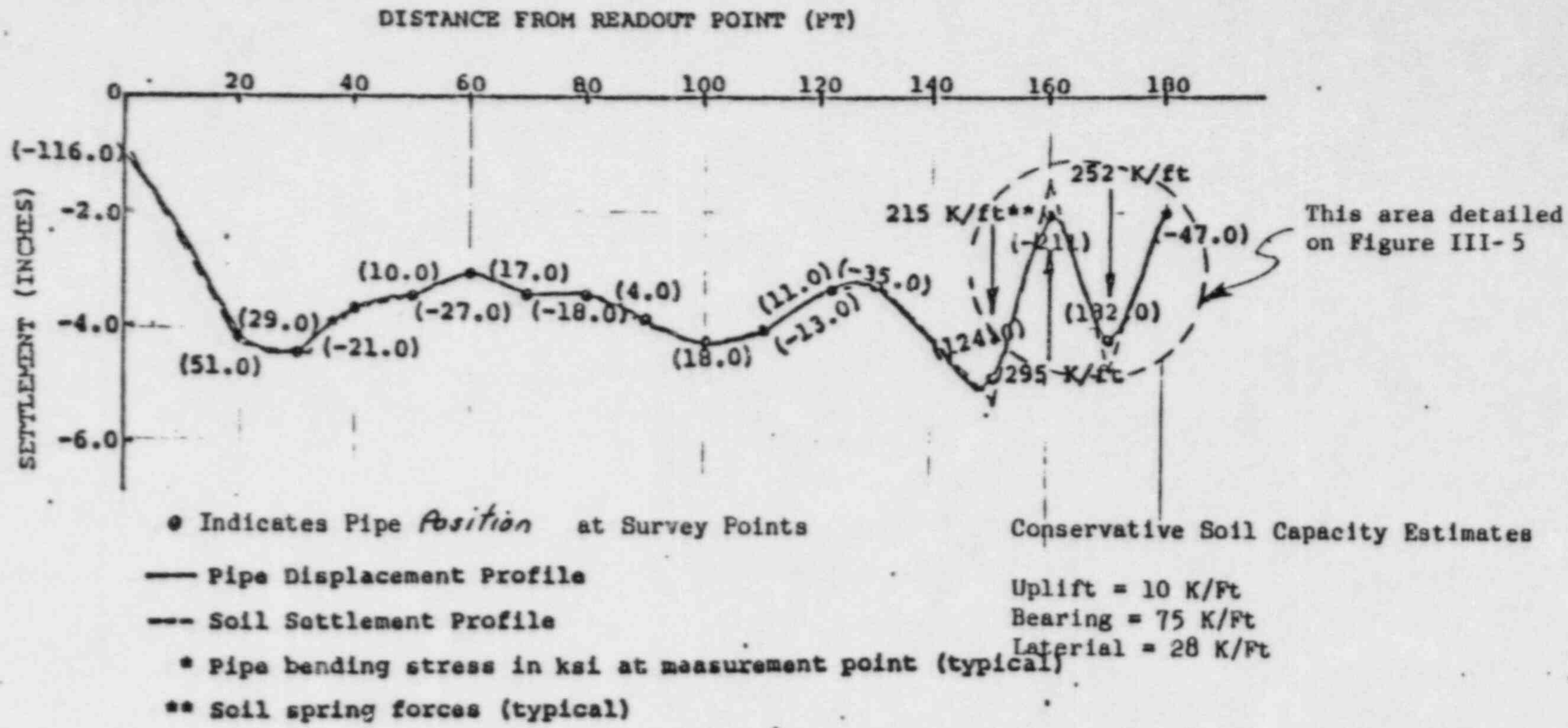
(B) TIME - SETTLEMENT CURVE

FIGURE III-3

CONSUMERS POWER COMPANY MIDLAND PLANT UNITS 1 & 2
Typical Laboratory Time-Settlement Behavior Under Constant Pressure
Figure 27-1

