BEHAVIOR OF RIGID AND FLEXIBLE CULVERT PIPES UNDER DEEP FILL

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PREFACE

Since the numerous photographs, sketches, and data plots necessary to the clarity of the report posed problems in layout, these have been appended.

The data collected were so voluminous that the cost of reproducing compilations for inclusion in the report was considered to be prohibitive. However, the raw data are available at copying costs.

SUMMARY

Along a section of Interstate 77 in Carroll County, in the mountainous region of southwestern Virginia, it was necessary to construct a fill approximately 256 ft (78 m) deep. The flow of a mountain stream had to be carried through this massive embankment and, because of the nature of the terrain, it was decided to use a temporary steel culvert and a permanent concrete culvert. The unusual features of this project provided a unique opportunity for a detailed experimental field study of the response of rigid and flexible culverts under deep fill.

A 60-in (1.52-m) nominal diameter corrugated steel pipe approximately 1,875 ft (572 m) long was instrumented and installed under a fill approximately 256 ft (78 m) deep as a part of the construction of Interstate 77. The steel culvert, which served as the temporary carrier of the mountain stream under the roadway, was instrumented to provide for determinations of the longitudinal and circumferential strains and cross section deformations at different heights of fill.

The permanent reinforced concrete culvert had a nominal inside diameter of 96 in (2.44 m), a length of 795 ft (242 m), and was designed for a cover of 163 ft (50 m). The instrumentation associated with the concrete culvert consisted of two transverse rings of deflection gages to permit a determination of the deformed shape of the cross section. In addition, a series of settlement plates was placed in the embankment above and adjacent to the concrete culvert.

Although only limited strain data from the steel pipe were available for analysis, the variation of strain versus fill depth was determined for selected locations. Strain data indicated the existence of high stresses and localized yielding which might be expected for the corrugated geometry used.

Deflection gage data were obtained up to a fill of 256 ft (78 m) for the steel pipe and up to 125 ft (38.1 m) for the concrete pipe. These data appeared to be extremely reliable. Using these data, deformed cross sections of the culverts were plotted for the various fill levels. Knowledge of the deformed cross section also permitted the determination of the circumferential bending moments. Plots were made of crown deflection versus fill depth and the maximum crown deflection measured was determined to be in excess of 6 in (0.15 m) for the steel pipe but only 0.10 in (0.25 cm) for the concrete culvert. Crown deflection was also found

to be approximately linear with fill depth. After completion of the placement of a total fill of 256 ft (78 m), well in excess of the design fill, the steel culvert appears to remain structurally sound. Research personnel who took deformation readings inside the steel culvert after the fill had been completed reported no plate tearing nor serious bolt tipping. However, the pipe has undergone severe local plastic deformations in the region of the crown. The concrete pipe, on the other hand, exhibited rather serious cracking and spalling at longitudinal locations along the spring line.

Settlement curves, plotted from the recorded settlement data, provided detailed information concerning the amount and rate of settlement in the vicinity of the concrete culvert. Settlements of almost 5 ft (1.52 m) were observed as well as two separate periods of rapid settlement.

As of October 1976, both culverts were still intact and useful, although efforts were under way to improve the condition of the concrete culvert before abandoning the temporary steel pipe.

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INTRODUCTION

Problem Statement

It is estimated that hundreds of millions of dollars are spent each year for metal and concrete culverts in the United States. The performance and response of these buried culverts under load are extremely difficult to predict because of the complex soil-structure interaction involved. Thus, it is necessary to recognize the fundamental coupling phenomenon between the culverts and the surrounding medium when evaluating or formulating design recommendations for pipe culverts.

Current conventional methods of analyzing and designing pipe culverts involve many assumptions and are based on largely empirical procedures which may be both obsolete and suspect for present day applications. These observations are particularly true for pipe installations where the fill depth exceeds 100 ft (30 m). If even small improvements in analysis and design procedures can be effected — improvements leading to more efficient culvert structures — it is likely that significant savings will result.

The first step in improving the overall state of knowledge in culvert design is to achieve a clear understanding of the magnitude and distribution of the load to which a buried culvert of known construction and design parameters is subjected. This understanding may be accomplished directly by accurately measuring soil pressures adjacent to and in the vicinity of buried culverts. The same objective may also be achieved indirectly by measuring the stresses or deformations in a buried culvert as it is loaded to its design fill depth and then back figuring the distribution of pressure which is consistent with the measured response of the culvert. Thus, wherever possible, it would seem highly desirable to obtain experimental data regarding the response of culverts which are placed under deep fill. It was for this purpose, to monitor the response of two pipe culverts being buried under a deep fill during construction, that this study was conducted.

Background

As part of the interstate system in Virginia, Interstate 77 was planned to provide a north-south route through the southwestern part of the state. To do so the highway had to pass through a range of the Blue Ridge Mountains. Along this portion, it was necessary to construct a fill approximately 256 ft (78 m) deep. The design called for two pipe culverts, one temporary and one permanent, to carry the flow of a mountain stream through this embankment. The temporary culvert consisted of a 60-in (1.52-m) nominal diameter corrugated steel pipe approximately 1,900 ft (580 m) long and was intended to be utilized during the initial stages of construction. The second culvert was intended as a permanent structure for the stream and consisted of a circular, reinforced concrete pipe with a 96-in (2.44-m) inside diameter to be placed under approximately 163 ft (50 m) of fill. This utilization of dual culverts was chosen for reasons of economy due to it being difficult to obtain a concrete culvert of sufficient size to withstand the full load of the fill.

Few culverts with fill deeper than 256 ft (78 m) have been installed in the United States. Because the corrugated steel pipe was intended only as a temporary carrier and was designed to withstand only a portion of the total fill, there was the possibility that the response spectrum of this pipe might extend through extreme plastic deformation to its total collapse.

The unusual features of this project provided a unique opportunity for a detailed experimental study of the response of culverts under deep fill. Representatives of both the Virginia Department of Highways and Transportation and the Federal Highway Administration recognized this opportunity, and in late

1972 they formed an advisory committee consisting of representatives from the Virginia Department of Highways and Transportation, the Virginia Highway and Transportation Research Council, the University of Virginia, and the Federal Highway Administration to plan the general scope of the research work to be conducted. Personnel from the University of Virginia and the Virginia Highway and Transportation Research Council were selected to conduct the investigation. Work on the research program was initiated in July 1973 and continued until completion of the interstate construction in the summer of 1976.

Objectives

The primary objective of the research reported here was to determine the response and behavior of both a flexible, corrugated steel culvert and a rigid, reinforced concrete culvert subjected to loads imposed by unusually deep fill. Since current design practices of such culverts are, to a large extent, empirical and based on previous field experience, a better understanding of the behavior of such culverts and their corresponding stresses and deformations under load will significantly aid in improving design procedures and, correspondingly, result in reduced construction costs.

The specific objectives of this study included (1) the observation, measurement, and recording of the gross deflections of points along the inside of both culvert pipes before, during, and after construction; (2) the rather extensive measurement of surface strains on both the inside and outside surfaces of the steel culvert; and (3) the measurement and recording of settlement of the earthen fill over and adjacent to the concrete culvert during and after the construction process. Such deflection measurements would permit a continual monitoring of the changes in cross section of the pipe while also providing an indirect approximate determination of the corresponding loadings imposed on the culvert. The surface strain readings, as measured from rectangular strain rosettes placed around the periphery of the pipe, would permit the direct determination of longitudinal and circumferential stresses as well as principal stresses and their directions. It was recognized that some difficulty in stress interpretation would likely result because of the nonuniform nature of the corrugated culvert. This settlement data would indicate the magnitudes of soil pressure that might be expected in the region of the concrete culvert, and would provide a comparison of field measurements with existing theories of soil behavior.

Because of time and budgetary considerations, it was not feasible to conduct as extensive a research study as would have been desired. For example, no data regarding soil pressures were obtained, no settlement data for the vicinity of the steel pipe were recorded, and no extensive strain measurements on the concrete culvert were made. Also, the massive size of the overall construction project, the priority of construction needs over research needs, the considerable distance between the construction site and the research team's office, as well as the numerous difficulties associated with any type of field construction, all contributed to making this study considerably more involved and complex than would have been the case for a similar, well controlled laboratory study. Nevertheless, it is felt that the scope of the project, the conduct of the research, and the extensive data collected provided extremely valuable information on the overall response of steel and concrete culverts to deep fill loadings, and should contribute significantly to the understanding of the response of such culverts and to subsequent improved design procedures.

CULVERT DESCRIPTION

Corrugated Steel Culvert

Specifications called for the temporary culvert pipe to be made of corrugated structural steel plates and to have a nominal diameter of 60 in (1.52 m) and a total length of 1,875 ft (572 m). The culvert was designed for a cover of 100 ft (30.5 m), had an elevation at the inlet invert of 2,098 ft (640 m) and an elevation at the invert outlet of 1,973 ft (601 m). The entire length of the culvert was to be bituminous coated, and specifications required that the bottom 25% of the culvert be fabricated from structural plate two gages heavier than that used on the sides and top. Although this culvert was selected to withstand a design fill of only 100 ft (30.5 m), it would ultimately be subjected to a fill depth of approximately 256 ft (78 m), and usage of this pipe was thus intended to be only temporary. In fact, the specifications indicated that the use of the steel culvert was to be discontinued when the permanent concrete culvert was in place.

The entire length of the steel culvert consisted of identical sections 12 ft (3.66 m) long. Each section was fabricated from four galvanized steel plates, each 12 ft (3.66 m) long in the longitudinal direction and having a dimension of approximately 52 3/4 in (1.34 m) in the flat measured circumferentially. These four plates were subsequently bolted together to form the cylindrical sections. In accordance with specifications, the culvert design resulted in a bottom plate section fabricated from No. 3 gage steel (approximately 1/4 in [.0064 m] thick) while the side and top plates were fabricated from No. 5 gage structural plate (approximately 7/32 in [.0056 m] thick). The top and bottom plate sections had an inner radius of 25 in (0.64 m), while the side plates had an inner radius of 35 3/4 in (0.91 m). The geometry was such that when the plates were bolted together the cross section would be approximately circular, with a nominal diameter of 60 in (1.52 m) but with the vertical axis elongated approximately 5%.

All plate corrugations had a 6-in (.15-m) pitch and a 2-in (.05-m) depth, and were formed such that the ridges and valleys of the corrugations ran circumferentially. A sketch of the plate cross section showing dimensions of the corrugation and section properties is provided in Figure 1.* The design properties of the steel sectional plate and the material properties are presented in Table 1. The sectional steel plates were provided by the Republic Steel Corporation, and the chemical analysis of these plates conformed to the base metal requirements of the following specifications: AASHO Designation M-167, AREA Chapter 1, Part 4, Bureau of Public Roads FT-61, and Federal Specifications WW-P-405A.

When field assembled, the 12-ft (3.66-m) plates were bolted together such that each plate was offset longitudinally from the adjacent plate by approximately 2 ft (0.6 m), resulting in a 10-ft (3-m) longitudinal lap seam which produced an alternating stagger in the circumferential joints. In accordance with specifications, sections were joined along the 10-ft (3-m) lap using 3/4-in (0.02-m) diameter, 1 1/2-in (0.04-m) long high strength, galvanized steel bolts tightened to a torque of 150 to 250 ft-lb (203 to 339 joules). A typical assembled section is shown in Figure 2. Circumferential lap joints were joined in a similar fashion.

All instrumentation, specifically the strain gages and mechanical deflection gages, was installed on a single 12-ft (3.66-m) section of the pipe. As provided in the specifications, the contractor delivered the unassembled plates and the test section was assembled and instrumented in the laboratories of the Civil Engineering Department at the University of Virginia.

* All figures and tables are appended.

The instrumented test section was transported by truck to the construction site and installed approximately 750 ft (229 m) from the outlet end of the culvert.

