UNITED STATES OF AMERICA
NUCLEAR REGULATORY COMMISSION

ATOMIC SAFETY AND LICENSING BOARD

In the Matter of	)	Docket Nos. 50-329 OM
	)	50-330 OM
CONSUMERS POWER COMPANY	)	
	)	Docket Nos. 50-329 OL
(Midland Plant, Units 1 and 2)	)	50-330 OL

TESTIMONY

OF

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ON BEHALF OF THE APPLICANT

REGARDING UNDERGROUND PIPING AND TANKS
AT THE MIDLAND PLANT

Midland Plant
Public Hearing Testimony

UNDERGROUND PIPING AND TANKS

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UNDERGROUND PIPING AND TANKS

1.0 BACKGROUND

1.1 SCOPE OF TESTIMONY

This testimony sets forth evidence regarding the present condition of the underground Seismic Category I piping and tanks at the Midland plant, regarding the ability of the piping and tanks to withstand postulated design conditions, including design basis safe shutdown earthquake forces, over the life of the plant, and regarding the ability to monitor the piping over the life of the plant to provide continued assurance of its capability.

1.2 DESCRIPTION OF PIPING

Four categories of buried Seismic Category I piping, ranging in size from 1½ inches in diameter to 36 inches in diameter, serve safety functions at the Midland Plant. The first category consists of the diesel fuel oil lines, which serve the emergency diesel generators. In this category are eight small-bore pipes, of which four are 1½ inches in diameter and four are two inches in diameter. These pipes provide fuel oil supply and return between the emergency diesel generators and four diesel fuel oil storage tanks buried in the vicinity of the diesel generator building.

The second category of piping consists of borated water lines, which provide borated water for emergency volume and reactivity control from the borated water storage tanks for normal functions and for such postulated accidents as a pipe break in the reactor coolant system. Four 18-inch pipes are included in this category.

The third category consists of piping in the service water system which supplies water to various systems needed under normal and accident conditions. Twenty-two lines, ranging from 8 inches in diameter to 36 inches in diameter, are included in this category.

The last category consists of piping and tanks in the control room pressurization system, which supply overpressurization air to the main control room from two tanks buried in the vicinity of the auxiliary building during postulated accidents such as releases of hazardous gases from offsite storage areas. One 4-inch pipe, one 1-inch, and 2 tanks are included in this category.

Table UP-1 contains a detailed listing of the Seismic Category I piping which is included in the scope of this discussion. Figure UP-1 indicates the locations of the buried piping and tanks. The diesel fuel lines run from the buried diesel fuel oil storage tanks northerly and then westerly to the diesel generator building. The borated water lines run a short distance in a generally southerly direction from the borated water storage tanks to the auxiliary building. The service water lines have the longest lengths, running from the service water pump structure to the auxiliary building and to the diesel generator building. The control room pressurization piping runs from the buried pressurization tanks westerly to the auxiliary building.

The diesel fuel oil lines, service water lines, and one of the control room pressurization lines are of carbon steel. The borated water lines and one of the control room pressurization lines are of stainless steel. The 18 inch and larger diameter pipes are seam welded, while the smaller lines are seamless. The pipes are fabricated in nominal lengths ranging approximately from 4 to 40 feet, fitted up, and welded. The welds are inspected and hydrostatically tested to assure integrity.

As has been previously indicated in this hearing, the construction excavations between the major power block buildings were filled with heterogeneous backfill material. Because of the location of the piping discussed herein and because of the depth at which it is buried, all of the pipes and associated tanks within the scope of this testimony rest on compacted backfill material.

As a result of the detection of insufficiently compacted fill material at a number of locations in the power block area, the Applicant initiated investigations to evaluate fill material contitions. Based on the results of the investigation the Applicant has undertaken a program of measurement, analysis and monitoring to assure that the buried piping and tanks can perform their intended functions throughout the life of the plant under both normal and accident conditions.

2.0 SOILS CONDITIONS AND PREDICTED FUTURE SETTLEMENT

2.1 RESULTS OF TEST BORINGS

As part of the investigation of the compaction of fill material conditions, extensive soil borings were taken throughout the power block area. Logs of exploration borings along the pipelines indicate that subsurface soil consists of heterogeneous compacted fill from ground surface at elevation 634 down to approximately elevation 600. The fill material rests on naturally occuring very dense sands and gravels or hard silty clays.

The heterogeneous compacted fill is primarily composed of silty clays and sandy clays that were excavated from on-site borrow areas. Contained within the site fill are pockets of sand and lean concrete placed during filling operations or during subsequent excavation and backfilling activities.

The records of exploration borings indicate that the consistency of the fill at the location of buried utilities, including piping, varies from soft to hard for silty clays and loose to dense for sands. Generally, the fill soils can be classified as medium stiff or medium dense below invert elevations of buried piping and other utilities. Exploration boring logs also indicate that the consistency of fill material can vary considerably in a vertical direction within a boring and also laterally, as evidenced by closely spaced borings.

Settlements that have been observed at buried utilities are primarily a result of the fill settling under its own weight. Areas that have been subjected to surcharge loading, such as the diesel generator building area and the borated water storage tank area, exhibit additional settlement from surcharging. The buried utilities add little, if any, weight to the fill and therefore have very little impact on present and future settlement below their invert elevations.

Depth profiles along pipelines were compared with subsurface conditions projected from adjacent exploration borings. No correlation could be established between lower profile areas and softer underlying fill soils or between higher profiles and stiffer underlying fill soils. In areas where closely spaced borings indicate stiffer soils and softer soils adjacent to one another, no abrupt differential variations were observed in the pipeline profiles.

2.2 PREDICTION OF MAXIMUM FUTURE PIPE SETTLEMENT

Records of monitored settlement within the fill have been utilized to predict future settlement for buried utilites. Borros anchors have been installed at eight locations in the vicinity of buried utilities not influenced by surcharge loadings. Settlement readings for anchors that have been established at depths of 7 feet to 12 feet below the surface were used in the analysis, since this depth is representative of the depth of most buried utilities. Soil conditions at these locations are representative of the variable soil conditions encountered throughout the fill.

Borros anchors BA 13, BA 14, and BA 34 were installed in December 1978. Settlement data have been taken on these anchors for over three years. Borros anchors BA 100 through BA 106 were installed in September 1979, and over two years of settlement data exist for these anchors. The plots of settlement verus log-time for each of these anchors form straight lines which extrapolate to 2.0 to 2.5 inches of additional settlement to occur over the next 40 years of anticipated plant life. Based on these projections, a conservative estimate of future maximum settlement of buried utilities is for not more than 3 inches of additional settlement to occur at any pipe location.

The maximum differential settlement along the longitudinal axis of buried utilities is anticipated to occur at anchor points, which may be at or near building entry. The maximum critical differential settlement expected along buried piping will be the difference between the future projected settlement of the building entered at the anchor locations and the maximum estimated settlement of the fill in which the pipeline is buried.

3.0 ASSURANCE OF SERVICEABILITY

The serviceability of the buried piping over the life of the plant will be assured by existing measurements and analysis coupled with a program of long-term monitoring, or by excavation, rebedding and rewelding particular piping as appropriate.

3.1 DIESEL FUEL PIPING AND STORAGE TANKS

The diesel fuel oil lines were installed in June 1980 after completion of the diesel generator building surcharge program. The small diameter flexible pipe in these lines can accept the predicted future plant fill settlement without exceeding allowable limits. The maximum settlement stress has been calculated assuming that the maximum value of three inches of predicted settlement was apportioned over a 40-foot span of pipe corresponding to the spacing between pipe footings. The highest stress value was 18 ksi. This value is well within the allowable stress of 45 ksi for these lines based on ASME Code (Reference 1).

The diesel fuel oil storage tanks were installed approximately two years after the fill was placed. This isolated the tanks from the effects of the initial settlement of the fill. The tanks were filled with water and the settlement monitored for approximately 8 months. The settlement of the tanks during this period was minimal (less than 0.2"). It has been estimated that the tanks will experience long-term settlement on the order of 1¼" during plant life. These buried tanks will settle with the surrounding soil. The connecting pipes will also settle with the tanks in the surrounding soil. Thus, the differential settlement between the pipes and tanks will be small. Nozzle loads due to settlement will be insignificant.

3.2 BORATED WATER PIPING

The borated water lines will be rebedded from the borated water storage tank valve pits to the dike around the outdoor tanks. These lines have been cut loose from the valve pits to isolate them from the settlement caused by the surcharge of the valve pits. This partial rebedding in conjunction with the existing program to monitor future settlement of the borated water storage tank and the auxiliary building will provide sufficient assurance of the continued serviceability of this piping.

3.3 CONTROL ROOM PRESSURIZATION LINES AND TANKS

This system was installed in early 1981 in a manner equivalent to that utilized for rebedding other piping. The late installation after the occurrence of major fill settlement in a manner equivalent to rebedding provides sufficient assurance of continued serviceability of the pipes and tanks in this system.

3.4 SERVICE WATER PIPING

3.4.1 Locations and Alignment

Extensive measurement data have been taken to define the present settled condition of the piping. The original position immediately after installation is less well defined. It is difficult to ascertain precisely how much of the current profile originated from settlement since installation and how much of it was due to the position of pipe after backfilling the pipe trench. For the purpose of assuring serviceability, it has been conservatively assumed that <u>all</u> deviations from design location are due to settlement.

In 1979 elevation or profile data were taken for one pipe line in each pipe trench. In June 1981 the Applicant retained Southwest Research Institute to develop a more accurate measurement technique and to reprofile all the service water piping which is 26 inches and larger in diameter using the new technique. The measurement technique uses pressure and ultrasonic tranducers and is accurate to 1/16 inch. The current location of the piping is very well defined from these accurately measured profile data taken at five foot intervals along the pipe length. The measurements have also identified the pipe spool weld joints. The pipe profiles for the large pipes are shown in Figs. UP-2 to UP-7.

The pipe profile measurement technique is based on the manometer principle. The technique measures changes in elevation by measuring the change in height of a reference water column. The instrumentation system maintains the reference water column constant and measures the change in pressure required to maintain the reference column of water at a constant elevation.

The components of the system consist of a water hose, water syringe, water level indicater, pressure transducer, ultrasonic transducer and several voltage readout devices. The pressure transducer is connected to the end of the water hose and the change in pressure is recorded as a change in voltage at the readout device. The ultrasonic transducer is used to monitor the reference column of water which is held constant by adding or removing water from the level indicator with the syringe. This reference is maintained very accurately by monitoring an electronic signal from this device.