2/80 Revision 5

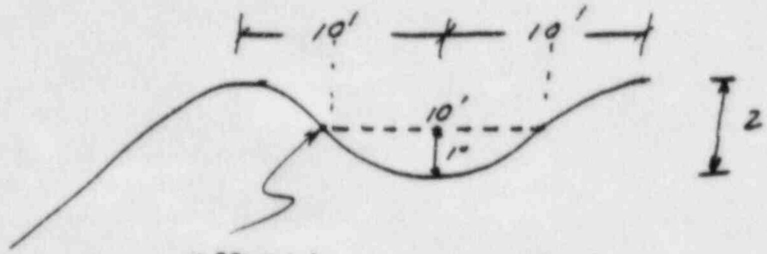
FIGURE III-4



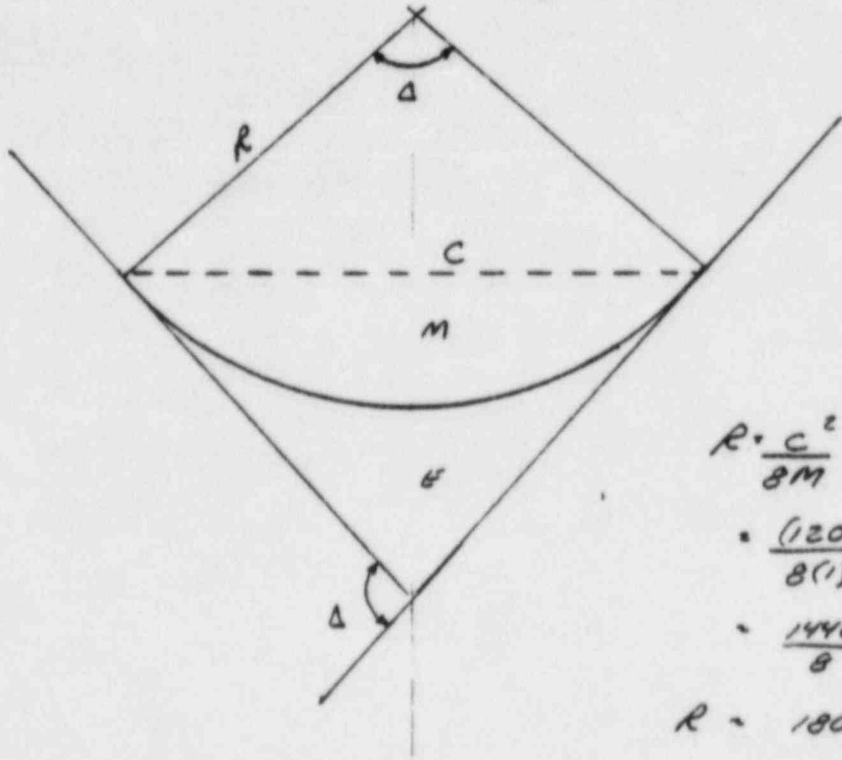
LINEAR ELASTIC ANALYSIS RESULTS FOR UPPER BOUND SOIL PROPERTIES

Determination of Bend Radius
and Degree of Bend

Detail of Area
Indicated on
Figure III-4



assume:
- inflection pt @ midpt.
- simple curve



c = chord = 10 ft
m = middle ordinate = 1 in.

$$R = \frac{c^2}{8M} + \frac{M}{2} \quad (9)$$

$$= \frac{(120)^2}{8(1)} + \frac{1}{2}$$

$$= \frac{14400}{8} + \frac{1}{2}$$

$$R = 1800.5 \text{ in.} = 150.04 \text{ ft.}$$

Degree of curve $= \frac{5730}{R} = 38.2°$

1 radian (57.30°) subtends an arc of 150 ft (1 radius)

an arc of 10 ft (approx. C) would be :

$$\frac{\Delta}{57.3} = \frac{10}{150} \qquad \Delta = \frac{57.3}{15} = 3.82°$$

FIGURE III-5

INSERVICE TESTS - LEAKAGE TESTS

- EACH INSPECTION PERIOD: 3, 7, 10, 13, 17...YEARS
- NOMINAL SYSTEM OPERATING PRESSURE: 57 PSIG
- ISOLATE BURIED PIPING
- PRESSURIZE WITH TEST PUMP
- MAINTAIN PRESSURE 4 HOURS
- MEASURE FLOW

FIGURE III-6

HYDROSTATIC TESTS - LEAKAGE TESTS

- EACH INSPECTION INTERVAL: ONCE EACH 10 YEARS
- 1.10 × DESIGN PRESSURE: 115.5 PSIG
- ISOLATE BURIED PIPING
- PRESSURIZE WITH TEST PUMP
- MAINTAIN PRESSURE 4 HOURS
- MEASURE FLOW