Reinforced Concrete Culvert

Specifications called for the permanent culvert to be a reinforced concrete pipe having a nominal inside diameter of 96 in (2.44 m) and a total length of 795 ft (242 m). This permanent culvert was designed for a cover of 163 ft (50 m) with an elevation at the invert inlet of 2,091 ft (637 m) and an elevation at the invert outlet of 2,079 ft (634 m).

The total length of the culvert was made up of identical 10-ft (3-m) sections with one end flared to permit coupling.

The culvert was designed in accordance with AASHTO Specification M 170-74 utilizing the design requirements for a Class V reinforced concrete pipe. The cylinder strength of the concrete was 6,000 psi (41.4×10^6 Pa) but the precise amount of reinforcement is unknown. The strength test requirement was modified to be slightly in excess of that required by the specification. For example, a standard Class V design requires that the D-load required to produce a .01-in (0.25-mm) crack be in excess of 3,000 psi/ft (67.9×10^6 Pa/m) of diameter and the D-load to produce ultimate load be not less than 3,750 psi/ft (84.8×10^6 Pa/m) of pipe diameter. In this case, the strength requirement was increased to require that the D-load for ultimate load be not less than 4,680 psi/ft (105.9×10^6 Pa/m) of diameter.

The test section on which all instrumentation was placed was one of the standard 10-ft (3-m) sections. Deflection gages were installed on the test section prior to delivery and placement. A limited number of strain gages were installed after placement of the culvert.

A photograph of typical sections of the reinforced concrete pipe is shown in Figure 3.

FIELD INSTALLATION

Site Description

The site for the installation of both the corrugated steel culvert pipe and the rigid, reinforced concrete culvert was on a stretch of Interstate 77 in Carroll County in southwest Virginia. A sketch of a portion of the county map is shown in Figure 4 with the circle identifying the approximate location of the two culverts. A more precise location of the culverts along the interstate route is indicated on Sheet No. 1 of the Department of Highways Plan and Profile Map of Interstate 77 extending from 2.587 mi (4.2 km) north of the Virginia-North Carolina state line to 0.265 mi (0.43 km) south of the intersection with the Blue Ridge Parkway.

The 60-in (1.5-m) structural plate pipe culvert was to be installed at station 303+50 SBL, while the 96-in (2.44-m) reinforced concrete pipe was to be installed at station 302+00 SBL, and in the embankment areas in the immediate vicinity of these highway locations. A detailed sketch of this particular location on Interstate 77 showing a clear view of the approximate layout of the culverts is shown in Figure 5.

A profile sketch of the terrain along the north-and southbound lanes is shown in Figure 6. As indicated on the sketch, the steel culvert was to be passed under the southbound lane at station 303+50 and the concrete culvert was to pass under the southbound lane at station 302+00. For information purposes, the original profile of the northbound lane as well as the proposed layouts of the north-and southbound lanes with the differences in elevation apparent are also shown in Figure 6.

As was indicated earlier, and as may be judged from Figure 6, the terrain in the vicinity of the steel culvert pipe was sufficiently severe to preclude installation of a pipe at that elevation which would be large enough to serve as a permanent culvert. In other words, a permanent culvert designed to support a fill of 256 ft (78 m) would have been so large and heavy that it could not be transported to and placed at that particular location. Accordingly, the concept of a temporary pipe to serve for the first 100 ft (30.5 m) of fill and a rigid permanent pipe to be placed at a higher elevation was finally decided upon and adopted.

The specifications called for 1,875 ft (572 m) of the 60-in (1.2-m) structural plate pipe to be placed such that the elevation at the invert inlet was 2,098 ft (640 m) and the elevation at the invert outlet was 1,973 ft (601 m). The test section of the pipe was designed to be located approximately 750 ft (229 m) from the outlet and directly under the southbound lane. The elevation of the pipe invert at that location was to be approximately 2,004 ft (611 m), with the elevation of the southbound lane at that location being approximately

2,260 ft (689 m). Thus, the fill height above the steel pipe invert was intended to be 256 ft (78 m).

Because of changes in grade and alignment of the steel culvert, some uncertainty existed as to the exact locations of the test section after placement had been completed. A final survey was conducted after the final grade elevations had been established and a line run back into the steel pipe to accurately locate the test section. The final profile of the 60-in (1.5-m) steel pipe is shown in Figure 7. Also shown on the sketch are the approximate locations and grade elevations of the north-and southbound lanes. A plan view of the steel pipe, also indicating the centerlines of the lanes, is presented in Figure 8. As may be observed from these figures, the actual location and elevation of the test section of the steel pipe were very close to the design figures. The actual elevation of the test section was 1,994 ft (608 m) but the final location of the test section was between the north-and southbound lanes, making the actual fill depth very close to that expected.

The alignment and location of the test section on the rigid, reinforced concrete culvert provided no problems since the pipe had a straight alignment and essentially constant grade. Specifications called for the length of the 96-in (2.44-m) concrete culvert to be 795 ft (242 m). It was to be placed such that the elevation at the inlet invert was 2,091 ft (637 m) and the elevation at the outlet invert was 2,079 ft (634 m). The test section was to be located under the southbound lane at station 302+00 and approximately 470 ft (143 m) from the outlet end of the pipe. The line of the concrete pipe as actually placed is shown in Figure 7.

Preparation and Placement of Pipe

The profile of the original grade along the north-and southbound lane centerlines is shown in Figure 6. The profile and foundation grade along the line for the steel pipe is shown in Figure 7.

Steel Pipe

Along the line of the steel pipe, the culvert bed was prepared by grading and then excavating to a depth of approximately 2 ft (0.6 m). In most areas this procedure required undercutting to solid rock, which resulted in essentially zero subsequent settlement for the steel culvert. After excavation for the bedding, the area was backfilled to the depth of the culvert with fine crushed stone. The bedding was then shaped to fit the contour of the invert pipe section by means of a contoured template. A photograph of the contoured bedding awaiting placement of the pipe is shown in Figure 9. The invert section of the pipe was placed starting at the downstream end. The photograph in Figure 10 shows a series of invert sections in place. Figures 11 and 12 show the test section being lifted in preparation for placement and then the test section in place and connected prior to backfilling. After the pipe section had been completely placed and assembled along the length of the bedding, fine silty sand backfill material was manually placed and compacted along the sides up to a depth of approximately 2 ft (0.6 m) above the crown of the culvert. The placement of the backfilling material is shown in Figure 13. The optimum moisture content and dry unit weight (AASHTO T-99) for the backfill soil were 15.1% and 116 lb/ft^3 (1789 kg/m^3), respectively. After fill had been placed up to 2 ft (0.6 m) above the crown, normal filling procedures, using shot rock and other cut material, were followed. Field density tests were not run on the 4-ft (1.2-m) layers of shot rock boulders which range up to 4 ft (1.2 m) in size.

Concrete Pipe

Because the concrete culvert was placed at a higher elevation than the steel pipe, the bedding preparation was somewhat different. As may be observed from Figure 6, the concrete culvert was to be placed approximately 100 ft (30.5 m) higher than the steel culvert and adjacent to a rock face on the south side of the ravine. Filling proceeded from the steel pipe in lifts of approximately 3 ft (1 m) until the fill elevation reached that corresponding to approximately 2 ft (0.6 m) above the crown elevation of the concrete culvert. A trench was then excavated to receive the culvert pipe. The original fill, composed of a silty sand with gravel, was well compacted in the vicinity of the pipe's future position prior to excavation. The depth of excavation was 1 ft (0.3 m) below the elevation of the pipe invert to permit placement of bedding material. This depth was then filled to the level of the invert using fine crushed stone. The bedding material was shaped to fit the contour of the invert pipe section by means of a contoured template. Figure 14 shows the template used and the contoured bedding prepared for the next pipe section. Sections were placed starting at the downstream end.

Backfilling around the culvert commenced as soon as practicable after each section had been placed. As with the steel culvert, backfilling material consisted of fine silty sand, which was placed by a gradall and manually compacted. This backfilling process is clearly shown in Figure 15. When the fill had reached the level of the culvert crown, approximately 19 ft (5.5 m) of baled straw were placed immediately above the concrete pipe along its entire length. Sufficient backfilling to keep the straw in place was performed, and normal backfilling procedures then resumed. Figure 15 shows the baled straw being placed above the pipe. As noted earlier, much of the fill above the pipe consisted of shot rock with a maximum size of 4 ft (1.2 m) placed in 4-ft (1.2-m) layers. Field density tests on this shot rock were not obtained.

The optimum moisture content and dry unit weight (AASHTO T-99) for the backfill soil were 15.1% and 111.7 lb/ft^3 (1789 kg/m^3), respectively.

INSTRUMENTATION

Because of time and budgetary constraints, the instrumentation installed on the two culverts for this project was not as extensive as would have been ideal. Nevertheless, adequate instrumentation was provided on the test section of the steel culvert to yield strain data on both the inside and outside surfaces of the culvert and to provide information defining the deformation of various points on the cross section. However, no instrumentation was provided which would yield any information on soil pressures or settlement in the vicinity of the steel culvert. Deformation measurements were also made on the reinforced concrete culvert and, in addition, extensive settlement data in the vicinity of the concrete pipe were collected. This section describes the three types of instrumentation employed.

Strain Gages

Extensive strain gage instrumentation was placed only on the steel culvert. All gages were installed on a single 12-ft (3.6-m) culvert section provided by the contractor and delivered in sections to the Civil Engineering Laboratory at the University of Virginia. The four plate sections were bolted together and assembled to form a single 12-ft (3.6-m) long circular section. The method and procedure for assembly were identical to those specified for field assembly. The test section provided by the contractor was not bituminous coated but was galvanized structural steel plate. A photograph of the test section during assembly, but before instrumentation, is shown in Figure 16.

All gages used for active and dummy gages were 45° foil strain gage rosettes, type WK-06-250 RD-10C, with a resistance of 1,000 ± .4% ohms. The strain rosettes were located at three circumferential locations approximately 2 ft (0.6 m) apart on the test section, with the three circumferential locations being symmetrically spaced with respect to the center of the test section. Along each circumferential location or ring, gage locations were spaced at approximately 45° intervals starting from the top, which provided eight gage locations on each circumferential instrumentation ring.

At each gage location, 4 strain rosettes were placed to provide strain measurements on both the corrugated ridges and valley locations on the inside and outside pipe surfaces. Thus, the instrumentation plan for the strain gages called for placing 32 strain rosettes circumferentially along the inside and outside of the pipe surface at three identical longitudinal locations. These three identical rings of strain gaging provided for 96 individual strain rosettes to be placed at all three locations. This resulted in 288 strain gages along the inside and outside of the pipe, corresponding to 96 rosettes. However, because of wiring and switching constraints, it was necessary to eliminate 2 rosettes from each ring in order to reduce the total number of gages from 288 to 270. Thus, as finally installed each circumferential ring consisted of 30 strain rosettes (90 individual gages) with a vacant rosette location staggered such that at least 2 rosettes were located along each longitudinal line. One purpose for installing three identical rings of instrumentation was to provide for duplication of data in the event that some gages might become inoperative.

For convenience and designation, the three cross sections at which strain rosettes were placed were designated as rings A, B, and C, beginning with the downstream location. As noted above, instrumentation on each circumferential ring was identical with the exception of periodic vacancies at rosette locations to provide the required maximum number of gages. The locations of the rosette gages on the test section, including their number designations and relative locations on the ridges and valleys, are indicated in Figure 17. Along each circumferential ring, rosettes were numbered 1 through 30 consecutively, with numbers 1, 5, 9, etc. being located on the outside ridge and rosettes numbered 2, 6, 10, being mirror images on the inside valley. Rosettes numbered 3, 7, 11, etc. were located in the outside valley immediately adjacent to the outside ridge containing the other rosettes, and rosettes numbered 4, 8, 12, etc. were mirror images located on the inside ridge. Thus, along a single circumferential ring, at each of three longitudinal locations, half of the rosettes were located at a single cross sectional plane corresponding to an outside ridge, while the remaining gages were located at a plane corresponding to the adjacent outside valley.