The measurement procedure consisted of a series of steps. The piping was cleaned and the measurement locations were marked and the pipe spool length welds were identified. The measurement locations were chosen to be two inches on either side of the circumferential weld and at approximately five foot intervals along the pipe spool. At pipe fittings, measurements were taken at closer intervals. A datum point was then established for the particular piping run to be measured and the instrument system was calibrated to this datum elevation. Once the datum was established the water hose and pressure transducer was manually positioned on the marked measurement locations inside the piping. The change in elevation was recorded by reading the change in voltage at the pressure transducer readout meter. The results of these measurements show that the service water pipe is 8 to 12 inches from the design elevation in some extreme locations and the majority of the piping is on average approximately 5 inches from its design location.

3.4.2 Ovalization Measurements

The serviceability of the piping is indicated by out-of-roundness/ovalization measurement data. Measurement of ovalization is an indirect measurement of the stressed condition of the piping because ovalization of the installed piping is a function of the bending curvature of the piping. (See Section 3.5 below.) These ovalization measurements were taken internally at the same locations as the profile points of the piping.

A measurement arm and jig was fabricated by Southwest Research Institute to record the maximum and minimum diameters of the piping at a given cross-section. From this data the percent ovality can be calculated according to the ASME code equation: $(D_{max} - D_{min})/D_{nominal} \times 100$.

The ovalization data was collected at the same measurement locations identified for the 1981 profile data collection. The measurement instrument consisted of a sliding arm mechanism spring loaded to expand or contract to conform to the internal diameter of the piping. This measurement arm has a scale mounted onto it with two sliding blocks which indicate the maximum and minimum diameter. There is also an azimuth scale used to indicate angular position of the maximum and minimum diameters of a given cross-section of pipe. This measurement arm was mounted on a supporting jig designed to maintain the measurement arm perpendicular to the meridional axis of the pipe. At the fittings the arm had to be manually positioned at circumferential locations for a given pipe cross-section to obtain accurate data perpendicular to the meridional axis of the fitting.

The measurement procedure consisted of locating the longitudinal weld seam of the pipe and positioning the measurement arm against that seam. The measurement arm was then rotated around the internal circumference of the pipe cross-section and spring action of the arm maintained contact with the pipe wall. The sliding blocks on the diameter scale move to the maximum and minumum diameters of the cross-section. These diameters were recorded as data along with their azimuth position. Plots of ovalization measurements are also shown in Figs. UP-2 through 7.

The results indicate general ovalizations of 1 to 1.5% with some locations of 2% and greater. The maximum ovalization recorded was 3% in one 36 inch diameter pipe where the pipe enters the service water pump structure.

3.5 OVALIZATION CRITERION

3.5.1 General

When circular pipe is deflected from its normal linear configuration a change in the cross-sectional shape of the pipe accompanies the longitudinal deflection. As the pipe bends, the cross section changes from a circular to a generally elliptical or oval shape, with the minor axis parallel to the direction of bending. The change from circular to elliptical or oval shape is referred to herein as ovalization or ovality. A convenient numerical measure of ovalization is obtained by the formula $(D_{max} - D_{min})/D_o$, where Dmax is the length of the major axis, Dmin is the length of the minor axis, and Do is the nominal diameter.

It is known that if pipe becomes sufficiently bent (deflected), inward collapse of the pipe will eventually result. During such deflection, the pipe becomes increasingly ovalized. It is possible to relate deflection to impending collapse, but ovalization is a more sensitive and direct indicator.

The Applicant has available extensive data on actual ovalization values of the buried 26 inch and 36 inch pipe at the Midland site. This data allows direct assessment of the actual pipe condition as it relates to the possibility of collapse. With the ovalization data in hand, the next step is to develop a criterion whereby observed ovalization can be judged acceptable or unacceptable from an engineering standpoint.

3.5.2 Flow Preservation

The major point of concern with respect to pipe deformation is whether sufficient flow will be maintained in the piping systems to perform safety functions under all anticipated conditions. The amount of flow through any given pipe is a function of the flow area. As a pipe deforms and ovalizes, the flow area is reduced and ultimately may reach the point where sufficient margin does not exist on the design flow to assure the safety function.

The piping at Midland has ample margin to withstand minor diminution in area due to ovalization. However, actual collapse of the pipe must be prevented.

3.5.3 Collapse Phenomena

Ovalization leading ultimately to collapse can come about through either of two mechanisms: first, the application of a specific continuous load (moment) to the pipe, and second, the imposition of a specific deflection on the pipe. There is a significant difference between these mechanisms with respect to the implications of a critical point in the ovalized condition indicating impending collapse.

Fig. UP-8 illustrates the difference in behavior depending on whether load or deflection is applied. A simple hollow-cylindrical beam is clamped on one end and a load (F) applied. In the process of applying the load (F) a continuous measurement of ovalization and deflection (δ_1) is made so that the value of the deflection at which the ovality in the pipe reaches a critical point can be ascertained (Fig. UP-8(a)). If the load (F) is still applied after this point, the pipe will continue to deform and ovalize with no load increase and will collapse rapidly, essentially shutting off flow completely (Fig. UP-8(b)). This is commonly referred to as "load controlled" deflection and collapse.

The second mechanism, deflection, is represented in Figs. UP-8(c) and UP-8(d). In Fig. UP-8(c), the deflection (δ_1) measured in Fig. UP-8(a) at the critical ovalization is applied. Since the end of the pipe is deflection limited, the critical ovality is reached, but uncontrolled collapse will not occur, since the end of the pipe is, by assumption, not forced to deflect further. Should subsequent deflections be applied (δ_2, δ_3), the critical ovality will be exceeded but uncontrolled collapse will still not occur since the motion of the end of the pipe is controlled. This is commonly referred to as a "deflection controlled" phenomenon. The above distinction is essential, since the situation at Midland involves a "deflection controlled" phenomenon. In this situation, piping will not proceed to collapse unless substantial additional deflection is brought about.

The term "collapse" is frequently used to describe different phenomena. As indicated above, in a "load controlled" situation the inability of the specimen to maintain the level of applied load is often referred to as "collapse." The discussion above indicates that, for the buried pipe at Midland, this is an inappropriate definition since the concern is deflection rather than load. A pipe which has reached the point of maximum load capacity can still be deflected further without seriously diminishing its ability to carry required flow. Furthermore, in some "load controlled" tests the applied load decreases at the time of bifurcation but then can be increased again after this phenomenon occurs.

For large (D/t) ratios (pipe diameter divided by wall thickness) (in the range of that of the 36" pipe at Midland) another phenomenon occurs which is also often referred to as "collapse." This phenomenon is more correctly defined as wrinkling or bifurcation. In this phenomenon, the pipe wall in the compression zone of the bending field develops wrinkles and eventually bifurcates in the local region. This bifurcation occurs as a result of the high compressive membrane stress in the local region. It should be recognized that this effect is local and has a minimal effect on the flow area. For a pipe which has developed a local bifurcation, substantial further deflection can be applied and the ovalization process can continue. That is, the pipe has not "failed" with respect to serving its intended function. Thus wrinkling is also an inappropriate definition of "failure."

3.5.4 Data Review

In order to develop a criterion for an allowable value of ovalization for piping at Midland a literature survey was performed. A summary of the critical ovalization data obtained from that survey is presented in Table UP-2. A plot of all data presently available to the Applicant is shown in Fig. UP-8A.

It should be recognized that all of the data are for "load controlled" tests and the distinctions made in Section 3.5.3 with respect to "deflection controlled" test data are applicable. Since "load controlled" critical ovalization is a more conservative measurement than "deflection controlled" collapse, use of a safety factor of as 1.5 between estimated "failure" point and the allowable is appropriate.

Additionally, significant information was uncovered which is not in the form of data but addresses the occurrence of ovalization in buried pipe and provides design guidance. This information supports the data in Table UP-2 and indicates that ovalization of less than 5% are of no concern. These sources suggest that there is no need to consider the collapse phenomenon at all at these values. A list of these sources is given in Reference 3.

3.5.5 Ovalization Criterion

The data exhibited in Table UP-2 and plotted in Fig. UP-8A show considerable scatter. However, an approximate best fit line would indicate a minimum critical ovalization of about 8% for the largest piping at Midland. Application of a conservative safety factor of two would suggest appropriate acceptance ovalization of approximately 4%.

In order to incorporate additional conservatism, however, the Applicant has also conducted an analysis based on the lowest data points. As the following analysis will indicate, these data, excerpted from the work of Merwin and others (Ref. 3 to Table UP-2), must be corrected for yield strength of pipe wall material to be valid. In addition, because of the conservatism already built into the data selection and the testing procedure, a safety factor of 1.5 is appropriate. On the basis of the analysis described below, the Applicant believes that acceptance criteria of 4% for 26-inch pipe and 3% for 36-inch pipe are extremely conservative but are acceptable at this time.

(1) General

The distinction established in Subsection 3.5.3 between the two uses of the term "collapse" is significant in establishing an appropriate ovalization criterion. It appears that bifurcation rather than true collapse occurred in tests reported by Merwin (Reference 3 to Table UP-2). This is substantiated by the data reviewed above. Further, other data available to Merwin indicate that two other tests at a (D/t) ratic of 96 resulted in 6.6% and 5.6% ovality values respectively. Merwin has indicated that in all cases ripples developed in the compression zone during loading. These ripples were approximately 1/16" to 1/8" in magnitude. At the point at which failure on a load carrying basis was reported, one of the ripples would predominate and form a wrinkle (bifurcation) of approximately 1/2" depth. Additional curvature was recorded and the pipe was deflected further without collapse after the bifurcation was produced.

Bouwkamp tested seven specimens of 48-inch diameter pipe with a (D/t) ratio of about 100. The tests were performed under combined bending and axial compression and also included internal pressurization. That author reported longitudinal bending strains prior to buckling (Specimens 1 to 4) in the range of 0.31% to 0.68%, which for a 48-inch diameter pipe represent ovalities of 4.8% to 22.8% respectively. For the remaining specimens (e.g., 5, 6 and 7) Bouwkamp did not clearly indicate the buckling point. In addition, Specimen 5 underwent an atypical two-part loading sequence. Specimen 6 was 0.10 inches in thickness, thicker (D/t = 85) than all others and Specimen 7 was of spiral-weld design.

Three of Sorenson's tests were identified as "not good tests" in his report since failure occurred at a support, indicating improper loading.

(2) Comparison with Code Criteria

Paragraph ND-3552.3(b) of the current ASME Code addresses the problem of a single deflection of a pipe as follows:

> the effects of any single nonrepeated anchor movements shall meet the requirements of Equation (10a):
>
> $$\frac{i\,M_D}{Z} \leq 3.0\ S_c \qquad (10a)$$
>
> Terms same as in ND-3652.1 except:
>
> M_D = resultant moment due to any single nonrepeated anchor movement (such as predicted building settlement), in lb.

Using Equation (10a) of the Code an equivalent ovalization criterion can be developed. Collapse is a yield strength phenomenon, and the equation:

$S_c = 2/3\ S_y$ (2/3 yield strength)

applies.