LEAKAGE TEST ACCEPTANCE CRITERIA

- SMALL ENOUGH TO DETECT PRESSURE BOUNDARY FAILURE
- LARGE ENOUGH TO ACCOMMODATE ANTICIPATED BOUNDARY VALVE LEAKAGE
- 0-5 GPM
- RESULTS IN INSIGNIFICANT FLOW LOSS
- TO BE REVIEWED FOLLOWING PRESERVICE TESTS

MIDLAND UNITS 1 AND 2
NRC PRESENTATION 10/2/81

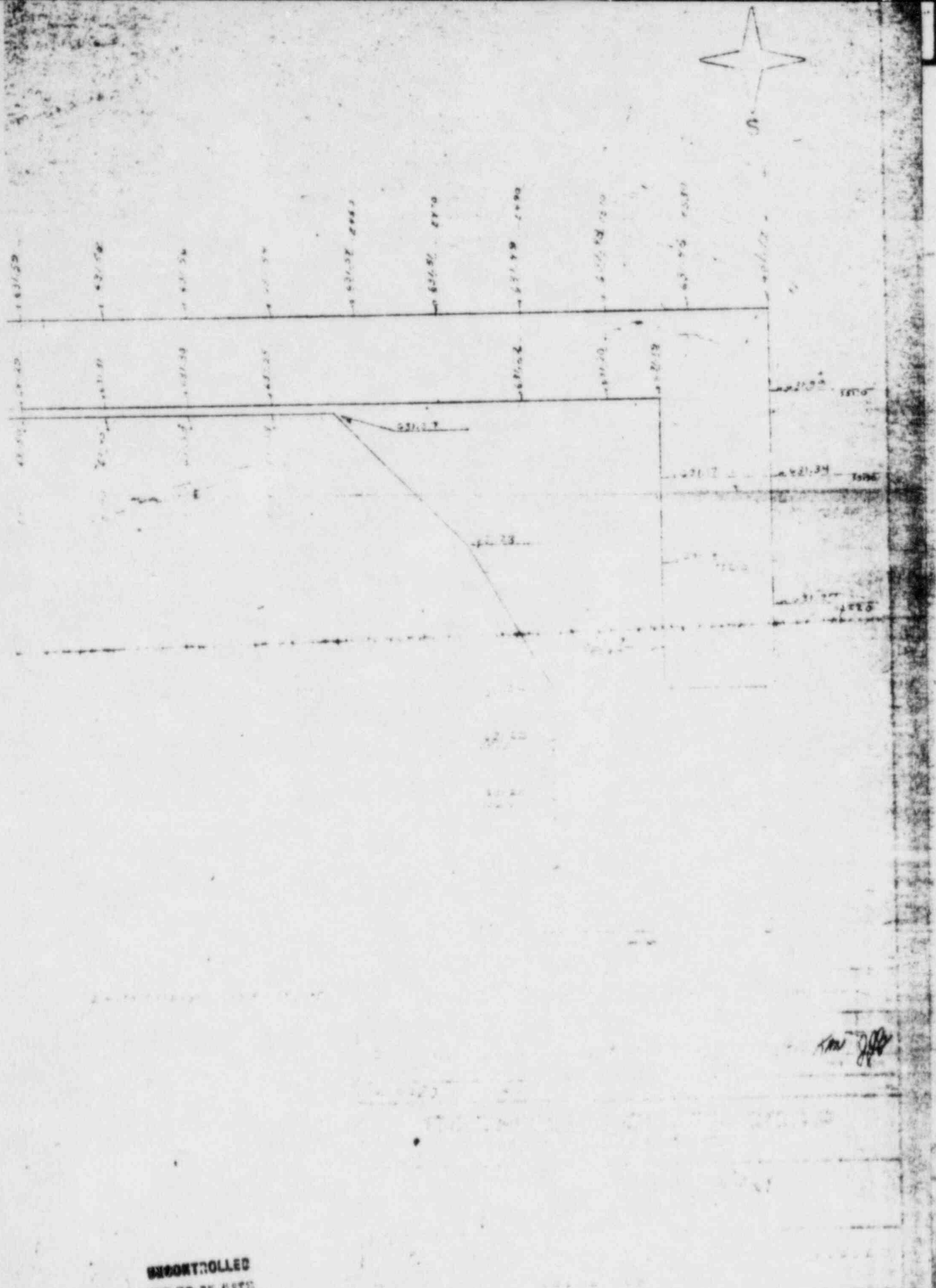
G-1868-08

FIGURE III-8

FLOW VERIFICATION