As may be seen from the sketch in Figure 17, it was necessary to vary the 45° circumferential gage interval slightly to avoid the longitudinal joints where the pipe sections were bolted together.

For convenience and designation, and for reference in subsequent sections of this report where results are presented and discussed, gage designations of each rosette are defined as follows. The gage of the strain rosette oriented circumferentially is designated as Gage X, the element located longitudinally is designated Gage Y, and the 45° angle gage is designated as Gage Z on each of the 90 strain rosettes. Accordingly, a particular strain reading can be identified by locating the circumferential ring (either Ring A, B, or C), a particular rosette number within that ring (e.g., rosettes 1 through 30) and within a rosette either Gage X, Y or Z. For example, Gage A1X identifies a circumferential strain gage of rosette No. 1 on Ring A, which means that this particular gage was located at the top of the pipe on the outside ridge of the pipe surface. Other gage designations can be located and identified in a similar fashion.

As noted earlier, physical constraints imposed by wiring and switchbox arrangements necessitated omitting two rosettes from each ring in order to have a total of only 30 rosettes per ring, or 90 rosettes total. Those eliminated, in order to provide maximum longitudinal duplication, were rosettes A2, A6, B5, B7, C4, and C8. Thus, as finally wired, each circumferential ring had 30 rosettes (90 gages) for a total of 270 active gages on the test section.

In the recording of strain data, a half bridge circuit was used which utilized one active strain gage placed on the culvert and one inactive or dummy gage on an unstrained culvert specimen to provide for temperature compensation. Eight rosette gages were used to serve as dummy gages in the circuitry. These dummy rosettes were identical to the active rosette gages and were placed on 8 small specimens of the pipe culvert and enclosed in protective enclosures both inside and outside the pipe. The dummy gages were labeled D1X, D1Y, ..., D8Z consistent with the numbering and designation of the active gages. These enclosures were made from standard plumbing metal pipe couplings 3 in (7.6 cm) in diameter and approximately 3 1/4 in (8.3 cm) long with standard plugs screwed into each end. A photograph of a typical enclosure is shown in Figure 18. The design and placement of these dummy gages within the enclosures were such as to preclude any straining of the material on which the gages were placed, but permitted the dummy gages to be placed in almost the identical locations as the active gages so that moisture and temperature environments for both types of gages would be essentially the same. Four of these dummy enclosures were mounted outside the culvert test section in a corrugated valley and four were located inside in a valley, all in the vicinity of the center or the B circumferential gage ring. The 8 dummy enclosures were mounted at 0°, 135°, 180°, and 270° positions circumferentially using one bolt through a wall of the enclosure into the pipe culvert. Those on the outside were fastened by hollow bolts to the culvert through which the dummy lead wires were passed to the inside.

In accordance with instructions provided by the strain gage manufacturer, the galvanizing was removed from locations on the test section where each rosette was to be placed. Grinding of the galvanizing and preparation of the surface for gage bonding were difficult and time-consuming tasks, and care had to be taken once the galvanizing was removed to prevent rusting on the bare metal prior to gage placement. After proper surface preparation the gages were bonded to the steel surface with M-Bond 600 and coated with a waterproofing compound. In addition to the waterproofing, each rosette was covered with a sealing compound pad to provide further environmental protection. For convenience in wiring, terminal strips were bonded to the metal surface in the region of each rosette. Two wires coming from each gage on the rosette resulted in a 6-wire rosette cable. This rosette cable size was reduced from a 6-conductor cable to a 4-conductor cable by making 1 lead wire of each rosette electrically common, thus providing a single common lead from each rosette. These 4 lead wires from each rosette were attached and soldered to rosette terminal strips bonded to the culvert wall. Leads from a 12-ft (3.66-m) 4-conductor cable were then soldered to each terminal strip and led downstream to the open end of the test section. These conductor cables were grouped together to facilitate placement and anchoring to the culvert surface, as well as to facilitate waterproofing of the entire cable length.

Gages located on the outside surface were wired in a similar manner, and the lead cables were brought inside the culvert section through a drilled hole near the top of the culvert pipe near the downstream end of the test section.

Certain sections of the cables were encapsulated in urethane compound to waterproof and secure the cables in place. At the downstream end of the test section, all rosette leads were assembled together into a single 98-cable (4 conductors per cable) wiring package.

Photographs showing typical gage installations and wiring and the completed instrumented culvert test section are shown in Figures 2, 19, and 20.

Upon completion of the instrumentation and wiring in August 1973, the test section was transported by truck to the construction site. The 12-ft (3.66-m) test section was placed approximately 750 ft (229 m) from the outlet end of the culvert. The culvert bed for the test section was prepared in the same manner as for the remainder of the culvert and has been described in a previous section. Photographs of the test section during installation are shown in Figures 11, 12, and 21.

All active gage and dummy gage lead wires were carried out of the pipe using a 1,200-ft (366-m), 400-conductor, shielded and waterproofed underground cable (Figure 22). This cable was obtained from the Virginia Telephone and Telegraph Company of Charlottesville, Virginia. Its length was sufficient to carry the instrumentation lead wires through the 750 ft (229 m) of culvert remaining to the downstream end, plus the additional distance required underground until the cable was finally led into an instrumentation shack where final strain readings were obtained. To facilitate installation of this cable, a top plate was left off of one 12-ft (3.66-m) culvert section near the test section, and the telephone cable was pulled through a 3-in (7.6-cm) plastic conduit which had been installed by the contractor along the inside crown of the downstream portion of the culvert. This 3-in (7.6-cm) conduit also served to carry a 115-volt circuit and 40-watt bulb outlets at 75-ft (23-m) intervals as provided in the specifications. From the outlet end of the culvert, the instrumentation cable was led underground approximately 200 ft (61 m) to the instrumentation shack.

Technicians from the Virginia Telephone and Telegraph Company were employed to install the telephone cable and to make all of the splices between the rosette cables and the telephone cable connections at the downstream end of the test section. The splices were made using 2-wire, scotch lock, U-series communication connectors packaged and encapsulated in urethane after completion.

As noted earlier, a half bridge circuit was used to connect the strain gages to the readout units and, within this bridge, one active strain gage on the culvert and one inactive or dummy gage on the unstrained culvert served as two arms of the bridge. The two gages were located sufficiently close together to nullify any differential effects of temperature or other environmental considerations. As finally connected to the readout units, the lead wires for each gage were approximately 1,200 ft (366 m) of AWG26 wire with a loop resistance of approximately 100 ohms at a temperature of 20° Celsius. Thus active gages with a gage resistance of 1,000 ohms were chosen to minimize the effect of this lead wire resistance. Corrections for the lead wire resistance could easily be incorporated into the circuitry to provide corrected strain readings. A Vishay/Ellis 20AL digital strain indicator was selected for the strain readout unit. The 115-volt 60-hz power supply was obtained from a small gasoline engine driven generator.

The V/E-20 strain indicator by itself is designed to accommodate only one input channel. Accordingly, a multiple channel data acquisition system was designed to permit all 270 active strain gages to be connected and read one at a time from the single strain indicator. The switching console, as finally designed, utilized nine single Vishay switch and balance units (SB-1), each of which could accept ten channels of strain information input. Thus the nine switchboxes permitted input of 90 active channels. An auxiliary switching panel was subsequently added to allow switching between circumferential rings A, B, and C. Thus with the switching panel set to any particular ring, it was possible to read all 90 strain gages from that ring using the switching console described. A photograph of this switching console is shown in Figure 23. The 8 dummy gages were wired directly to the nine SB-1 switchboxes and thus required no switching between the three rings. All of the switching gear was assembled in the laboratory in a 5-ft x 3-ft (1.5-m x 0.9-m) cabinet with appropriate devices to facilitate wiring attached to the cabinet mountings. When finally assembled, input from all of the active and dummy strain gages, provided through the telephone connection leads, were led through the floor of the test shack and wired into the switchbox console.

Extensive preloading checkout was conducted on the gages to ascertain gage resistance, lead wire resistance, any leakage through attachment to the metal, and any bad gages or switching. This particular wiring and switching configuration had to be extremely convenient in order to provide a complete set of strain readings at any particular stage in the loading process. Details concerning the strain reading obtained, the manner in which these data were processed, and resulting information gleaned from the strain gage readings are provided in a subsequent section of this report.

Deformation Measurements

Deformation gages were installed and extensive deformation measurements recorded on both the steel and concrete culverts. Since the methods of installation and the recording and analysis procedures were the same for both culverts, the descriptions provided in this section apply to both pipes.

Gross deformation measurements of the cross section shape of the pipe culverts, both before and during loading, were made from chord measurements made along the internal circumference of the pipes at two longitudinal locations on each of the test sections. At each transverse location, 12 gage points were located equally spaced along the interior circumference, starting at the top, which resulted in approximately equal angular intervals of 30° between gage points. At each mechanical gage point location, a small hexagonal nut with a 1/16-in (1.6-mm) diameter hole drilled in one face was rigidly attached to the interior wall of the culvert. In the steel culvert, small holes were drilled and threaded and the hex nut was screwed into the wall. In the concrete culvert, a small threaded insert was injected into the wall, and the measuring nut screwed in.

Each of these small "acorn" nuts was placed in position and tightened such that the 1/16-in (1.6-mm) diameter holes were all aligned with their axes parallel to the transverse axis of the culvert. These holes then served as the reference point for subsequent measurement of the chord lengths. Deformation measurements were then made by using mechanical extensometers to measure the chord distances between various gage locations. To obtain these chord readings, it was necessary to use three mechanical extensometers made of two sections of telescoping rod to permit slight adjustments for fitting between the various gage point locations. A relative reading of deformation was then taken between two fixed points located on either side of the telescoping section. This relative reading, when added to a constant representing the closed length of the extensometer, provided an accurate measure of the chord distance, or distance between holes in the "acorn" nuts placed at each gage location, to within .001 in (.025 mm). Figure 24 shows an extensometer with the calipers used in making the measurements.

A total of 21 chord measurements were made at each of the transverse sections each time data readings were obtained. Chord measurements included the measuring of long chords between gage points 120° apart, which resulted in 3 long chord readings. This triangularization thus permitted the location of two points on the interior wall of the culvert relative to the reference point, which was taken to be the crown point, and a relative orientation. A subsequent series of readings included the measurement of chord distances between gage points 60° apart, again starting at the crown, for a total of 6 medium gage length readings. A final series of chord measurements were taken between each successive pair of gage locations. This total set of 21 readings thus permitted an accurate determination of the precise location of each gage point on the interior wall of the culvert cross section at any stage of the loading relative to a reference point and a reference axis.

For this study, the reference point was selected to be the gage point at the invert of the culvert, and the reference axis was selected to be the vertical axis between the mechanical gage point at the bottom invert and the mechanical gage point at the crown of the pipe culvert. Thus, it was assumed that if the pipe were placed and aligned such that the vertical axis was in fact a true vertical, this axis remained vertical during subsequent loading. Plan and cross section views of the mechanical gage locations are shown in Figures 25 and 26 for the steel culvert. The locations and numbering are similar to those for the concrete culvert.

Details concerning the reduction of these data to provide coordinate locations for each gage point are provided in a subsequent section.

It should be noted that since all of the deformation readings were made from the interior using a mechanical extensometer, it was necessary for the individuals recording the measurements to physically be inside the pipe. During the final stage of fill, above the design height, deformation readings were suspended on the steel culvert because of the possibility of some local failure in the culvert wall. After filling had been completed, final readings were taken and observations of distress were made. For reasons of safety, minimal time was spent in the culvert on these readings and observations.