The measured yield strength of the 36" pipe is 50,000 PSI. Using 2 S_y = 100,000 PSI and Young's Modulus (E) = 30 x 10^6, strain at yield can be calculated as follows:

$$\varepsilon = \frac{2\ S_y}{E} = \frac{100{,}000}{30 \times 10^6} = 0.0033$$

Using the relationship between strain and ovalization set forth in Subsection 3.5.7 and Fig. UP-9, the corresponding ovality for this strain is 4.6%.

(3) Yield Strength vs. Ovality

Professor Merwin's data support the conclusion that ovalization at bifurcation is a strong function of material yield strength. Computer modeling analyses of pipe collapse also indicate that as the yield strength of the material increases bifurcation will occur at lower ovalities.

Using Merwin's data, a curve of ovality versus yield strength plots as a straight line, as shown in Figure UP-8B. This plot indicates that for material at approximately 64 KSI yield strength, zero ovality would result in failure. This is not a physically meaningful result; the curve should properly become asymptotic to zero ovality. However, interpolation rather than extrapolation should be accurate on a straight line basis.

In Figure UP-8C the yield strength values of the 26-inch (45 KSI) and 36-inch (50 KSI) buried pipes at Midland have been shown on the "Design Curve." These values are shown as 6.25% and 4.6% ovality respectively. Using a safety factor of 1.5 based on the above-described difference between load controlled testing and deflection controlled application, the design curve figures must be multiplied by 2/3 to obtain safe limits. Figure UP-8C indicates that two-thirds of the "Design

Curve" value for 26-inch pipe results in an ovality limit of 4.17%. Taking two-thirds of the "Design Curve" value for 36-inch pipe results in an ovality limit of 3.06%. These values provide a margin of 1.5 on the lowest meaningful data points available.

3.5.6 Ovalization vs. Flow Area

Since the real concern is sufficient flow to assure safety function, it is necessary to compare the criterion with reduction in flow area to determine if it is significant.

Pipe Size	Nominal Flow Area (in^2)	Reduction In Flow Area of 4% Ovality (in^2)	Percent Reduction
2	3.356	.00174	.05
6	28.9	.0213	.07
10	78.9	.0773	.10
26	501	.46	.09
36	976	.39	.04

Based on the foregoing, the reduction in flow area at 4% ovalization is shown to be insignificant.

3.5.7 Conversion to Strain Criterion for Monitoring

Since in actual operation the piping under discussion will be filled with fluid, direct internal measurement of ovalization will be impossible. However, ovalization is related simply to longitudinal strain in the pipe, and sensitive and durable instrumentation (see section 5.2 below) is available to measure longitudinal strain. As a result, longitudinal strain will be monitored and converted to ovalization for comparison to the foregoing ovalization criterion.

The theory relating longitudinal strain to ovalization is developed in Reference 4. The equation relating strain to ovalization is

$$\text{Oval.} = \frac{D_{max} - D_{min}}{D_0} = \frac{D_0 - 2W_{90}) - (D_0 - 2W_0)}{D_0}$$

$$= \frac{2(W_0 - W_{90})}{D_0}$$

where

$$W_0 = \varepsilon \frac{\nu a}{2} + \frac{a\varepsilon^2(1 - \nu^2)}{(t/a)^2} \left[1 - \frac{\varepsilon\nu}{16}\right] \qquad W_{90} = - \frac{a\varepsilon^2(1 - \nu^2)}{(t/a)^2}$$

In these equations

a = mean radius
t = thickness
ε = longitudinal strain
ν = Poisson's ratio

A graph of these equations using values appropriate for the 26-inch and 36-inch service water pipe is shown in Fig. UP-9. These curves will be used to convert strain measurement to ovalization.

4.0 STRESS ANALYSIS OF BURIED PIPE

4.1 SEISMIC LOADS

Earthquakes can in principle exert two kinds of influences on buried pipes: faulting and shaking. Faulting is the direct shearing displacement of bedrock which may carry through to the ground surface. However, surface faulting is not a factor in the design of a nuclear power facility at the Midland site (Reference 2). Therefore, faulting was excluded from consideration in the seismic analysis of the buried piping which supports this testimony.

The effects of ground shaking on buried piping are:

1) Axial tension and compression due to traveling seismic wave

2) Shear and bending due to traveling seismic wave

3) Strain caused by dynamic differential movement at connections.

For very long, straight pipes the analysis is based on the assumption that there is no relative motion between the pipe and the surrounding soil. Seismic stresses in the pipe are calculated from the maximum soil strain in the surrounding soil due to the passage of seismic waves. For short pipes, slippage may occur between the pipe and soil and the calculated axial stresses will be proportionately less than those assuming the pipe strain equal to the soil strain. The effects of bends or tees and differential displacements at connections to buildings are analyzed using procedures based on equations for beams on elastic foundations.

The calculated seismic stresses are combined with stresses from other loading conditions according to the recommended appropriate ASME code equations for the final design.

4.1.1 Maximum Soil Strain

To determine the maximum soil strain, the maximum axial and bending strains due to the different types of seismic waves were calculated. The seismic waves considered were:

1) Compression wave
2) Shear wave
3) Surface (Rayleigh) wave

The maximum axial and bending strains for each wave type were based on the wave propagation velocity, maximum particle velocity and maximum particle acceleration corresponding to each wave type. For example, the maximum axial strain due to a compression wave is calculated by:

$$\varepsilon_{ap} = \pm \frac{V_{mp}}{C_p} \cos^2\theta$$

where: ε_a = maximum axial strain due to a compression wave

C_p = compression wave propagation velocity

V_{mp} = maximum compression wave particle velocity

θ = angle of incidence of the propagating wave measured from the longitudinal axis of the pipe. The maximum axial strain will occur when $\theta = 0$.

The value of wave propagation velocity used was the effective velocity of the ground motion disturbance past the pipe. For the Midland site, the effective velocity of the ground motion disturbance is the wave velocity of the underlying bedrock. (For further explanation of this point, see Reference 5.) The wave propagation velocities of the underlying bedrock at the Midland site were determined from on site tests conducted by Weston Geophysical Engineers, Inc. The bedrock wave velocities are:

Compression wave	10,000 fps
Shear wave	5,000 fps
Surface wave	4,675 fps*

The values of maximum particle velocity and acceleration for each wave type were conservatively assumed to be equal to the maximum site acceleration and velocity for the particular earthquake under consideration, either Operating Basis Earthquake (OBE) or Safe Shutdown Earthquake (SSE).

The maximum axial and bending strains for each type of seismic wave were combined by the square root of the sum of the squares method. The maximum combined axial and bending strains were added to find the maximum soil strain.

4.1.2 Bends and Tees

In the case of a long straight pipe buried in the soil, the transfer of soil strain as axial strain into the pipe depends on the end bearing of the pipe against the soil and the frictional resistance between the pipe surface and the

*Calculated from the compression and shear wave velocities.

soil. Portions of the pipe far from the ends are assumed to move and deform with the soil. At the ends, frictional resistance will develop for some length along which the pipe will displace relative to the surrounding soil due to strain incompatibility between the soil and the pipe.

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 (the leg parallel to the direction of maximum soil strain.) The pipe bends were analyzed as flexible bends with the flexibility coefficients calculated in accordance with Reference 1. Each bend was analyzed twice (once for maximum soil strain parallel to each leg). The results from the two analyses were combined by the square root of the sum of the squares method.

The modulus of subgrade reaction (spring value for the elastic foundation) was based on the shear modulus (G) of the soil. The shear modulus is calculated as follows:

$$G = \rho C_s^2 (\alpha)$$

where:

ρ = Mass density of the soil

C_s = Shear wave velocity of the soil

α = Ratio of soil shear modulus at seismic strain to the shear modulus at low shear strain (10^{-4}%)

The value of shear wave velocity used was 500 fps and was determined by onsite testing. This value is the shear wave velocity in the fill at the level of the pipe and is not the same as the value used in the calculation of the maximum soil strain. The value of the ratio is determined from a relationship developed by Seed and Idriss for sands and depends upon the soil shear strain due to the deflection of the pipe through the soil.

Since the soil properties depend upon the deflection of the pipe and the deflection of the pipe depends upon he soil properties, an iterative procedure must be used to arrive at the final solution. The steps of the procedure are as follows:

1) estimate deflection
2) calculate soil properties
3) calculate new deflection
4) compare deflections
   - if within required accuracy - stop
   - if not -- use new deflection as an estimate and go to step 2

The analysis of tees is the same as that for bends except for the equations used in calculating deflections.

4.1.3 Connections

The connections of all buried Seismic Category I piping, except some of the diesel fuel lines, to buildings at the Midland site are considered to be free connections. That is, there is no rigid attachment at the points where the pipes penetrate the buildings. The first anchors inside the buildings are normally several feet away from the penetration. Therefore, the seismic analysis of the pipes at the penetrations did not consider stresses. Instead, the maximum differential movements between the pipes and the buildings were calculated.

The differential movements between the pipes and buildings consist of two parts: the movements of the pipes relative to the soil and the movements of the buildings relative to the soil. The movements of the pipes in the soil are determined by calculating the axial slippage and rotation at the free ends due to he maximum soil strain. The movements of the buildings are determined independently from the building seismic analyses and are added directly to the pipe movements to arrive at the maximum differential movements. The maximum differential movements between the pipes and buildings are included in the seismic analyses of the piping systems inside the buildings from which pipes stresses at the penetrations are determined. For the lines that are fixed at the building penetrations, some of the diesel fuel lines, the analysis is performed by assuming the lines to act as beams on elastic foundations with the building displacements as input.

4.2 COMPARISON TO ASME CODE ALLOWABLES

The computed seismic loads have been combined with other loads in accordance with Reference 1. The following code equations were used:

1) Equation 8
2) Equation 9 (normal and upset condition)
3) Equation 10
4) Equation 11

Table UP-2 sets forth the preliminary results of the comparison of the computed loads to allowables. The stress allowables are taken from the ASME Code Appendix I (Reference

1) for the materials and operating temperature relevant to the piping under discussion. As the Table indicates, in all cases the code requirements are satisfied.

5.0 MONITORING

The effect of future soil settlement on the service water piping will be monitored using externally mounted strain gage instruments located at various points along the piping system. This technique will allow the Applicant to directly measure the change in internal energy of the pipe wall due to the settlement of the soil.

5.1 MEASUREMENT OF PIPE STRAIN

The measurement of pipe strain will be made using a strain gage attached directly to the surface of the pipe buried in the ground. Some local excavation at the selected monitoring stations will be necessary to attach the instruments. The piping and instrument would then be covered with soil. The strain gage selected is a reliable vibrating wire strain gage. It is used in geotechnical practice and in the mining industry and also has been used to measure the effect of subsidence on oil and gas piping systems. The instrument is a passive mechanical component which requires excitation at the critical frequency of the wire lengths only when a measurement is to be recorded.