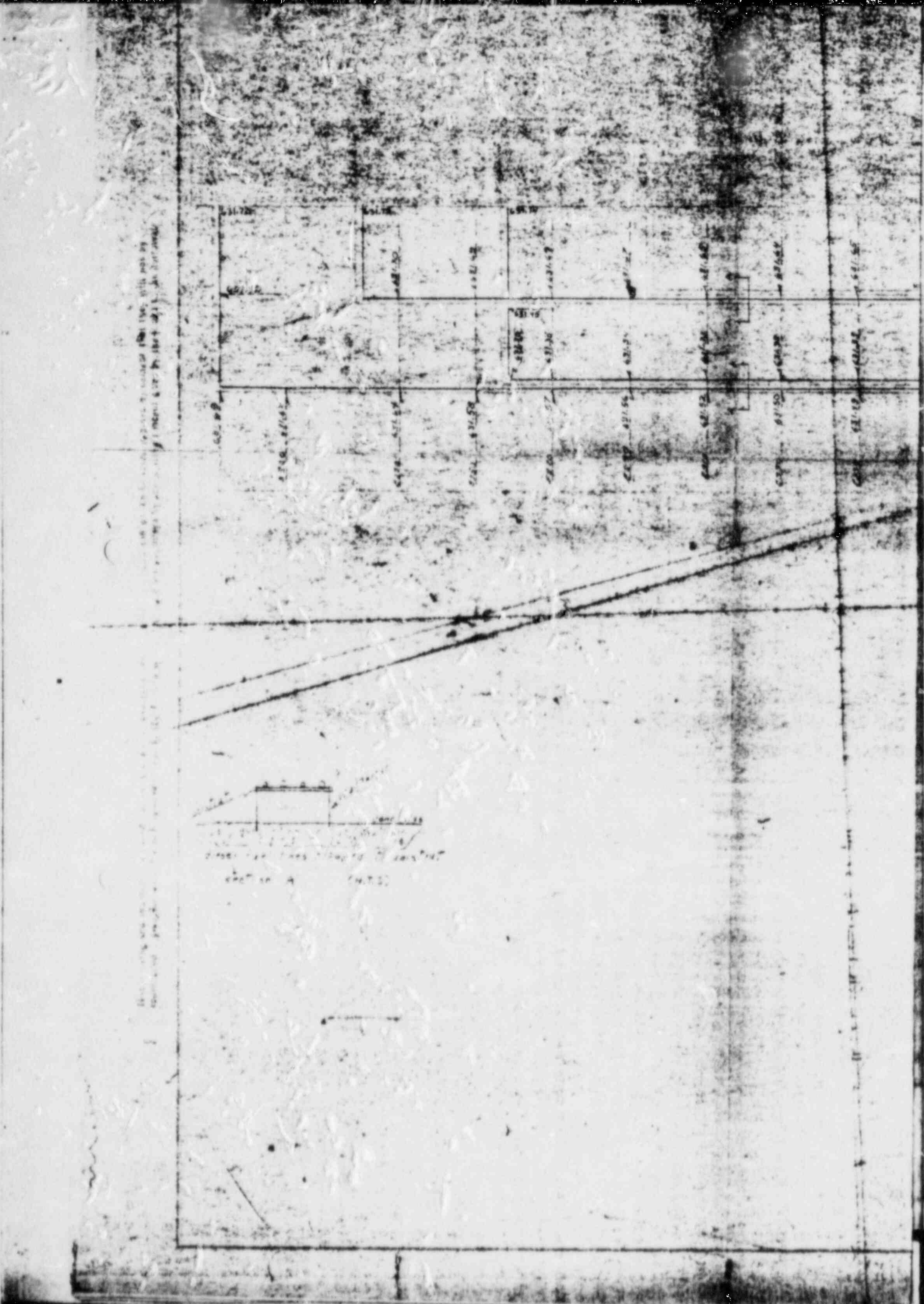
- ENSURE ABILITY OF BURIED PIPING TO MAINTAIN FLOWS REQUIRED FOR SAFETY FUNCTIONS
- ESTABLISH PUMP AND SYSTEM LINEUPS TO OBTAIN KNOWN CONFIGURATION THAT PROVIDE REQUIRED FLOWS
- UTILIZE INSTALLED INSTRUMENTATION TO VERIFY REQUIRED FLOW IN EACH BURIED LINE
- ONCE PER YEAR
- TO BE INCLUDED IN TECHNICAL SPECIFICATIONS