No similar problems were encountered on deformation recordings on the concrete culvert due to the lesser height of the fill.

Settlement Readings

To determine the magnitude, rate, and nature of soil movements over and adjacent to the concrete pipe, a series of settlement plates were placed in the embankment as filling over the pipe proceeded. The vertical movement of these plates was periodically monitored with an Idel Sonde unit.

The Idel Sonde system employs thin, flat metal plates approximately 16 in (40.6 cm) in diameter with a hole approximately 3 1/2 in (8.9 cm) in the center. These plates were installed at selected depths in the embankment, as the filling proceeded, by placing them in four vertical arrays each centered about a hollow plastic tube which extended upward through the 3 1/2-in (8.9-cm) center hole as shown in Figure 27. The initial elevations of these plates were referenced to established benchmarks. Each plate in a given vertical array could move freely with respect to the vertical column of plastic tubing, and the tube itself could also move vertically with the fill, since each joint of tubing was placed with a 6-in (15.2-m) gap between it and the adjacent section. This gap was bridged with a plastic sleeve and rubber boots which connected and covered a short section of the outside of both tubes and permitted free vertical sliding movement of the tubes. The plastic tubing, connecting sleeve, and rubber boots are shown in Figure 28. Thus both the tube and surrounding settlement plates could move freely with the fill in a vertical direction. The vertical position of each plate was monitored periodically by lowering a small radio frequency transmitter on a metered cable into each tube. The position of this transmitter in the tube was known at all times, and by monitoring the signal from the transmitter with a receiver at the ground surface, the location of each plate along the plastic tube could be pinpointed.

The four plastic tube columns, with associated settlement plates spaced approximately 10-ft (3-m) apart vertically, were located above the centerline and 12, 24 and 36 ft (3.7, 7.3, and 11.0 m) from the centerline of the culvert as shown in Figures 29 and 30.

The 96-in (2.44-m) diameter precast concrete pipe was not installed until approximately 80 ft (24.4 m) of fill had been placed in the ravine below. The position of the concrete culvert was so chosen that it hugged one side of the ravine and rested on or only slightly above hard bedrock exposed in the steep-sided ravine. In places, rock was blasted and excavated to make room for the pipe. Once the future bedding surface was free of rock, selected soil consisting of silty sand with some gravel was compacted in layers to an elevation approximately equal to the planned position of the top of the pipe. A trench to receive the pipe was then excavated into this fill. Sections of the pipe were then placed and bedded in the trench as shown in Figure 15. Backfilling around and over the pipe with selected silty sand with gravel was quickly completed. Baled straw was placed over the pipe in layers 96-in (2.44-m) wide to a depth of 19-ft (5.8-m) at the test section as backfill soil was placed alongside the straw.

Material for the fill beyond the immediate vicinity of the pipe consisted of shot rock grading from boulders 3-ft (0.9-m) in diameter to gravel with little sand. The shot rock was placed in layers approximately 4 ft (1.2 m) in thickness, as shown in Figure 31, while the silty sand with gravel was compacted in 8-in to 12-in (20-cm to 30-cm) layers. The relative amount of shot rock or silty sand used in the fill above and adjacent to the concrete pipe was governed by the material, rock or soil, then being excavated in nearby cuts. Much of the fill, however, appeared to be of shot rock with an occasional layer or two of the silty sand sandwiched between the thick layers of rather porous shot rock.

DATA COLLECTION AND ANALYSIS

Strain Data

As soon as the installation of the corrugated steel culvert was completed and initial backfilling to the crown had been accomplished, the stream was immediately diverted to run through the culvert. This resulted in a condition of approximately 100% humidity inside the culvert. In addition, normal ground water was sufficient to ensure that the exterior of the pipe was also continuously moist. Thus, from the point of installation, essentially all strain gages were operating in an environment of extreme moisture and this condition resulted in significant degradation of the strain readings as the project progressed.

Initial zero strain readings indicated that essentially all of the gages were operative. By the end of the project, when the fill had progressed to well in excess of 100 ft (30.5 m), the majority of the gages had either ceased to function or were providing readings so erratic as to be judged unreliable. The probable cause for these malfunctions could very likely be the moisture infiltrating the gage area. Whether any steps could have been taken to alleviate this situation is doubtful.

Nevertheless, in spite of the large number of gages that became inoperative or unreliable, a significant amount of reliable strain data were obtained during the first 45-ft to 60-ft (15-m to 20-m) of fill. This height of fill would seem to be the most crucial range for strain data, since beyond this level it is likely that the response of the corrugated steel culvert became nonlinear because of local yielding in certain areas of the corrugations. Accordingly, the bulk of the data presented and discussed in this section will be limited to those judged to be accurate and reliable and in general will be for heights of fill less than 100 ft (30.5 m).

When initial attempts were made to obtain the zero reference readings on the strain gages, it was discovered that the half-bridge input circuit was sufficiently unbalanced so that the controls on the V/E-20 strain indicator did not have sufficient range to compensate and produce an initial reading of zero on the strain indicator. Although all gages were rated 1,000 ohms $\pm$ 0.4%, the difference between the active and dummy gage circuits as measured by a Wheatstone bridge amounted to 40 to 50 ohms. This discrepancy was too large to be accounted for by the slight difference in cable length. Since the active and dummy gages were from different lots, apparently the gage resistance variation between lots considerably exceeded the manufacturer's specifications of $\pm$ 4 ohms. Although it would have been possible to place precision resistors in the circuit parallel to one bridge arm to produce a zero reading, it was determined that this would only unduly complicate the data collecting procedure. Accordingly, this particular problem was resolved by modifying the calibration reading set on the strain indicator such that all strain readings would have a constant multiplying factor of approximately 9.86.

An attempt was made to obtain a complete set of strain readings at approximately each 10 ft (3 m) of fill depth. As noted earlier, all strain data were provided by 45° rectangular strain rosettes mounted circumferentially around the pipe at three transverse locations in the test section. Along with each strain reading, a recording of the internal calibration reading of the instrument was noted. Since the correction factor used for all of the data was dependent on the calibration readings, this double recording provided a continuous check to ensure that the correction factor used was still valid and appropriate.

Strains were recorded in horizontal, vertical, and 45° directions. Utilizing these three values of strain, it was possible to determine not only the strains in the horizontal and vertical directions but also the shearing strain, stresses in the circumferential and longitudinal directions, and principal strains and directions.

The analysis of the strain data was conducted in the following manner. The sketch in Figure 32 indicates the typical orientation for a 45^{o} rectangular strain rosette. In this study, the x-axis (gage A) on the sketch was oriented longitudinally on the exterior pipe surface with the positive direction pointing upstream. The y-axis (gage C) was oriented circumferentially with the positive direction coinciding with the counterclockwise direction looking upstream. With reference to earlier gage designations used in recording data, gage A in this section corresponds to gage Y, gage B corresponds to gage Z, and gage C corresponds to X. This correspondence may be observed by comparing the sketches in Figures 17 and 32.

In terms of the rosette gage strains, the following relations hold between gage strains and rectangular strains (refer to Figure 32).

$$\varepsilon_x = \varepsilon_A \, , \qquad \varepsilon_y = \varepsilon_B \, , \text{ and}$$

$$\gamma_{xy} = 2\varepsilon_B - \varepsilon_A - \varepsilon_C \, , \tag{1}$$

where ε_x and ε_y are normal strains and γ_{xy} is the shear strain.

Rectangular stresses in terms of gage strains are given by the familiar expressions

$$\sigma_x = \frac{E}{1-\nu^2} (\varepsilon_A + \varepsilon_C) \, ,$$

$$\sigma_y = \frac{E}{1-\nu^2} (\varepsilon_C + \nu\varepsilon_A), \text{ and} \tag{2}$$

$$\tau_{xy} = G\gamma_{xy} = \frac{E}{2(1+\nu)} (2\varepsilon_B - \varepsilon_A - \varepsilon_C),$$

where σ_x and σ_y are normal stresses, τ_{xy} is the shearing stress, and E and ν are the modulus of elasticity and Poisson's ratio, respectively.

Principal strains may be expressed as

$$\varepsilon_1 = \frac{1}{2} (\varepsilon_A + \varepsilon_C) + \frac{1}{2} \; [(\varepsilon_A - \varepsilon_C)^2 + (2\varepsilon_B - \varepsilon_A - \varepsilon_C)^2] \text{ and}$$

$$\varepsilon_2 = \frac{1}{2} (\varepsilon_A + \varepsilon_C) - \frac{1}{2} \; [(\varepsilon_A - \varepsilon_C)^2 + (2\varepsilon_B - \varepsilon_A - \varepsilon_C)^2], \tag{3}$$

and the angle ϕ between the x-axis and the direction of principal strain is given by

$$\tan\phi = \frac{2\varepsilon_B - \varepsilon_A - \varepsilon_C}{\varepsilon_A - \varepsilon_C}.$$

Finally, the principal stresses are determined from Eq. (3) as

$$\sigma_1 = E\left[\frac{\varepsilon_A + \varepsilon_C}{2(1-\nu)} + \frac{1}{2(1+\nu)}\left((\varepsilon_A - \varepsilon_C)^2 + (2\varepsilon_B - \varepsilon_A - \varepsilon_C)^2\right)\right] \text{ and}$$

$$\sigma_2 = E\left[\frac{\varepsilon_A + \varepsilon_C}{2(1-\nu)} - \frac{1}{2(1+\nu)}\left((\varepsilon_A - \varepsilon_C)^2 + (2\varepsilon_B - \varepsilon_A - \varepsilon_C)^2\right)\right]. \quad (4)$$

Use of Eqq. (2) and (4) permitted the calculation of local circumferential, longitudinal and shear stresses, and principal stresses at a particular location using the strains obtained from a rosette at that location. It should be noted that the calculation of circumferential and longitudinal stresses required a knowledge of only circumferential and longitudinal strains, while the determination of the shear stress and principal stresses required a knowledge of all three strains as measured by a rosette gage. For this reason, considerable difficulty was encountered in stress calculations since the failure of a single gage could essentially render useless the data from an entire rosette.

Computer programs were written to calculate rectangular and principal stresses and strains as given in Eqq. (1) through (4) using the surface strains as measured by the strain rosettes. The stress data were subsequently utilized to calculate bending moments circumferentially along the pipe.

Deflection Data

Deflection data were obtained from both the steel and concrete pipes, and the method of measurement and the reduction of the data were the same for both culverts. For the deflection data, measurements were made of the chord distances between gage points equally spaced along the internal circumference of the pipe culverts. These mechanical gage points were placed at two transverse locations in each test section.

The distance between the gage points, or the chord length, was determined using a mechanical extensometer which could be read to within .001 in (.025 mm). Three chord lengths were measured: one between gage points at 120° intervals, one set between gage points at 60° intervals, and the final, shortest distance between gage points located at 30° intervals around the interior circumference. A description of the procedure used to determine coordinate locations of the points on the interior of the pipe where the gage points were placed is provided in the following section.

A sketch of a typical cross section indicating the approximate location and layout of the mechanical deflection gages is shown in Figure 33, where each of the gage locations and chord distances have been identified for reference. The procedure for measurement was as follows. First, using the longest gage length extensometer, chord distances between gage points 1 and 2, gage points 2 and 3, and gage points 3 and 1 were measured. At this point it was assumed, for reference purposes, that gage point 1 was the local reference origin and that line 1-2 joining gage points 1 and 2 defined the location of the local x-axis. This, in turn, was sufficient to locate the direction of the y-axis and, with the chord distances obtained, local coordinates with respect to this coordinate system could be established for each gage point. Since the chord distances defined the lengths of all three sides of a triangle, the associated angles could be easily determined. Thus gage point 1 had local coordinates of (0,0), gage point 2 had an x' coordinate equal to the chord length A2 between gage points 1 and 2 and a y' coordinate of 0, and the coordinates of gage point 3 could be determined as

$$x'(3) = A_3 \cos (A_2,A_3) \text{ and}$$

$$y'(3) = A_3 \sin (A_2,A_3) ,$$

where (A_2,A_3) is the angle between chord A_2 and chord A_3.