5.2 MONITORING INSTRUMENTATION LOCATIONS AND MONITORING FREQUENCY

The criteria to determine where monitoring stations will be located were established from the ovalization measurement data collected on the piping system in the summer and fall of 1981. The following criteria were used:

1. Monitor all points in a pipeline with present ovalization measurements two percent or larger.

2. Four points would be the minimum number of selected points per pipeline if Item 1 was not fulfilled. The remaining points would be selected based on the highest magnitude of ovalization.

3. One monitoring point per pipeline was selected on the basis of highest seismic stress. This seismic point is inclusive in fulfilling the four-point minimum requirement.

4. The first anchor point of all the piping systems will be monitored as the pipe enters a building.

These locations are indicated on the profile and ovalization plots, Figs. UP-2 through 7, and on the monitoring location diagram, Fig. UP-11.

The monitoring frequency selected is consistent with the measurement program for future settlement readings to be taken at the various buildings on the plant site. We have selected the following monitoring schedule:

1. Monitor all 65 stations at 90-day intervals for the first 5 years of plant operation.

2. After the fifth year, monitor the 24 anchor stations on a yearly basis. The need to continue monitoring the field stations will be evaluated at this time from time history plots of the collected data.

3. In case of a seismic event, monitor all stations immediately to ensure the reliability of the service water piping.

4. If the technical specification limit (not yet defined) is reached at a monitoring station, the monitoring frequency shall be increased to a monthly basis at that point until remedial action is taken.

6.0 CONCLUSION

The Applicant has undertaken a program of measurement, analysis, and monitoring to assure that the seismic Category I buried piping and tanks at the Midland nuclear site can perform their intended functions throughout the life of the plant under both normal and accident conditions. The measurement programs have demonstrated that most of the piping and tanks are presently in an acceptable and functionally capable condition. Marginal lines are being modified. Analysis has demonstrated that the piping and tanks have substantial margins to allow for anticipated conditions during the life of the plant. These analyses considered the predicted fill settlement, piping ovality, and seismically induced stresses.

A monitoring program has been identified that utilizes strain gauges located at various points along the piping. The acceptance criteria for the monitoring have been established based on highly conservative considerations of acceptable ovality. This monitoring program assures that the piping condition will be known and acceptable throughout plant life.

REFERENCES

1. American Society of Mechanical Engineers Boiler and Pressure Vessel Code, Section III, Rules for Construction of Nuclear Power Plant Components, Division 1, Subsection ND (1971 ASME Code with addenda through Summer 1973).

2. Midland FSAR, Rev. 39, subsection 2.5.3.

3. Available Information:

   3.1 "Buried Pipelines - A Manual of Structural Design and Installation."

   3.2 R.E. Barnard, "Design and Deflection Control of Buried Steel Pipe Supporting Earth Loads and Live Loads."

   3.3 "Steel Pipe Design and Installation," American Water Works Association.

   3.4 G.G. Meyenkof and C.L. Fisher, "Composite Design of Underground Steel Structures."

   3.5 M.G. Spangler, "The Structural Design of Flexible Pipe Culverts."

   3.6 M.G. Spangler and Donovan, "Applications of the Modulus of Passive Resistance of Soil in the Design of Flexible Pipe Culverts."

   3.7 E.C. Rodabaugh, S.E. Moore, "Evaluation of the Plastic Characteristics of Piping Products in Relation to ASME Code Criteria," NUREG/CR-0261.

   3.8 J.G. Bouwkamp, R.M. Stephen, "Large Diameter Pipe Under Combined Loading," ASCE Journal of Transportation, Volume 99, 1973.

4. J.D. Wood, "The Flexure of a Uniformly Pressurized Circular, Cylindrical Shell," Journal of Applied Mechanics, December 1958, Pages 453-458.

5. M.J. O'Rourke, S. Singh, R. Pikul, Seismic Behavior of Buried Pipelines, Lifeline Earthquake Engineering -- Buried Pipelines, Seismic Risk, and Instrumentation. Third National Congress on Pressure Vessels and Piping (San Francisco, California, June 25-29, 1979).

TABLE UP-1

SEISMIC CATEGORY I LINFS

A. Service Water Lines

8"-1HBC-310	26"-OHBC-53
8"-2HBC-81	26"-OHBC-54
8"-1HBC-81	26"-OHBC-55
8"-2HBC-310	26"-OHBC-56
8"-1HBC-311	26"-OHBC-15
8"-2HBC-82	26"-OHBC-16
8"-1HBC-82	26"-OHBC-19
8"-2HBC-311	26"-OHBC-20
10"-OHBC-27	36"-OHBC-15
10"-OHBC-28	36"-OHBC-16
	36"-OHBC-19
	36"-OHBC-20

B. Diesel Fuel Oil Lines

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

C. Borated Water Lines

18"-1HCB-1
18"-1HCB-2
18"-2HCB-1
18"-2HCB-2

D. Control Room Pressurization Lines

4"-ODBC-1
1"-OCCC-1

TABLE UP-2

CRITICAL OVALIZATION MEASUREMENTS

Investigator	Date	D/T	% Ovality at Collapse	Reference No.
Sorenson	1970	99.8	9.	2
		75.	6.	2
		54.6	3.2	2
		40.5	4.0	2
		62.	11.0	2
		55.3	6.0	2
		51.4	8.0	2
		48.6	10.0	2
		39.8	10.8	
Reddy (Steel)	1978	67	9.0	1
		51	11.8	1
Wilhoit & Merwin	1972	78	3.0	3
		62	8.0	3
		46	8.0	3
		31	no failure	3
Merwin		96	6.6	*
		96	5.6	*

Table References

1. B.D. Reddy, "An Experimental Study of the Plastic Buckling of Circular Cylinders in Pure Bending," International Journal of Solids and Structures, Volume 15, Pages 669-683.

2. J.E. Sorenson, et al, "Buckling Strength of Offshore Pipelines," Battelle Memorial Institute, July 13, 1970.

3. J.O. Jirsa, Fook-Hoy Lee, J.C. Wilhoit, and J.E. Merwin, "Ovaling of Pipelines Under Pure Bending," 4th Annual Offshore Technology Conference, May 1972.

*Private Communication.

TABLE UP-3

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

Stresses in PSI

		Normal EQ. 8		Upset EQ. 9		Faulted Code Case 606		Thermal EQ. 10	
Line No.	Description	Actual Stress	Allowable Stress	Actual Stress	Allowable Stress	Actual Stress	Allowable Stress	Actual Stress	Allowabl Stress
36/26-OHBC-15	S.W. Supply	2442	17,500	6060	21,000	12,536	42,000	5,214	26,250
36/26-OHBC-16	S.W. Return	2442	17,500	7505	21,000	26,383	42,000	10,420	26,250
36/26-OHBC-19	S.W. Supply	2442	17,500	9190	21,000	26,953	42,000	10,814	26,250
36/26-OHBC-20	S.W. Return	2442	17,500	9190	21,000	27,232	42,000	21,613	26,250
26 -OHBC-53	S.W. Supply	1742	17,500	5438	21,000	17,378	42,000	12,513	26,250
26"-OHBC-54	S.W. Return	1742	17,500	5218	21,000	22,223	42,000	25,009	26,250
26"-OHBC-55	S.W. Supply	1742	17,500	4370	21,000	13,802	42,000	13,857	26,250
26"-OHBC-56	S.W. Return	1742	17,500	5296	21,000	13,582	42,000	--	--
10"-OHBC-27	S.W. Supply	695	15,000	5740	18,000	14,750	36,000	-------	Not A
8"-1HBC-81	S.W. Supply	695	15,000	5740	18,000	14,750	36,000	-------	Not A
8"-2HBC-310	S.W. Supply	695	15,000	5740	18,000	14,750	36,000	-------	Not A
8"-1HBC-310	S.W. Supply	625	15,000	2625	18,000	8,077	36,000	-------	Not A
8"-1HBC-311	S.W. Return	625	15,000	1297	18,000	9,875	36,000	-------	Not A
8"-2HBC-81	S.W. Supply	625	15,000	2625	18,000	5,462	36,000	-------	Not A
8"-2HBC-82	S.W. Return	625	15,000	1455	18,000	5,864	36,000	-------	Not A
10"-OHBC-28	S.W. Return	695	15,000	4403	18,000	12,155	36,000	-------	Not A
8"-1HBC-82	S.W. Return	695	15,000	4403	18,000	12,155	36,000	-------	Not A
8"-2HBC-311	S.W. Return	695	15,000	4403	18,000	12,155	36,000	-------	Not A

*Analyses are underway. These analyses will not affect the outcome of the settlement assessment.

e	EQ. 11 Actual Stress	Allowable Stress
	--	--
	--	--
	--	--
	--	--
	--	--
	--	--
	--	--
	29,437	43,750
nalyzed*	-------	
nalyzed*	-------	
nalyzed*	-------	
nalyzed*	-------	
nalyzed*	-------	
nalyzed*	-------	
nalyzed*	-------	
nalyzed*	-------	
nalyzed*	-------	
nalyzed*	-------	

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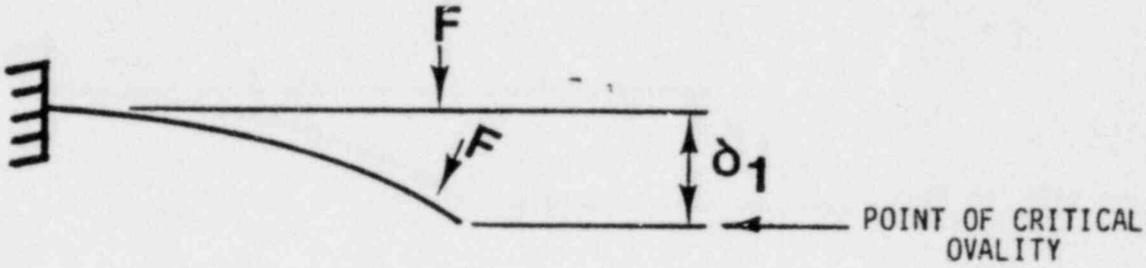

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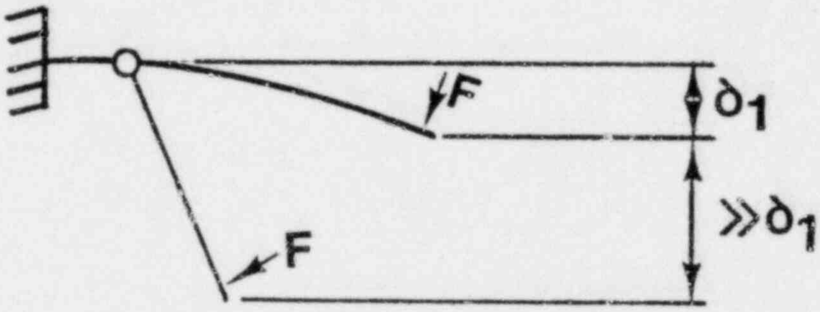