FIGURE III-9



UNCONTROLLED
NOT TO BE USED
FOR CONSTRUCTION

Figure III-10



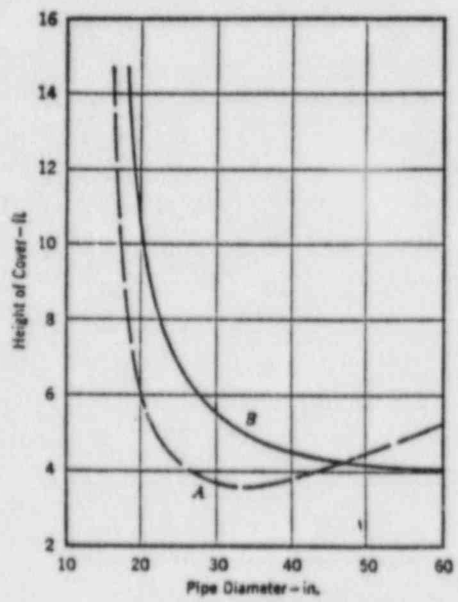


Fig. 8.5. Relationship Between Calculated Height of Cover and Diameter of ¼-Inch Steel Pipe

The relationship was computed from Eq 8.3 and Eq 8.4a for a deflection of 1 per cent of the pipe diameter; lag factor, 1.5; K, 0.10; and soil weight, 110 lb/cu ft. For Curve A, e = 30 psi/in.; for Curve B, er = 700 psi.

FIGURE III-11

VII. APPENDIXES

		Referenced
A.	Teledyne Engineering Services letter, D.F. Landers to W.J. Cloutier of Consumers Power Company, November 11, 1981	3, 10
B.	Southwest Research Institute, four reports on pipe profile measurements at Midland	7, 8
C.	Procedures and Drawings for Diameter Verification Pigging Procedure	25
D.	Partial Copy of Response to Question 49 of NRC Requests Regarding Plant Fill	32
E.	Response to Question 34 of NRC Requests Regarding Plant Fill	33

J. Kane
Rec'd 12/18/81

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 Cloutier 10/27/81
Written By Date

Donald E. Sibbald 11/3/81
Technical Review Date

KC Akey 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 exits 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 igging 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.

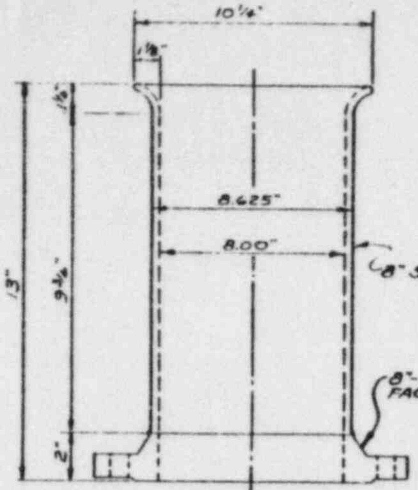
Respectfully submitted

NORTHWOOD'S CONSTRUCTOR'S, INC.