Using the intermediate gage length extensometer, chord distances were measured between gage points 3 and 4, 4 and 2, 2 and 5, 5 and 1, 1 and 6, and 6 and 3 and these distances are defined as B_1 through B_6, respectively. Again using the same origin and coordinate system established previously and knowing the locations of gage points 1, 2 and 3 in terms of their local coordinates, the locations of gage points 4, 5 and 6 could be determined as below.

<u>Pt. 4</u>

$$x'_4 = x'_3 + B_1 \cos [(A_1,A_2) - (A_1,B_1)]$$

$$y'_4 = y'_3 - B_1 \sin [(A_1,A_2) - (A_1,B_1)] .$$

Pt. 5 $$x_5' = B_4 \cos (A_2,B_4) \text{ and}$$

$$y_5' = -B_4 \sin (A_2,B_4)$$

Pt. 6 $$x_6' = x_3' - B_6 \cos [(A_2,A_3) - (A_3,B_6)]$$

$$y_6' = y_3' - B_6 \sin [(A_2,A_3) - (A_3,B_6)]$$

Finally, using the smallest gage length extensometer, the remaining 12 chord lengths were measured and designated C_1 through C_{12}. The coordinates of gage points 7, 8, 9, 10, 11, and 12 were determined in a similar manner and are given by the following relationships:

Pt. 7 $$x_7' = x_3 + C_1 \cos [(A_1,A_2) - (A_1,C_1)]$$

$$y_7' = y_3 - C_1 \sin [(A_1,A_2) - (A_1,C_1)]$$

Pt. 8 $$x_8' = x_2 + C_4 \cos [180^\circ - (A_2,C_4)]$$

$$y_8' = C_4 \sin [180^\circ - (A_2,C_4)]$$

Pt. 9 $$x_9' = x_2 - C_5 \cos (A_2,C_5)$$

$$y_9' = - C_5 \sin (A_2,C_5)$$

Pt. 10 $$x_{10}' = C_8 \cos (A_2,C_8)$$

$$y_{10}' = -C_8 \sin (A_2,C_8)$$

Pt. 11 $$x_{11}' = C_9 \cos (A_2,C_9)$$

$$y_{11}' = C_9 \sin (A_2,C_9)$$

Pt. 12 $$x_{12}' = x_3' - C_{12} \cos [(A_2,A_3) - (A_3,C_{12})]$$

$$y_{12}' = y_3' - C_{12} \sin [(A_2,A_3) - (A_3,C_{12})]$$

The procedure outlined above was used for determining gage point local coordinates at both transverse stations on the culverts where gage points were located. At this stage of the data reduction, each of 12 gage locations at each station on the interior pipe circumference had been located with respect to an arbitrary origin and coordinate reference system. It was next necessary to redefine these gage point locations in terms of the final origin and coordinate reference system to be used in the analysis. For this procedure, it was decided to use the gage point location at the pipe invert as the origin and to define a vertical line between the invert and the crown, e.g. a line between gage points 5 and 3, as defining the y-axis of the final reference coordinate system. Final coordinates at the gage points could thus be obtained by a simple transformation from one coordinate system to another. Details of this transformation are provided below.

First, a translation transformation is performed to translate the origin to gage point 5 on the pipe invert. If primes denote the local coordinate system with origin at point 1, and bars denote the same coordinate system translated to the new origin at point 5, the relationships between coordinates are given by

$$\begin{Bmatrix} \bar{x} \\ \bar{y} \end{Bmatrix}_i = \begin{Bmatrix} x' \\ y' \end{Bmatrix}_i - \begin{Bmatrix} x' \\ y' \end{Bmatrix}_5 .$$

The final transformation then rotated the coordinate axes such that the y-axis was coincident with a line joining gage points 5 and 3. This transformation, for gage point i, was achieved as follows.

$$\begin{Bmatrix} x \\ y \end{Bmatrix}_i = \begin{bmatrix} \cos\theta & -\sin\theta \\ \sin\theta & \cos\theta \end{bmatrix} \begin{Bmatrix} \bar{x} \\ \bar{y} \end{Bmatrix}_i ,$$

where θ is the angle between the $\bar{y}$-axis and the line joining points 3 and 5. It is defined by

$$\theta = \arctan (\bar{x}_3/\bar{y}_3) .$$

At this point, all coordinates corresponding to each gage location were defined.

By utilizing the procedure just described on the initial chord measurements which defined the undeformed cross section of the pipe, it was possible to establish a zero reference for the cross sectional configuration. Subsequent readings which would establish deformed cross sections corresponding to various levels of fill could then be measured relative to this initial reference.

Radial deformations of each gage location were determined as follows. First, it was assumed that each of the culverts would retain an essentially circular shape even for fairly large values of fill. Accordingly, the center of the culvert was treated as the reference center and deformation gage points, which were located on essentially 30° radial lines measured from this center (30 in or 48 in [76 cm or 122 cm]) above the invert, were assumed to remain on that radial line during deformation. This assumption was certainly valid for essentially all deformation data recorded because relatively little tangential displacement was observed. From the radius of the reference circle, the coordinates of the initial and final gage points and the angular inclination with respect to the reference axes, the corresponding radial deflection could easily be computed. This radial deformation was one category of data determined from the mechanical gage measurements.

While radial displacements, particularly at the crown, provided valuable information with regard to gross culvert deformation, additional useful indicators of culvert response could be best obtained from some type of representation of the overall deformation of the cross section. Such a representation could best be displayed visually in the form of sketches of the cross section in which the deformed cross sectional shape is compared with the initial undeformed cross sectional shape. Since the volume of data precluded manual plotting, consideration was given to methods for generating computer plots. Accordingly, this approach required the modification of existing deformation data in terms of chord readings to obtain deformation data that could be used as part of an automatic plotting routine.

The following procedure was adopted as a means of generating such deformation data. First, the radial deformation, e.g. the difference between the final position of a gage point at a particular level of fill and the initial unloaded position of the same gage point, was identified as an offset. With the assumption that a perfect circle could reasonably well define the initial positions of the gage points, at least as far as radial directions were concerned, these offsets were then treated and calculated as vertical offsets on an x-y plot by effectively unrolling the cross section starting from the invert. Thus the invert became the origin of the coordinate system, the crown of the undeformed culvert became a point on the x-axis, and the invert again defined the far end of the unrolled culvert on the x-axis. This procedure permitted the offsets to then be plotted as either positive or negative displacements in the y direction. Various curve fitting techniques were then applied to determine a best fit curve that would pass through these displaced points on the culvert.

It was determined that reasonable results could be obtained by a simple polynominal approximation to represent the cross section shape. A computer program was developed which would determine the coefficients for this polynominal using the chord. Once this was done, offsets or ordinates to the deformed curve in the flattened x-y coordinate system could be determined for any distance along the x-axis which would correspond to the circumferential distance around the inside of the undeformed culvert. These points could then be transformed back, using a standard coordinate transformation, to the original culvert shape, and each point could then be easily input to a plotting routine permitting the generation of smooth automated plots. These plots could be generated to any desired scale and would clearly depict the deformed cross section of the loaded culvert.

From the standpoint of providing information related to design, and related to an indirect determination of culvert loads, the most important experimental data would seem to be stress information. Such information relating to circumferential stresses could be obtained from a knowledge of the circumferential bending moments. Thus, considerable effort was expended in developing and evaluating various procedures for determining experimental bending moments in the pipe culvert. As noted earlier, strain data for the steel pipe were sufficiently scattered and unreliable to prevent any utilization of strains to determine bending moments. However, the deformation data on both the steel and concrete culverts were extremely reliable and consistent and were felt to be the best source of information for the determination of circumferential bending moments.

Circumferential bending moments were calculated using the relationship

$$\frac{d^2w}{ds^2} + \frac{w}{R^2} = \frac{M}{EI} ,$$

where w = radial displacement, s = distance measured along the interior circumference, R = interior pipe radius, M = bending moment, E = modulus of elasticity, and I = moment of inertia.

The stiffness parameters for the corrugated steel culvert were known, but similar characteristics for the concrete pipe could not be ascertained. Hence, for consistency, the non-dimensional moment term M/EI was calculated for both pipes.

The radial displacement, w, was determined from chord measurements as described previously. The second derivative term was calculated by two methods. The first procedure involved analytically differentiating the polynomial approximation to the deformed shape. The second method involved numerical differentiation to calculate the second derivative. This latter process was accomplished using both third order and fifth order central difference approximations applied at each gage location. The derivatives of the polynomial representation could be calculated at any point in the cross section but the difference approximations for the derivatives were felt to be reliable.

RESULTS

Strain Data

Initial strain readings taken prior to the placement of fill indicated essentially all of the strain gages to be operative. However, by the time the fill had exceeded 100 ft (30.5 m), the majority of the gages had either ceased to function or were providing readings so erratic as to be judged unreliable. Moisture was identified as the most likely cause of these problems.

A complete set of strain readings from each rosette was recorded at approximately 10-ft (3-m) intervals of fill. These provided direct indications of strains in the circumferential and longitudinal directions and also principal stresses and directions at each rosette location. In addition, an attempt was made to investigate circumferential bending in the pipe by careful examination of the circumferential stress distribution through the thickness.

Although difficulty was experienced in analyzing the strain data, it was possible to extract some information indicating the overall culvert response. For example, typical plots of strain versus fill depth for typical locations are presented in Figures 34 and 35. These data were selected from gages that were still functioning normally after several months of construction.

The calculation of circumferential and longitudinal stresses required values of both ε_x and ε_y at the same point.

Unfortunately, the variability of the strain readings precluded the determination of σ_x and σ_y over any significant range of fill. However, it was possible to extract some information regarding the distribution of circumferential strain through the thickness, although these data became unreliable beyond approximately 50 ft (15 m) of fill. Typical plots of strain variation through the thickness for those gage locations where recorded data permitted such a determination are presented in Figures 36-38.

As may be observed from these figures, the circumferential strain variation through the thickness remained linear at most gage locations up to a fill depth of approximately 50 ft (15 m). Considerable variation in strain levels was observed at given fill depths. For example, at a fill depth of 38.5 ft (11.7 m) the maximum surface strains varied from 500 micro in/in (cm/cm) to 1,600 micro in/in (cm/cm).

Both rectangular and principal stresses were calculated from the available strain data. Because some gage data may have been inaccurate, the stress values are not considered reliable.

Deflection Data

Sketches of the deformed cross section of the steel pipe at four levels of fill are presented in Figures 39-46. These figures clearly show that, as expected, the pipe is flattened at the crown and deformed radially outward along the lower portion below the spring line. Of particular interest are the deformed shapes of two locations at a fill depth of approximately 256 ft (78 m) shown in Figures 45 and 46. These sketches show the extensive deformation experienced by the culvert at this fill depth and also dramatically illustrate some difference in loading that must have existed at two locations of the pipe only 6 ft (1.8 m) apart. At the upstream gage locations, the deformation pattern was approximately symmetric, while at the downstream location there was a significant nonsymmetric deformation pattern in the upper quadrants in the vicinity of the crown. This nonsymmetric deformation pattern could be attributed to a number of factors, but was likely the result of uneven loading of the pipe. At fills in excess of 100 ft (30.5 m), the deformation patterns at both locations were similar.