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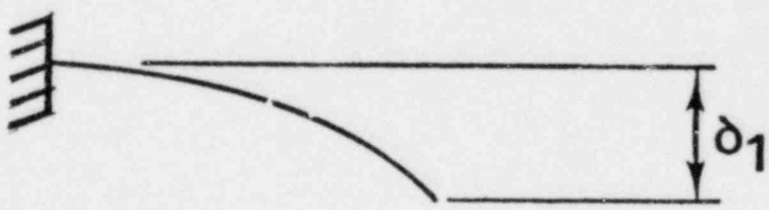

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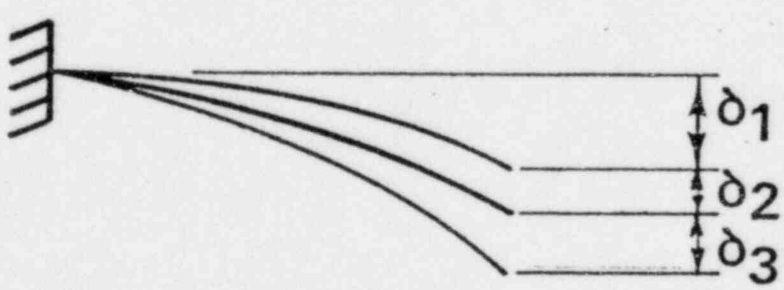
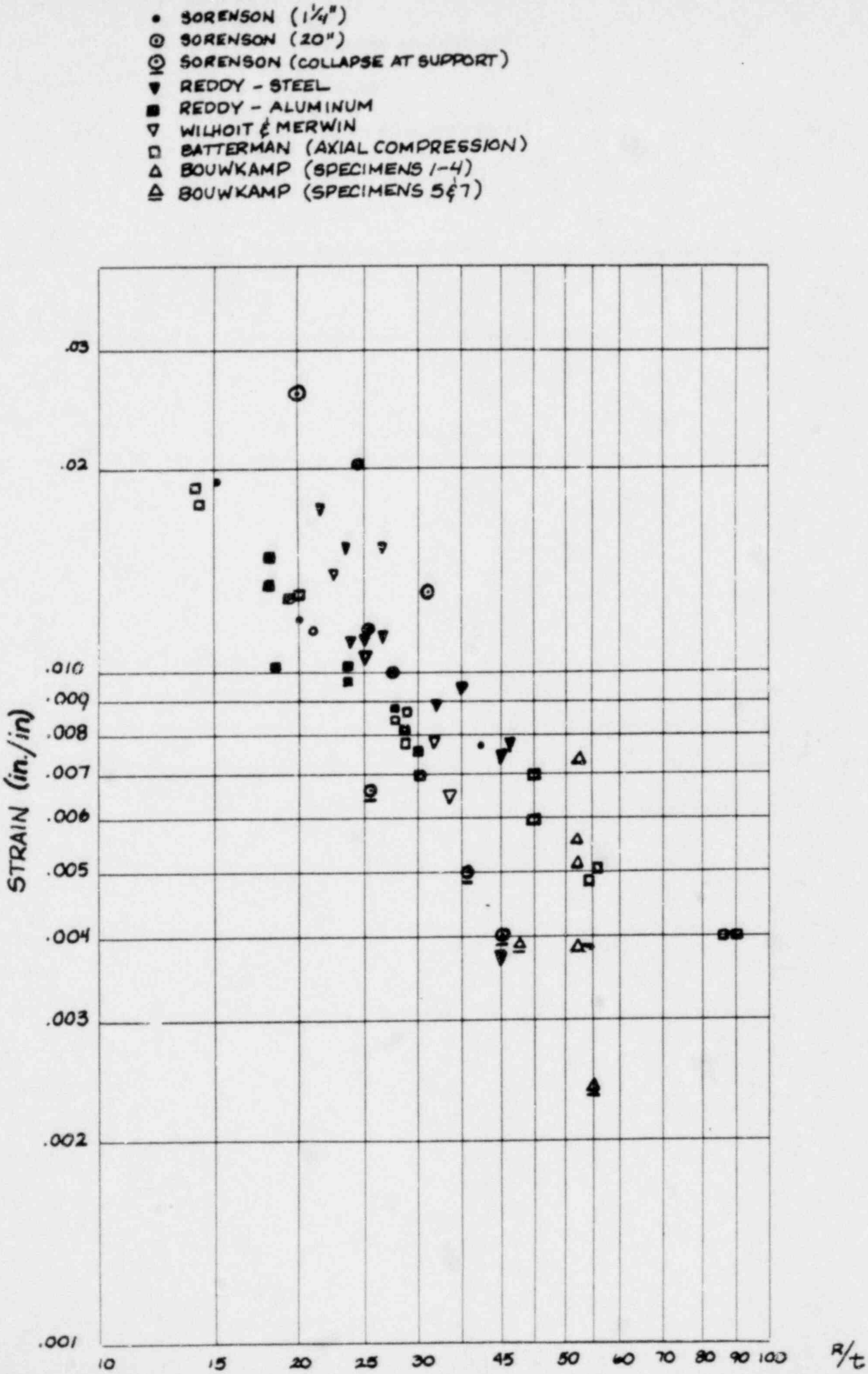


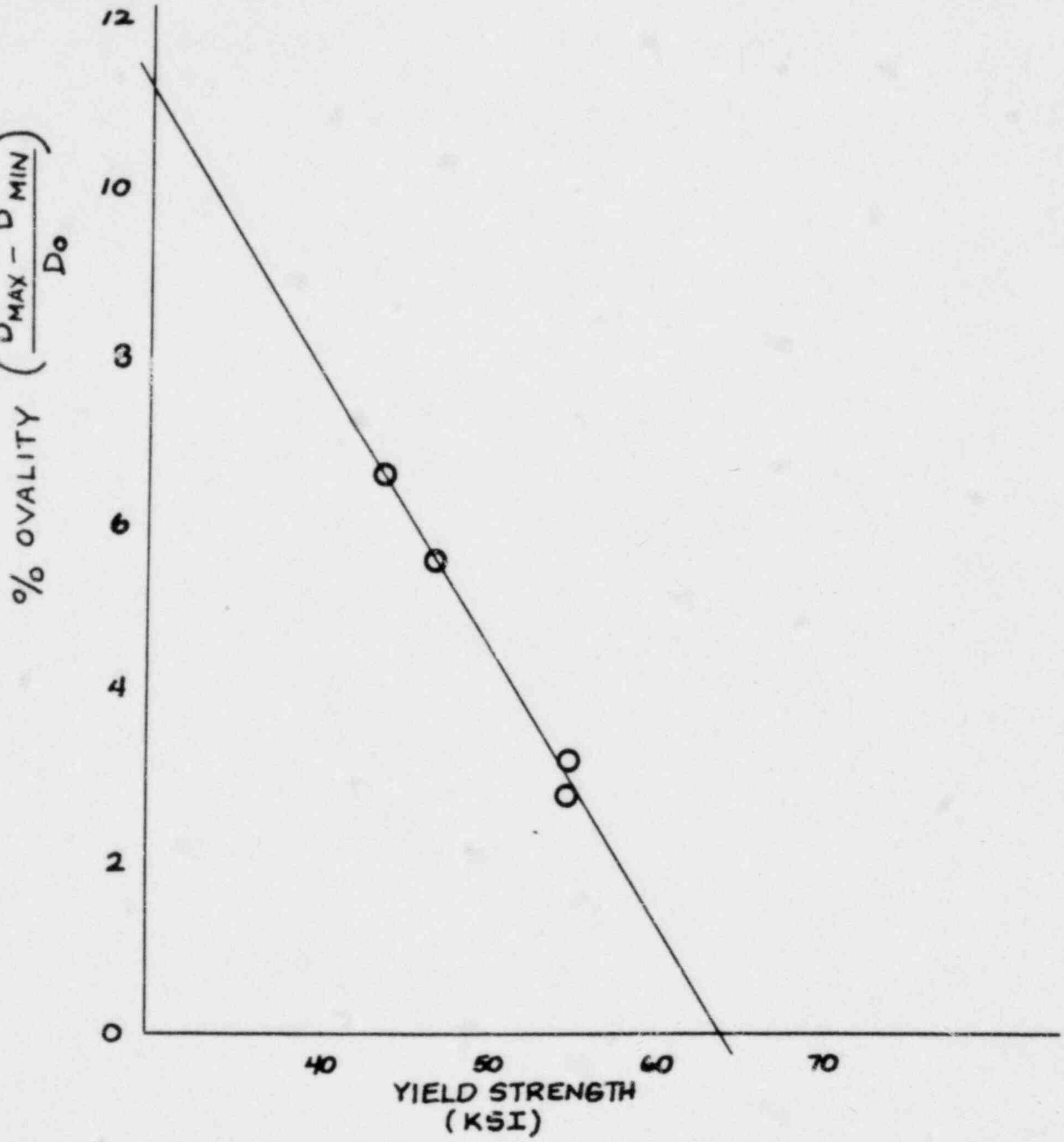
1(d)

FIGURE UP-8



SORENSON (1¼")
SORENSON (20")
SORENSON (COLLAPSE AT SUPPORT)
REDDY - STEEL
REDDY - ALUMINUM
WILHOIT & MERWIN
BATTERMAN (AXIAL COMPRESSION)
BOUWKAMP (SPECIMENS 1-4)
BOUWKAMP (SPECIMENS 5 & 7)
STRAIN (in./in)
.03
.02
.010
.009
.008
.007
.006
.005
.004
.003
.002
.001
10
15
20
25
30
45
50
60
70
80
90
100
R/t

FIGURE UP-8A



12
10
8
6
4
2
0
% OVALITY $\left(\frac{D_{MAX} - D_{MIN}}{D_o}\right)$
40
50
60
70
YIELD STRENGTH
(KSI)