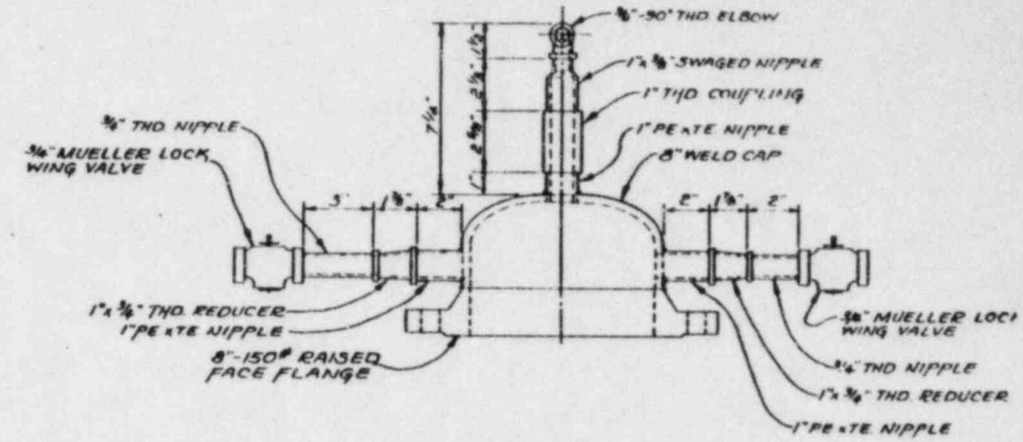
J W Fluharty

J. W. Fluharty, President

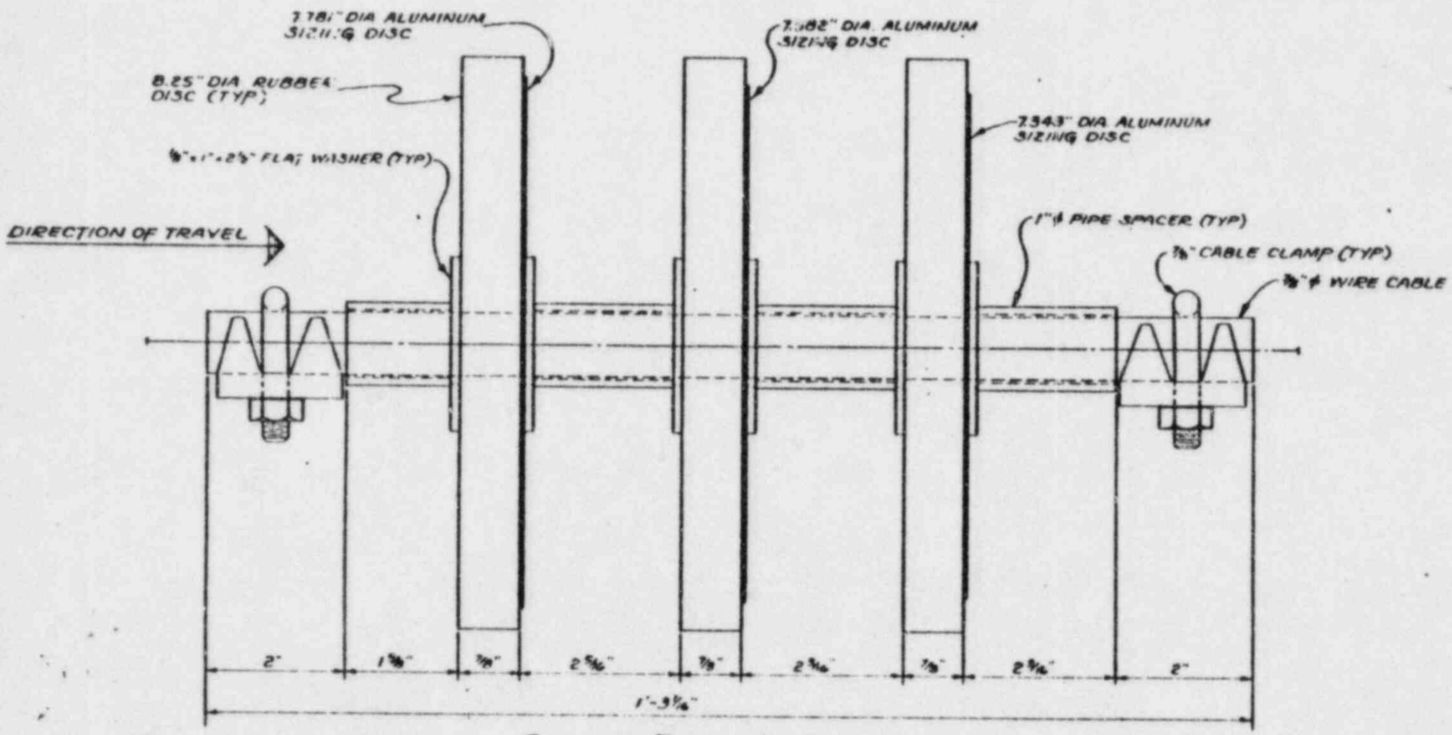
JWF/mls



PIG LAUNCHER
SCALE: ¼" = 1"



PRESSURE ASSEMBLY
SCALE: ¼" = 1"



SIZING PIG - SIDE VIEW
SCALE: ⅜" = 1"

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

FOR CONSUMERS POWER COMPAN
NUCLEAR FACILITY
MIDLAND, MICHIGAN

AS NOTED
11-1-81

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

J. Kane
Rec'd 12/18/81

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 5
ASME code requirements and are considered sound and conservative[2].

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 5

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

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.	5
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%	

J. Kane
Rec'd 12/18/81

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:

$$\begin{array}{lll} h = H & X = 0 & t > 0 \\ h = 0 & X > 0 & t = 0 \end{array}$$

10

The solution adapted from Bear, 1972, is:

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

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):

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

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

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

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 x 7.48 gal/ft^3 x $\frac{1}{8 \text{ ft} \times 8 \text{ ft}}$

$\bar{m}$ = 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)].