Similar plots of deformed cross section for the reinforced concrete pipe are shown in Figures 47-52. While the deformations are much smaller than those experienced by the steel pipe, these deformations do appear to be unexpectedly large for a culvert designed to behave rigidly. For example, Figures 49 and 50 show the deformation patterns of the concrete culvert at a fill depth of 87 ft (26.5 m). The plot clearly shows an asymmetric deformation on one side of the culvert. This deformation at this particular location and fill depth are of interest because it occurred at approximately the same time as did an observed period of rapid settlement to be discussed subsequently. It has been suggested that extreme deformations may have been associated with a postulated but unproven sliding lateral movement of soil into the space originally occupied by the baled straw resulting in a localized loading in the pipe.

It should be kept in mind that the deformation plots in Figures 39-52 were computer-generated and were based on a polynomial approximation to the true shape. Thus, it is possible that certain of these deformation patterns may not accurately represent the true shape of the cross section although the curves do pass through the deformed gage points.

Another quantitative measure of the deformation of the two culverts is provided by examining the variation of the radial displacements of various points on the culvert wall as a function of fill depth. A plot of the displacement of the crown versus fill depth is given in Figure 53 for both locations on the steel pipe. The fill depth was measured from the culvert crown and, as indicated in the figure, the crown displaced slightly upward as the fill was first placed. After the fill reached a depth of approximately 5 ft (1.5 m) over the crown, the crown point began to displace vertically downward. Although there were slight variations in displacement, there was an approximately linear relationship between crown deflection and fill depth, even up to a fill of 256 ft (78 m), the deepest fill for which deflection measurements are available. The data in Figure 53 also show the difference in behaviors of the steel culvert at the upstream and downstream gage locations, with the downstream location consistently undergoing larger crown deflections.

Research personnel who entered the steel culvert to obtain deformation readings after the fill was completed reported no plate tearing nor serious bolt tipping. However, this pipe has undergone severe local plastic deformations in the region of the crown. Visual observations of the interior of the concrete pipe indicated rather serious cracking and spalling at 10 and 2 o'clock of the middle sections of the pipe. Differential settlement of the fill under the concrete culvert is believed to have caused joints in the top of the culvert to open and those in the bottom to close. These movements were not monitored as part of the study, but the openings at the top were observed to be as much as 2 to 3 inches (5.1 to 7.6 cm).

Radial deflections of the gage points on the concrete culvert were also calculated. However, the absolute displacements were sufficiently small to suggest that any such plot of radial deflections, which were calculated values and subject to error, might be misleading. Thus no plots are presented. As an indication of the order of magnitude of the displacements calculated, the maximum crown deflection was approximately 0.10 in (0.25 cm). At a point 30° from the crown, the maximum radial displacement was calculated to be approximately 0.55 in (1.40 cm) at a fill depth of 87 ft (26.5 m). This displacement was measured at the downstream gage location and is consistent with the cross section deformation of that location shown in Figure 50.

The deformation data were also utilized to calculate bending moments in the pipe wall. Polynominal representations of the pipe cross section were generated using fourth-degree, eighth-degree, and twelfth-degree polynominals. These functions were then differentiated twice to yield curvatures and, after corrections were made for the initial curvature in the pipe, bending moments were calculated. It was found that the lower degree polynominal representation could be used with no significant loss in accuracy.

Typical bending moment distributions around the circumference, normalized with respect to EI, are plotted in Figures 54-61 for the steel pipe. Similar plots for the concrete pipe are shown in Figures 62-67. Several interesting features may be noted from these plots. First, the distribution of bending moment is not symmetric, which indicates some asymmetry of loading. Also, even for the relatively low levels of fill for which moments are shown, the calculated bending stresses exceed the yield strength of the steel material in localized regions and are sufficient to cause cracking in the concrete culvert. Stresses determined from bending moment calculations cannot be directly compared with stresses determined from strain gage measurements, since the former are based on gross deformations while the latter are based on point information. Nevertheless, both sets of data indicate the likelihood of local yielding or cracking of the pipe wall for loads equal to or less than the design load.

Settlement Data

The use of baled straw over the rigid concrete pipe, being a fairly common practice throughout the country until recently, at least, was followed on this project. This straw above the pipe, according to accepted theory, should have deformed more than the soil alongside the straw and thereby redistributed and reduced the weight and pressure of the soil on the top of the pipe.

The Idel Sonde readings of plate settlement have continued for almost two years and are plotted in Figures 68-71. In these figures the settlement of each plate in a given vertical array is plotted as a function of time. Also shown in each figure is the average rate of fill construction above the invert of the concrete pipe. Settlements of almost 5 ft (1.52 m) were observed under some plates. Due to the porous nature and unsaturated condition of most of the fill, particularly the shot rock, the settlement was primarily an initial compression or internal consolidation. Therefore, the settlements occurred quickly and appeared to be stabilizing at the time of the last readings (June 1976).

Two periods of rather rapid settlements may be observed on these figures. The first occurred between April 22 and June 2, 1975, when approximately 75 ft (22.9 m) of fill had been placed above the invert. The second rapid rate of settlement began when the rate of filling was accelerated about mid-July 1975. The first period of rapid settlement is believed to have been associated with a postulated but unproven lateral movement of soil into the space occupied by the baled straw, at which time the settlement plates below the top of the pipe were cut off in the column 12 ft (3.66 m) from the center of the pipe.

Shown in Figure 72 are the settlement curves for four plates located at essentially the same elevation but positioned at different distances from the centerline of the concrete pipe. In this figure it can be noted that the ultimate settlement increased with distance from the centerline of the pipe. This type of movement is fairly typical of all of the plates at a given level in the embankment. However, this pattern of settlement is not consistent with that postulated for the baled straw installation, where greater settlement for certain depths of fill over the pipe, at least, would normally be expected to occur over the centerline of the pipe. The presence of increasing depth of fill under plates farther from the centerline of the pipe may in part explain this apparent inconsistency. However, for this installation, there is evidence that the baled straw and surrounding soil did not perform as theory predicts.

CONCLUSIONS AND RECOMMENDATIONS

Much of the data, collected as a part of this study and which are available, should be of interest to the pipe analyst and designers and help them in checking the validity of existing or proposed analysis and design methods or theories.

Because of seepage of corrosive and extremely slick liquids from the water soaked and decomposing straw through open joints and cracks in the concrete pipe, it is the opinion of the researchers that the Virginia Department of Highways and Transportation should seriously consider alternate materials as backfill over culverts where groundwater is available and the fill is porous. Consideration should be given to the use of loose inorganic soil until a better material is found. There was some, but not conclusive, evidence that the compacted fill on at least one side of the straw backfill may have failed in shear and moved laterally into the zone occupied by the straw. Therefore, the height, size, shape, etc., of the zone of soft or compressible backfill material above pipes should be restudied.

The placing of pipe adjacent to a ravine or valley wall as opposed to placing it midway between valley walls should be restudied. There was evidence to indicate that greater settlement developed farther from the valley wall, which transferred unexpected soil pressure to one side of the pipe.

LATEST DEVELOPMENTS

Since the installation of the concrete culvert the Virginia Department of Highways and Transportation has modified two aspects of the imperfect trench method: 1) The height of straw above the culvert has been reduced from twice the diameter of the culvert to the diameter of the culvert; and 2) the specifications have been changed to require that 1 ft (0.3 m) of soil cover be placed over the culvert rather than placing the straw directly over the culvert.

ACKNOWLEDGEMENTS

The experimental phase of this study required the assistance and cooperation of a number of agencies and individuals. The contractor, Vecellio and Grogan, Inc., cooperated fully in both carefully placing the culvert test section and in providing services needed during subsequent site visits for data acquisition. The Virginia Department of Highways & Transportation and the Federal Highway Administration were extremely helpful in providing material, assistance, and advice during the entire project.

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The project was conducted under the general supervision of the Virginia Highway & Transportation Research Council. M. C. Anday, senior research scientist, actively participated in many phases of the project and offered valuable guidance during the entire study.

Special thanks are due M. O. Harris, G. T. Gilbert, and R. W. Givens, who were responsible for much of the data collection; G. G. Fornes, who not only assisted in many phases of the instrumentation, but also designed and fabricated the mechanical extensometers used; and to M. W. Jones, who designed the major instrumentation facilities, supervised the installation of recording equipment, and significantly contributed to all phases of data collection.

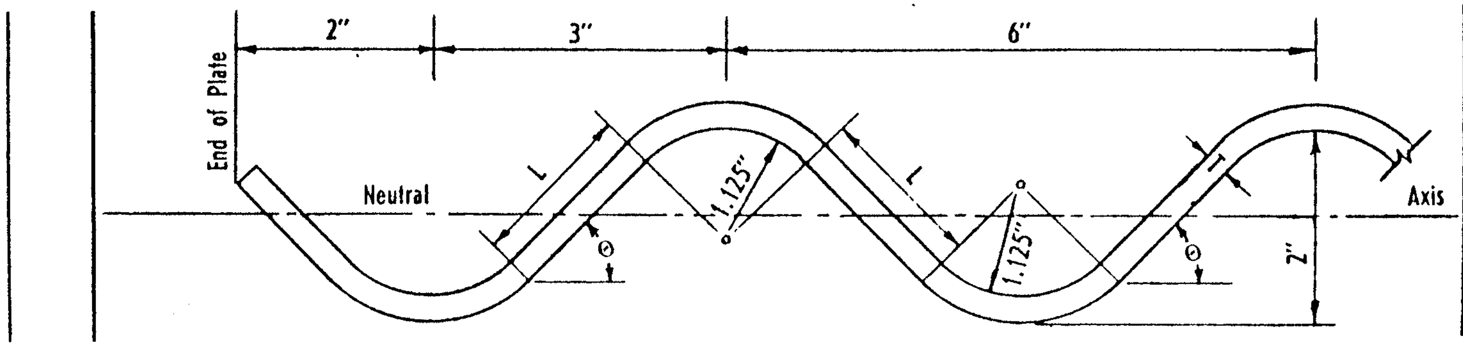
Finally, appreciation is expressed to the Project Task Group, which provided valuable guidance and advice during all phases of the project and which included, in addition to persons already mentioned, E. C. Cochran, Jr., L. D. Walker, L. E. Ashbaugh, and G. S. Meadors, Jr.

Table 1

Plate and Material Properties

Chemical Property	Percentage	Physical Property	
Carbon	0.10	Tensile Strength	45,000 psi
Manganese	0.40	Yield Strength	33,000 psi
Phosphorus	0.015	Elongation	30%
Sulphur	0.03	Rockwell Hardness	B-55
Copper	0.25	Modulus of Elasticity	30×10^6 psi

Conversion: 1 psi = 6,895 Pa



Gage	Foot Weight (Lb/Sq.Ft)	Uncoated Thickness "T" (In)	Tangent Length "L" (In)	Tangent Angle Θ (Degrees)	Area of Section A* (In^2/Ft)	Moment of Inertia I* (In^4/Ft)	Section Modulus S* (In^3/Ft)	Radius of Gyration R (In)	Developed Width Factor
5	8.750	.2145	1.773	45.47	3.199	1.523	1.376	.690	1.243
3	10.000	.2451	1.738	45.77	3.658	1.754	1.562	.692	1.244

*Values shown are for one foot of horizontal projection.