FIGURE UP-8B

TREND CURVE USING 2/3 OF DESIGN CURVE

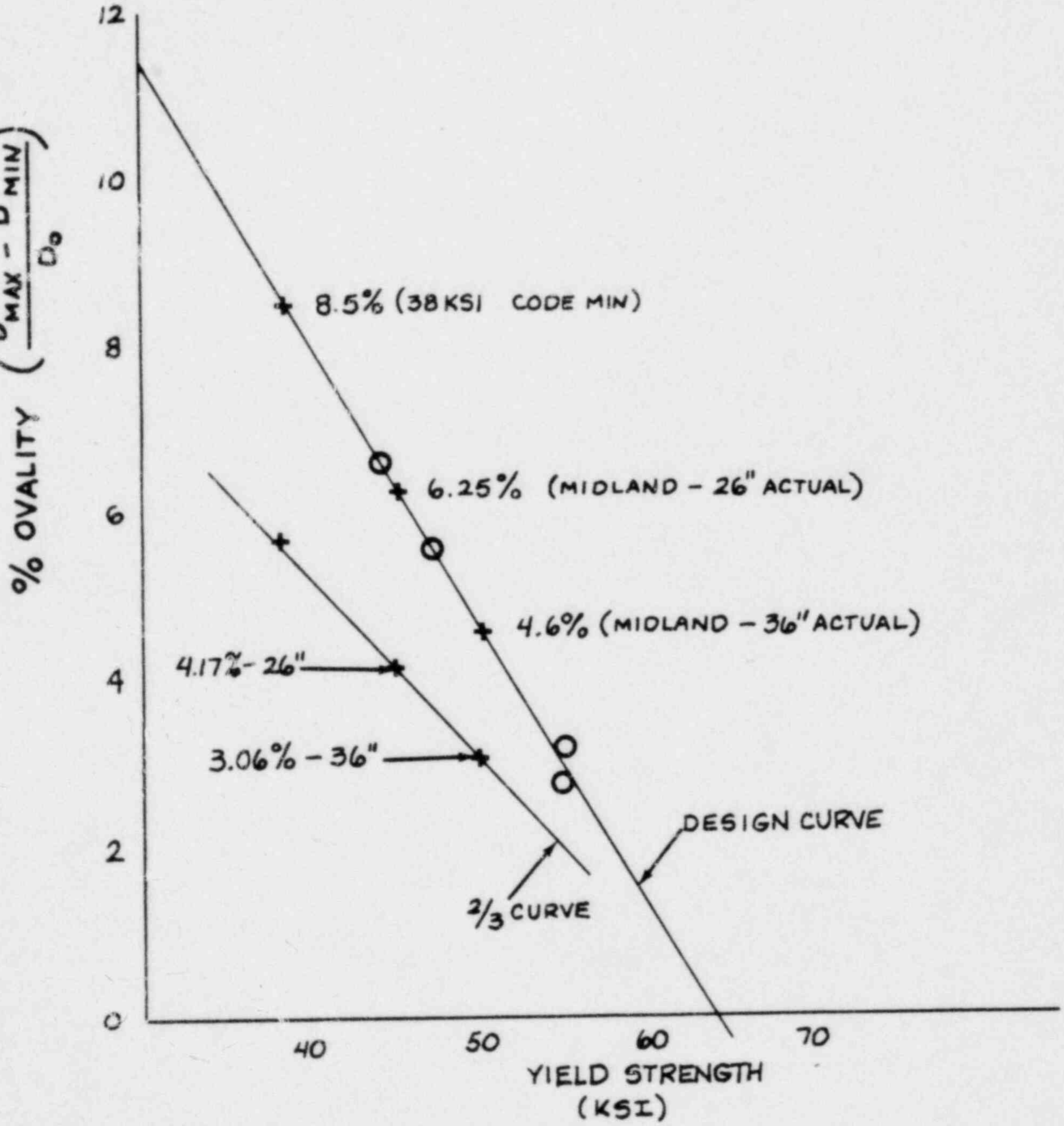


FIGURE UP-8C

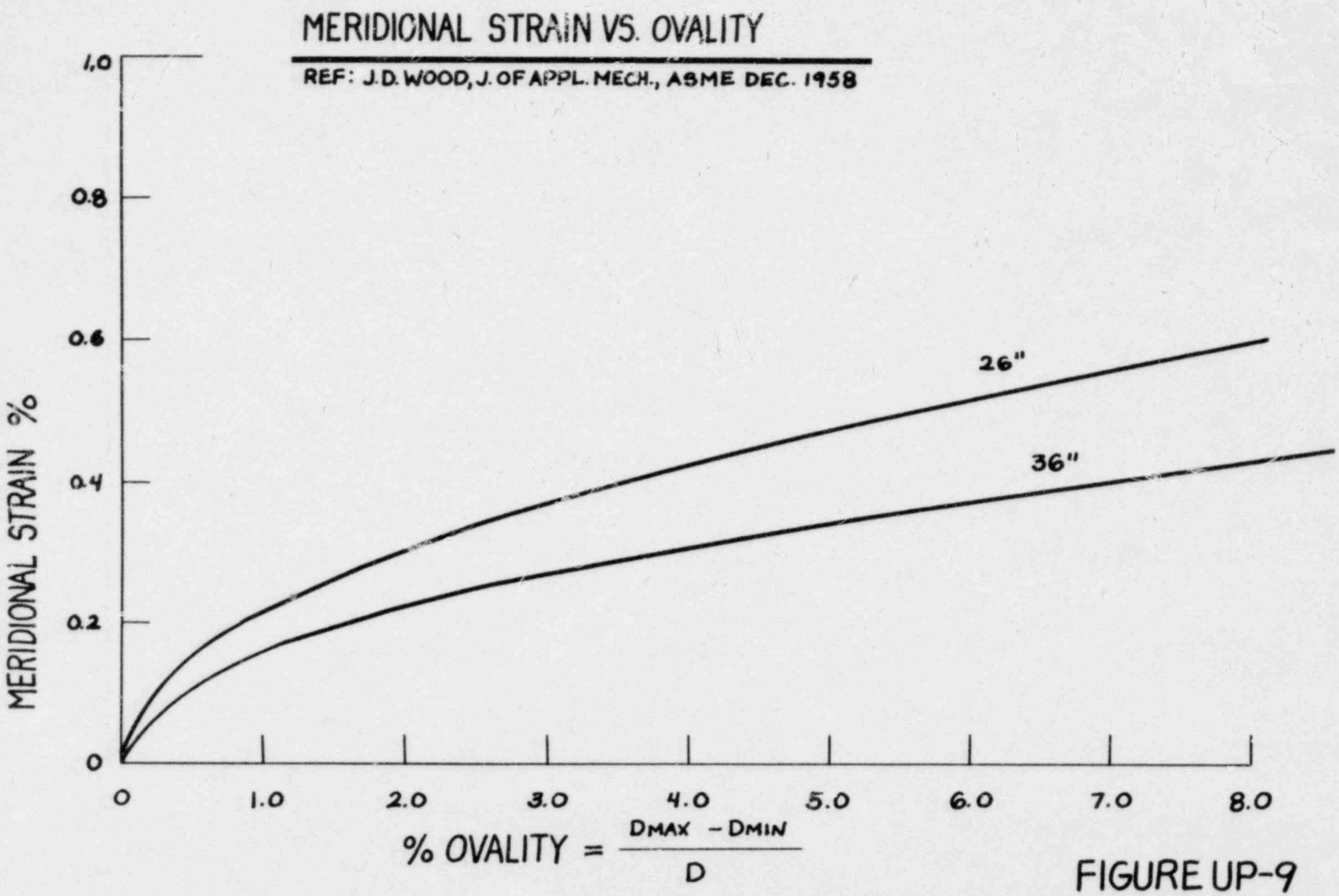
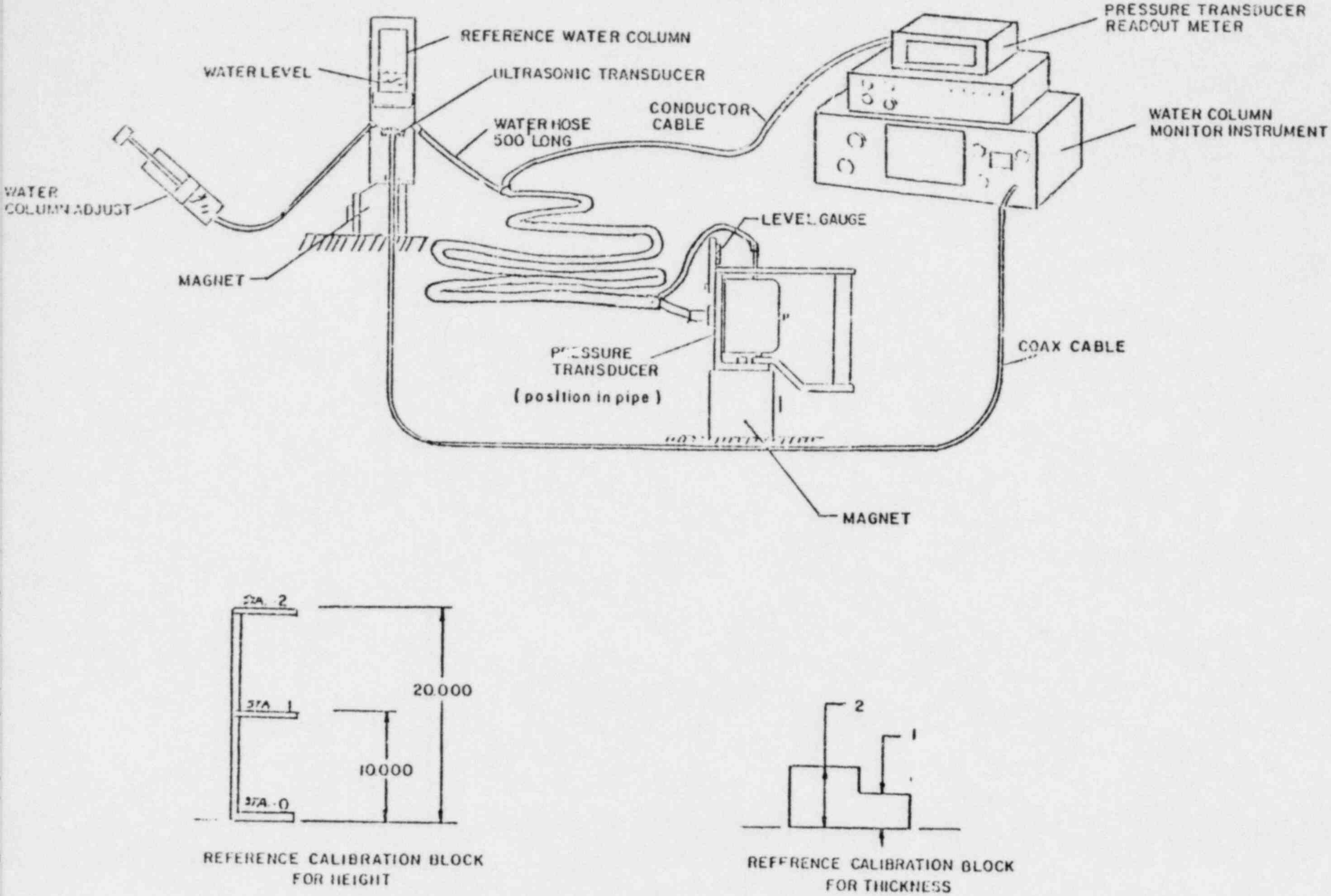

MERIDIONAL STRAIN VS. OVALITY
REF: J.D. WOOD, J. OF APPL. MECH., ASME DEC. 1958
MERIDIONAL STRAIN %
0
0.2
0.4
0.6
0.8
1.0
26"
36"
0
1.0
2.0
3.0
4.0
5.0
6.0
7.0
8.0
% OVALITY = DMAX - DMIN / D