Figure 1. Dimension and design properties of Republic sectional plate.
Conversion: 1 lb/ft^2 = 4.88 kg/m^2;
1 in = 2.54 cm;
57.296° = 1 Rad

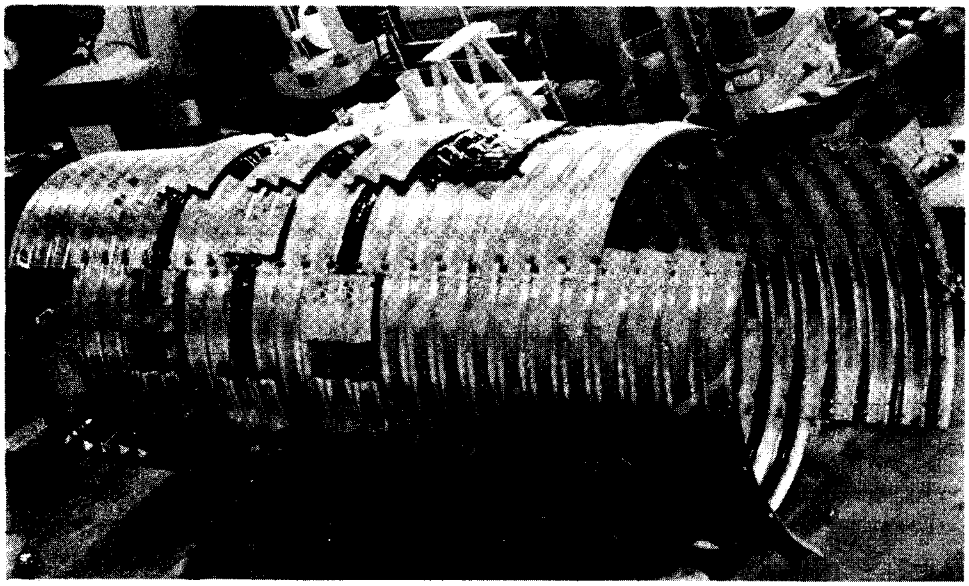


Figure 2. Assembled steel culvert section.

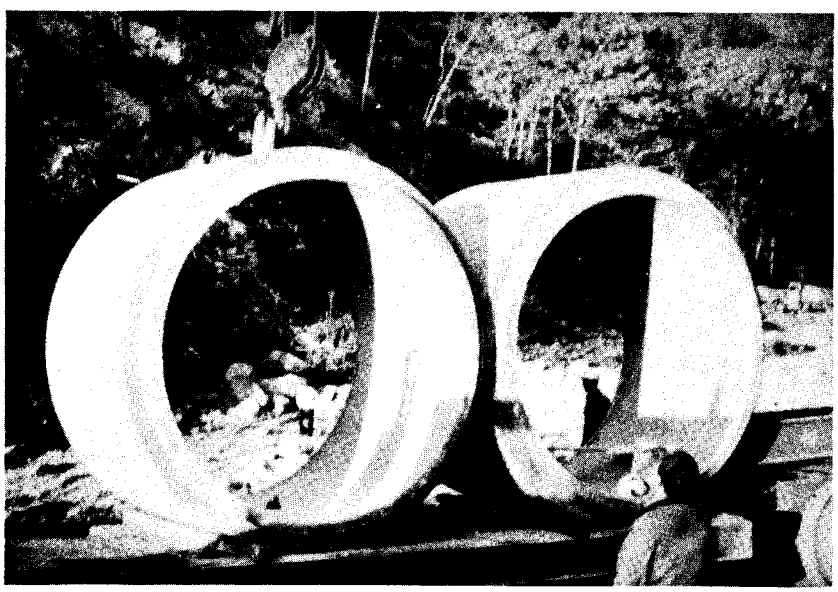


Figure 3. Reinforced concrete pipe sections.

Figure 4. Culvert location in Carroll County.

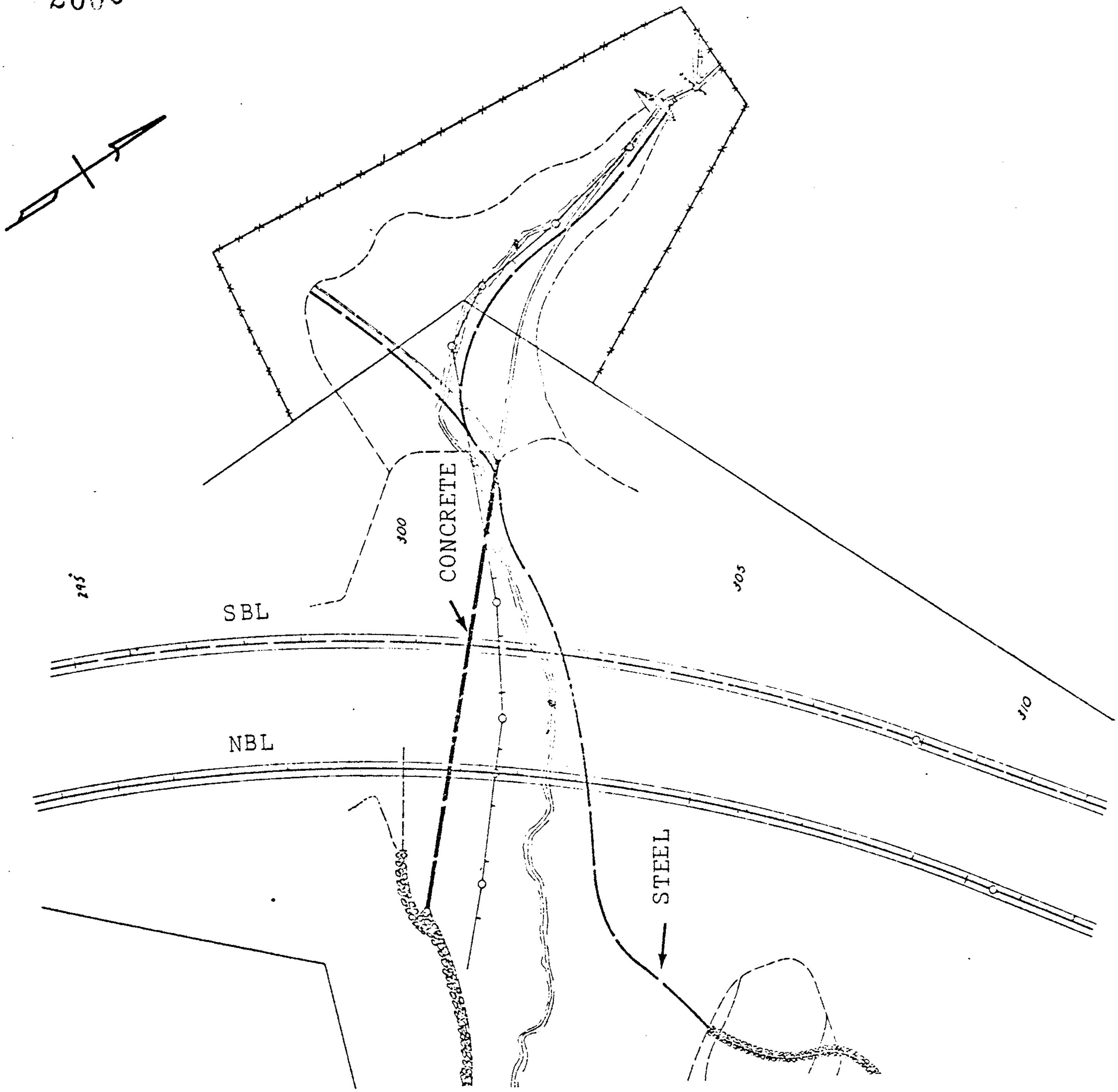


Figure 5. Plan view of steel and concrete culverts.

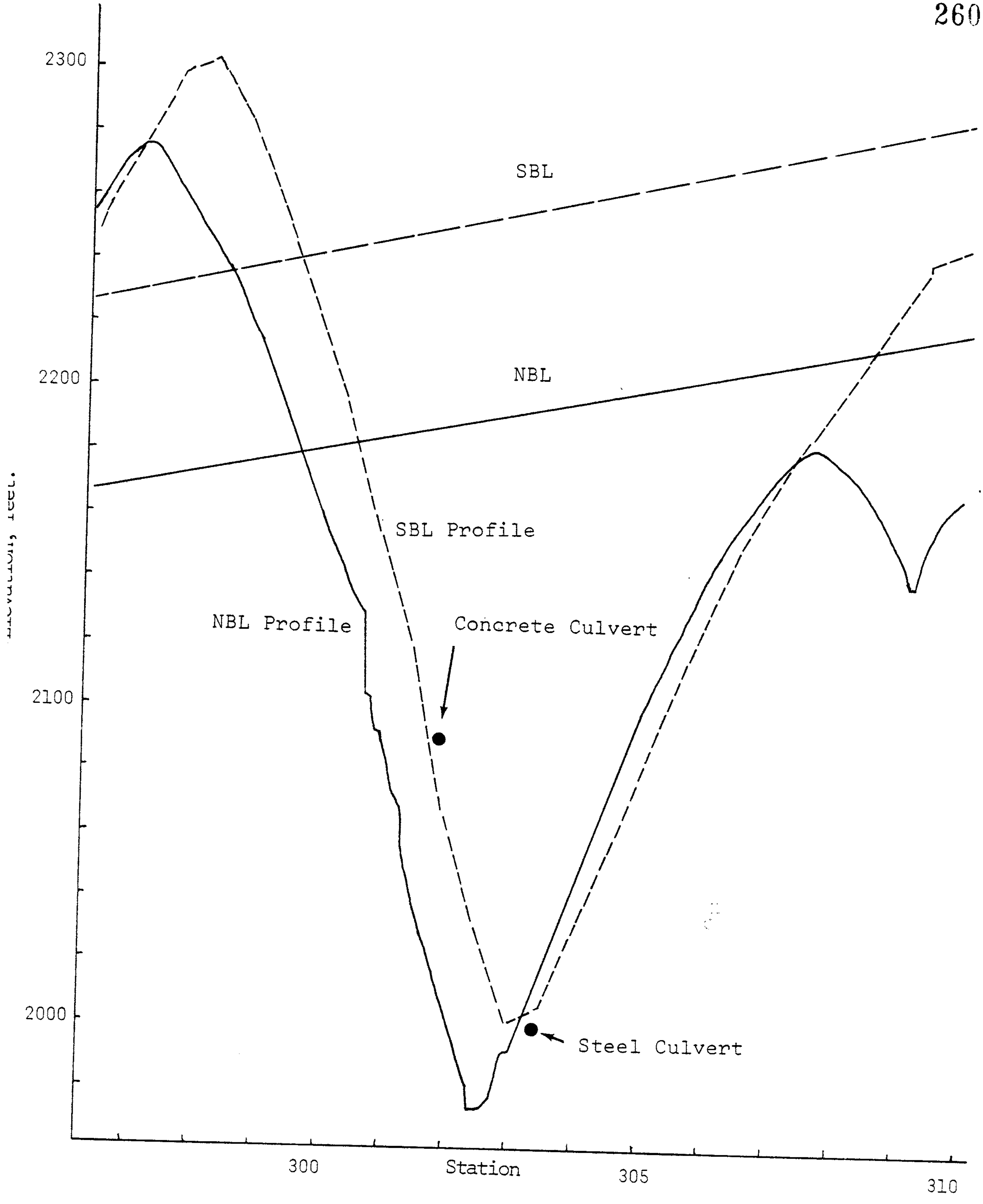


Figure 6. Sketch of terrain at culvert location.