FIGURE UP-9



SCHEMATIC - PIPE ELEVATION PROFILE MEASURMENT SYSTEM

FIGURE UP-10

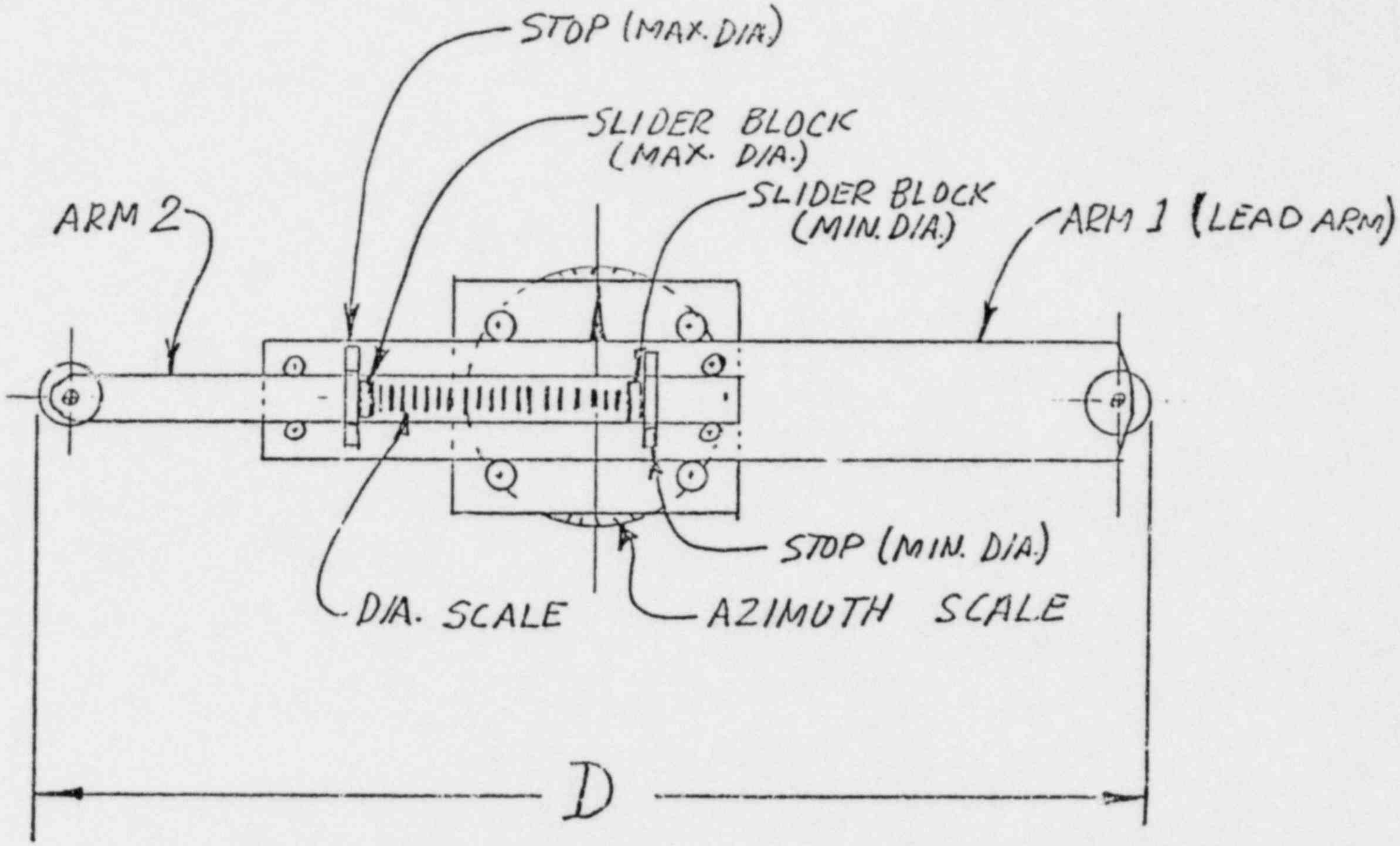
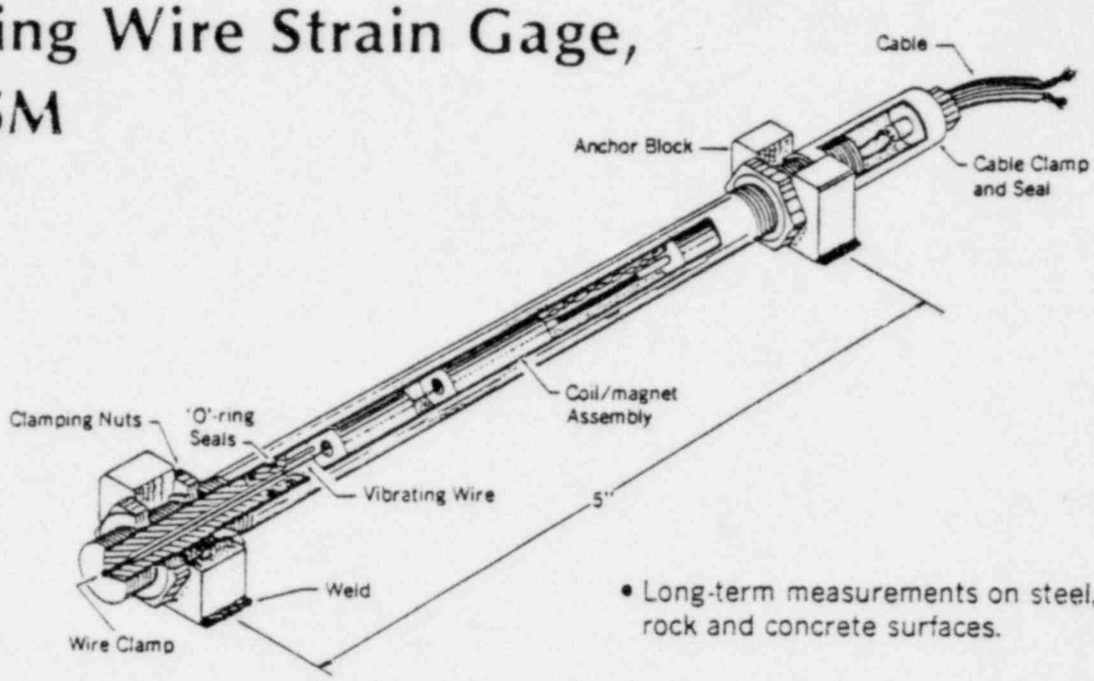


FIGURE UP-11

SKETCH - SwRI OUT-OF-ROUNDNESS MEASUREMENT INSTRUMENT

FIGURE UP-12

Vibrating Wire Strain Gage, Type SM



- Long-term measurements on steel, rock and concrete surfaces.

The IRAD GAGE Type SM Vibrating Wire Strain Gage has been designed to measure strains on structural steel work as well as on the surface of rock and concrete constructions. The gages are rigidly clamped by anchor blocks which are welded or bolted to the structure at predetermined spacing using a special jig. The initial wire tension is set to the required value by rotating the clamping nuts using a standard wrench.

The wire vibrations are measured using a coil/magnet assembly mounted inside the gage. The lead wires to this assembly are brought out through one of the ends. The period of the resonant frequency is easily read on the display of the IRAD GAGE Readout Box MB-6 (or MB-3).

Either tensions or compressions can be monitored and no loads other than those required to tension the vibrating wire are applied to the structure. As the coefficient of expansion of the wire is closely matched to that of the structural steel there are no temperature corrections. If temperature measurements are required they can be monitored by a thermistor (optional extra) mounted inside the gage.

Where gages are susceptible to impact damage such as in high traffic areas or during shotcreting, it is recommended that they be shielded by a metal cover (optional extra).

The gages are provided with heavy duty cable. Further cable protection can be provided by means of flex conduit coupled to the gage cover.

Calibration data are supplied with the gages to enable the observer to convert the period readings to strains.

Specifications

Model No.	SM-5
Active Gage Length (anchor block spacing)	5 inches (127 mm.)
Maximum Strain Range	2000 μ in./in.
Sensitivity	1 μ in./in.
Temperature Range	-40° to 150°F
Overall Length	7½ inches (190 mm.)
Tube Diameter	½ inch (12.7 mm.)
End Block Dimensions - (weldable)	1 x 1 x ½ inches
(bolted)	1½ x 1 x ½ inches
Weight with 10 ft. of cable	1 lb.

Essential Accessories

Setting Jig (bolting type).
Setting Jig (weldable type).
Wrench.
Model MB-3 (or MB-6) Readout Box.

Optional Extras

Thermistors.
Model MT-1 Thermistor Readout.
Gage Cover.
Flex Conduit.

Ordering Information - Model SM-5

Specify: 1. Cable Length.
2. End Block Type. (Weldable or Bolted).

UNITED STATES OF AMERICA

NUCLEAR REGULATORY COMMISSION

ATOMIC SAFETY AND LICENSING BOARD

In the Matter of	)	Docket Nos. 50-329 OM
	)	50-330 OM
CONSUMERS POWER COMPANY	)	
	)	Docket Nos. 50-329 OL
(Midland Plant, Units 1 and 2)	)	50-330 OL

AFFIDAVIT OF DONALD F. LANDERS

My name is Donald F. Landers. I am Senior Vice President of Teledyne Engineering Services, a division of Teledyne Industries. Teledyne Engineering Services engages in the practice of consulting engineering with particular, though not exclusive, emphasis on the engineering problems of nuclear facilities. I am a mechanical engineer with specialization in the field of piping engineering. An outline of my education and professional qualifications is contained in the attached resume.

My experience includes over twenty years of engineering work, including design, fabrication, installation, and testing of commercial nuclear power plant piping systems as well as other categories of high-reliability piping, including fuel piping for Titan missile bases and piping for nuclear surface ships. I am chairman of the ASME Boiler and Pressure Vessel Code Section III, Nuclear Components SubGroup on Design, a member of the Section III Committee and of the Working Group on Piping Design. I am a member of the Pressure Vessel Research Committee of the Welding Research Council and currently chairman of the Technical Committee on Piping Systems. I have been a lecturer at more than 40 seminars throughout the world on the Design of piping systems and ASME Code criteria. A list of my publications relating to nuclear power plant component design is contained in the attached resume.

I am the author of Section 3.5, including all of Subsections 3.5.1 through 3.5.7, of the foregoing Underground Piping Testimony. I am not responsible for any other section of this testimony. I believe that, by virtue of the education and experience set forth in this affidavit and in the attached resume, and as a result of my review of the circumstances of the underground piping at the Midland Plant, I am qualified to testify as an expert with respect to the serviceability of the Midland Plant underground piping.

I declare under penalty of perjury under the laws of the United States of America that the foregoing is true and correct.

Executed on February 2, 1982

Donald F. Landers

Donald F. Landers

DONALD F. LANDERS
Senior Vice President

Professional Resume

Education

Lincoln Technical Institute, A.S. in Mechanical Engineering, 1962
Northeastern University, B.B.A. in Engineering and Management, 1963

Experience

Teledyne Engineering Services, Teledyne Materials Research, and Lessells and Associates, Inc., since 1961: Engineering design, analysis and construction management for nuclear power and fossil power plant modifications; theoretical and experimental stress analysis of piping and pressure vessels; preparation of Design Reports; consulting on design criteria, design specifications, and pressure vessel and piping design and analysis; Design Review of nuclear and LNG piping systems including installation.

Arthur D. Little, 1959-1960: stress analysis and field engineering of fuel loading piping for Atlas and Titan missile bases.

Bethlehem Steel Co., Nuclear Power Section, Central Technical Dept., 1957-1959, 1960-1961: stress analysis of shipboard piping, pipe hanger design, supervision of nuclear piping installation.

Charles T. Main Co., 1955-1957: power plant and textile mill design.

U.S. Navy Weather Forecaster, 1951-1955

Membership

ASME, Boiler and Pressure Vessel Code, Section III Committee Member; Working Group on Piping Design Member; Subgroup on Design Chairman.

Welding Research Council, Pressure Vessel Research Committee

ANSI, B31.7 Code for Nuclear Piping, Member; Chairman, ANSI B31.7 Task Group on Design.

Registered Professional Engineer - Commonwealth of Massachusetts

(over)

→ 10 ; 1 [illegible]

Authorship

"Specification Guidelines for Nuclear Pressure Vessels, " with W.E. Cooper, AEC Report NYO-3416-1, October 1964

"Nuclear Piping Design Guide," with R.D. Hookway, USAEC Division of Reactor Development and Technology RDT Standard.

"Effect of ANSI-B31.7 - 1969 on the General Piping Industry," Heating, Piping and Air Conditioning Magazine, June 1970.

Computer Software - Problems and Preferred Resolutions," ASME Booklet on Computer Software.

"Problems Occuring in Nuclear Piping System Analysis and Operation," Second International Conference on Structural Mechanics in Reactor Technology - Berlin, Germany, 1973.

"B31 Piping Design Philosphy," 1973 Annual Meeting, Mexican Society of Mechanical and Electrical Engineers.

"Design Specifications." ASME Philadelphia and Delaware Sections, 1973,1974 and 1975 Nuclear Power Plant Components Course and 1976 ASME Annual Meeting Short Course.

"Section III - Nuclear Piping Design," ASME 1975 and 1976 Annual Meeting Short Courses.

"Nuclear Piping Design - A Critique ", July 1978.

"Technical Program to Identify Signifigant Problems Related to Piping Systems in LWR Power Plants", August 1980 - Sandia Laboratories.

"Effects of Postulated Event Devices on Normal Operation of Piping Systems in Nuclear Power Plants" with R.D Hookway, TES, and K.D. Desai, USNRC - NUREG/CR-2136, May 1981.

UNITED STATES OF AMERICA
NUCLEAR REGULATORY COMMISSION

ATOMIC SAFETY AND LICENSING BOARD

In the Matter of	)	Docket Nos. 50-329 OM
	)	50-330 OM
CONSUMERS POWER COMPANY	)	
	)	Docket Nos. 50-329 OL
(Midland Plant, Units 1 and 2)	)	50-330 OL

AFFIDAVIT OF DONALD F. LEWIS

District of Columbia: SS:

My name is Donald F. Lewis. I am employed by Bechtel Associates Professional Corporation as the acting assistant project engineer and the engineering group supervisor for the Midland Nuclear Project. In this position, I am responsible for licensing activities, including evaluation of specific design issues with respect to licensi q and technical requirements.

I have a total of fifteen years of experience in the nuclear power industry. Nine of these years have been in the design and construction of commercial nuclear power plants. The balance of my experience has been in the United States Navy as an officer in the Naval Nuclear Propulsion Program. I have a Bachelor of Science degree in Physics from Rensselaer Polytechnic Institute. In addition, during my service as a naval officer, I attended the United States Navy Nuclear Power School in Bainbridge, Maryland and the United States Navy Nuclear Power Training Prototype Unit in West Milton, New York.

In 1973, after leaving the Navy, I went to work for Bechtel Power Corporation as the nuclear steam supply system coordinator on Portland General Electric Company's Pebble Springs Nuclear Project and held the same position on Iowa Power Company's Central Iowa Nuclear Project. In these positions, I was responsible for incorporation of the reactor and reactor auxiliary systems into the plant design, schedule and licensing effort.

Beginning in 1976, I served as a nuclear discipline specialist in Bechtel's Ann Arbor area office. In this position, I was responsible for providing technical assistance to projects on nuclear, environmental, and licensing matters. I have also held the position of mechanical nuclear design group supervisor for the American Electric Power Nuclear Plant studies. I am also the current Vice Chairman of the Michigan Section of the American Nuclear Society, and was a past member of the ANS 51 Standard Committee to develop PWR design criteria.

In connection with my current positions as assistant project engineer and engineering supervisor for the Midland nuclear project, I am responsible for licensing activities with respect to the underground safety related piping and tanks at the Midland Nuclear Plant, as well as evaluation of specific design issues with respect to licensing and technical requirements.

I am primarily responsible for the underground piping testimony, with the exception of Sections 2.1, 2.2, and 3.5, for which James Meisenheimer and Donald Landers are responsible. I affirm that the statements in this affidavit and in those portions of the underground piping testimony for which I am responsible are true and correct, to the best of my knowledge and belief.

Donald F. Lewis

Donald F. Lewis

Subscribed and sworn to before
me this 3rd day of February,
1982.

Phyllis D. Dacey

Notary Public, District of Columbia
My commission expires: 1-1-87.

UNITED STATES OF AMERICA
NUCLEAR REGULATORY COMMISSION

Before the Atomic Safety and Licensing Board

In the Matter of	)	
CONSUMERS POWER COMPANY	)	Midland Plant Units 1 & 2
Application for Reactor	)	Docket No 50-329
Construction Permit and	)	Docket 50-330
Operating License	)	

AFFIDAVIT OF JAMES K MEISENHEIMER

My name is James K Meisenheimer. I am presently employed by Consumers Power Company as the coordinator of geotechnical activities relating to soils remedial work for the Midland Project. I am on leave from my job as Supervisor of Geotechnical Services at Gilbert/Commonwealth which is an architectural and engineering firm specializing in power plant design. I have a BSCE and a MS in geological engineering from the University of Missouri at Rolla. I have over 12 years of professional experience in geotechnical engineering. My resume is attached.

I am the author of that portion enclosed testimony which deals with subsurface soil conditions and future predicted settlement (Section 2.0). My testimony is based on my review of all pertinent data furnished by Consumers Power Company and Bechtel. Based on this review and on my professional experience and training, I believe I am qualified to give this testimony.

I swear that the statements made in this Affidavit, the attached resume and three portions of testimony for which I am responsible are true and correct to the best of my knowledge and belief.

Sworn and subscribed to before me this 1st day of February 1982.

James K. Meisenheimer
James K Meisenheimer
Coordinator of Geotechnical Activities

Barbara R. [illegible]
Notary Public, Jackson County
My Commission Expires September 8, 1984

CC: State of Michigan
County of Jackson

mi0182-0017a100

JAMES K. MEISENHEIMER
Supervisor – Geotechnical Services

Background of over twelve years of professional experience in civil/geological engineering including studies, analyses, cost estimating and construction management of geotechnical phases of dams and cooling systems, power generation projects, highways, subsurface and surface mines, and waste treatment systems with special emphasis on soil and rock mechanics, foundations engineering, and siting studies.

EXPERIENCE: 1977 to Present	**Gilbert/Commonwealth since 1977** Supervisor – Geotechnical Services Section of the Environmental Systems Division responsible for directing the activities of a group of geotechnical engineers and geologists and a soils testing laboratory, involved in site investigations, foundation studies, mining and hydrogeological studies and environmental reports.
1971-77	**Dames & Moore, Park Ridge, Illinois** Project Engineer/Project Manager/Principal Investigator on studies, analysis, design, cost estimating and construction management for geotechnical phases of earth, gravity and tailing dams and cooling systems including construction surveillance of foundation preparation and treatment, geologic mapping and earthwork operations, instrumentation, design of controlled and production blasting, grouting for foundations and grout curtains, rock anchors, dewatering, borrow area development, and selection and development of quarries for riprap materials. Also responsible for exploration, testing and analysis to evaluate static and dynamic stability for existing hydraulic filled dam embankments. Exploration, testing analysis and design for open pit and underground mining to include slope stability, room and piller and long wall mining, shaft sinking, rock bolting and instrumentation. Construction surveillance of nuclear power plant foundations; site development of cooling water and waste treatment systems.
1969-70	**U.S. Army** One year as Engineering Construction Officer involved in development, design and analysis of military construction of 75 miles of South Vietnam national highway. One year as Instructor in soil analysis and construction engineering at the U.S. Army School in Ft. Belvoir, Virgina.
1969 (3 months)	**U.S. Army Corps of Engineers, Kansas City District** Civil Engineer/consultant on $1 million troop housing and facilities project.
1967 (summer) 1965 (summer) 1964 (summer)	**Illinois Department of Transportation, Paris, Illinois** Assistant Resident Engineer on two miles of state highway and storm sewers; quality control and construction inspection of concrete and asphalt mix for highway repair work.

(Continued)

1962-63	Assistant Maintenance Field Engineer involved in design, management and maintenance of 600 miles of state highway.
EDUCATION:	B.S.C.E., University of Missouri at Rolla, 1967 M.S., Geological Engineering, University of Missouri at Rolla, 1969
REGISTRATION:	Professional Engineer in Illinois (1975)
SOCIETIES:	Association of Engineering Geologists American Society of Civil Engineers Earthquake Engineering Research Institute

UNITED STATES OF AMERICA
NUCLEAR REGULATORY COMMISSION

ATOMIC SAFETY AND LICENSING BOARD

In the Matter of	)	Docket Nos. 50-329 OM
	)	50-330 OM
CONSUMERS POWER COMPANY	)	
	)	Docket Nos. 50-329 OL
(Midland Plant, Units 1 and 2)	)	50-330 OL

CERTIFICATE OF SERVICE

I hereby certify that copies of the "Testimony of Donald F. Landers, Donald F. Lewis, and James Meisenheimer on Behalf of the Applicant Regarding Underground Piping and Tanks" in the above-captioned proceeding were served on the persons listed in the attached Service List either by deposit in the U.S. Mail, First Class, postage prepaid, or by hand delivery as indicated in the Service List, on the 5th day of February, 1982.

Frederick C. Williams
Frederick C. Williams

One of the Attorneys for
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