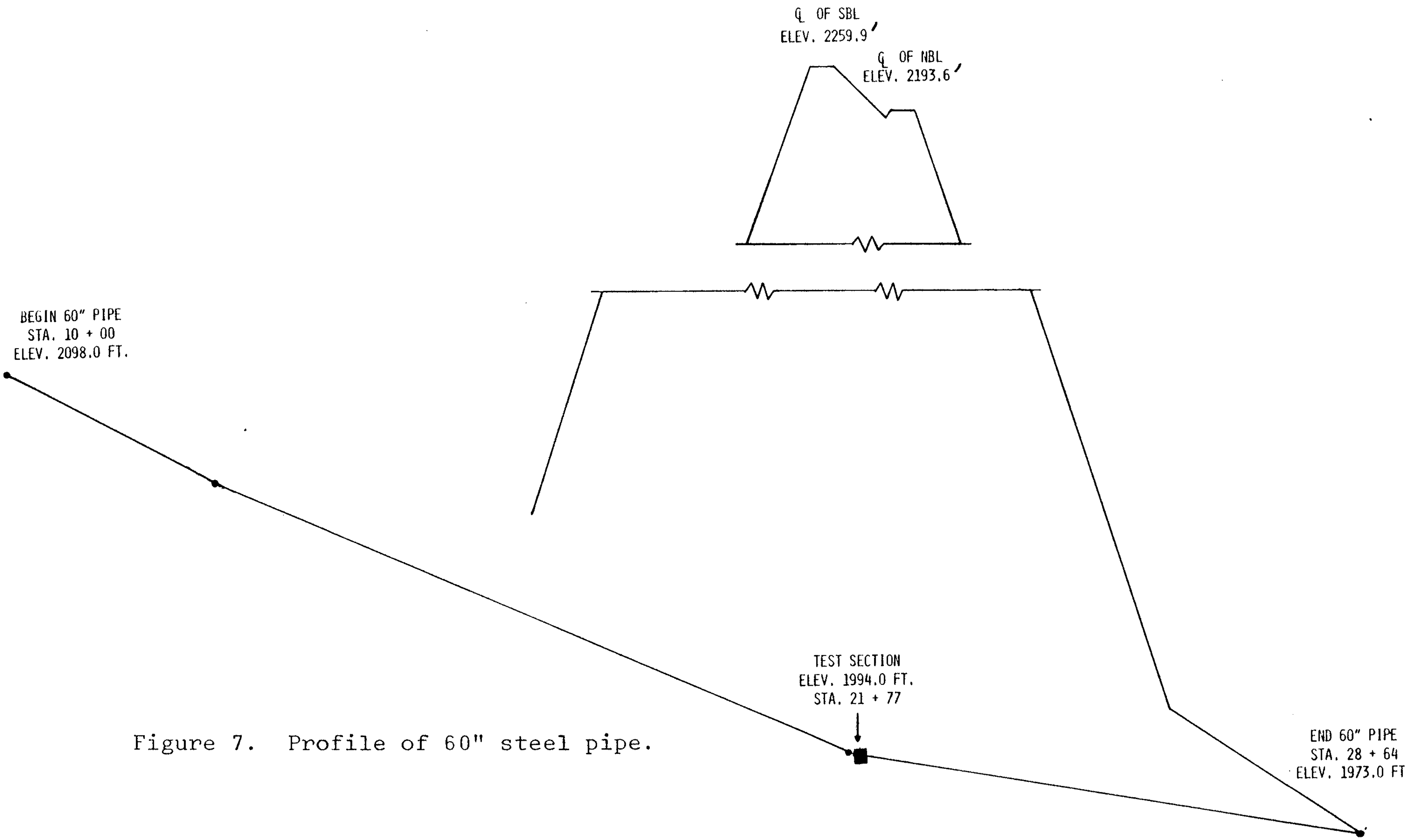


Figure 7. Profile of 60" steel pipe.

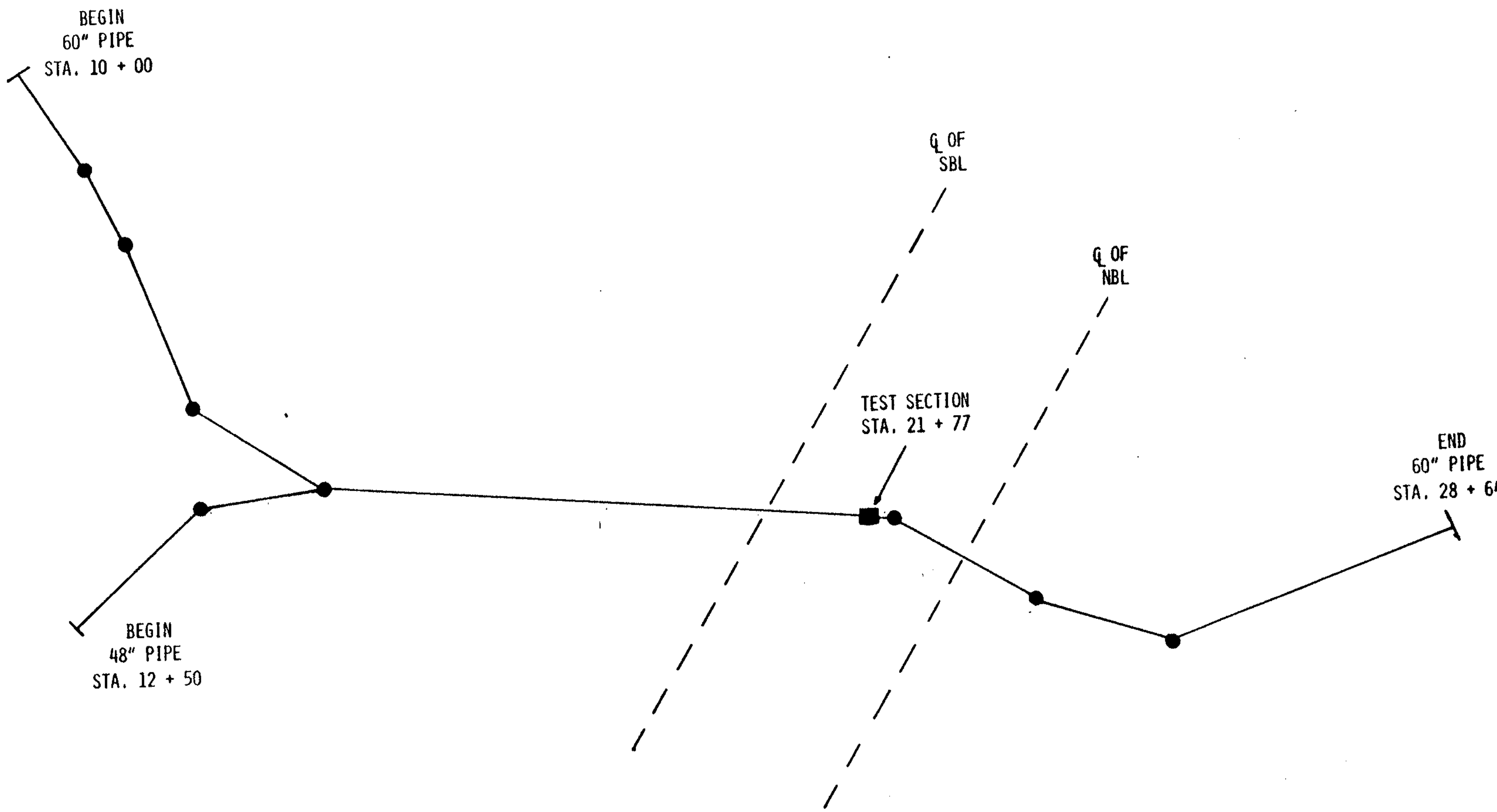


Figure 8. Plan view of 60" steel pipe.

Figure 9. Prepared bedding for steel culvert.

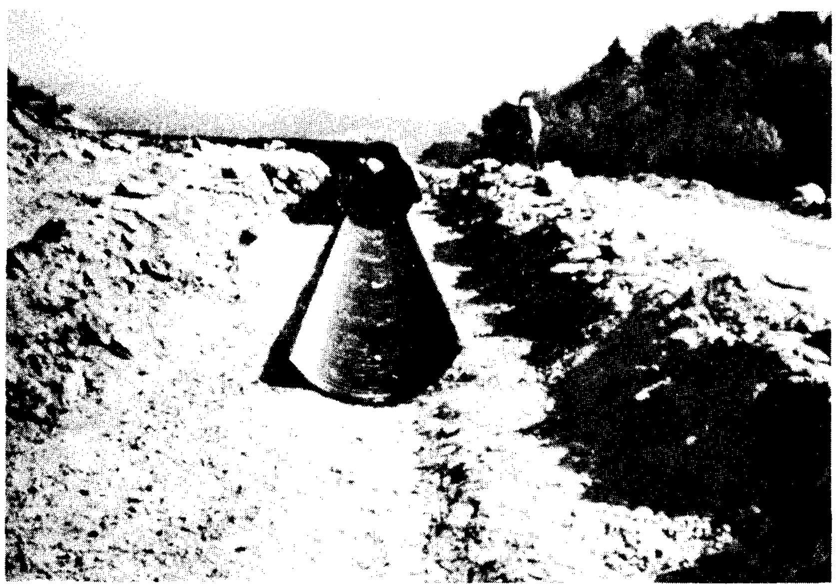


Figure 10. Invert sections in place.

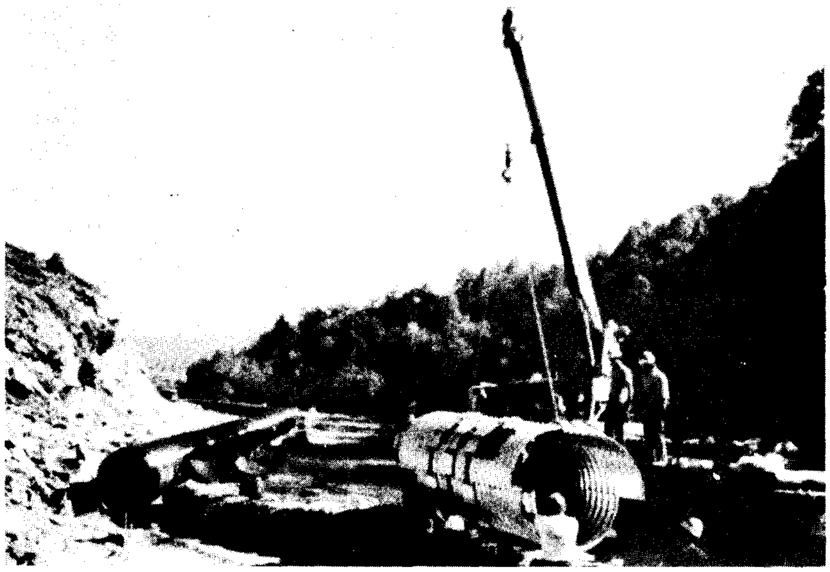


Figure 11. Test section being placed.

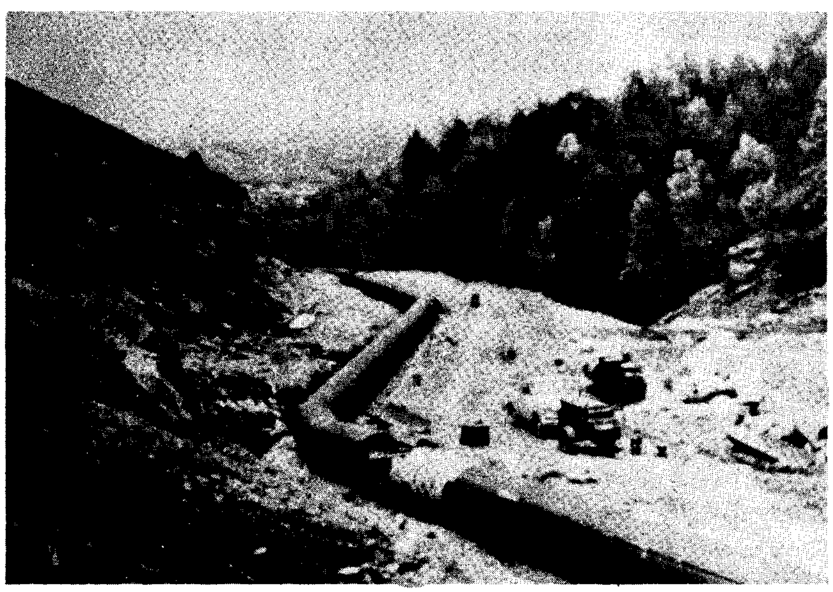


Figure 12. Test section in place.

Figure 13. Backfill around steel culvert.

Figure 14. Preparation of bedding for concrete culvert.

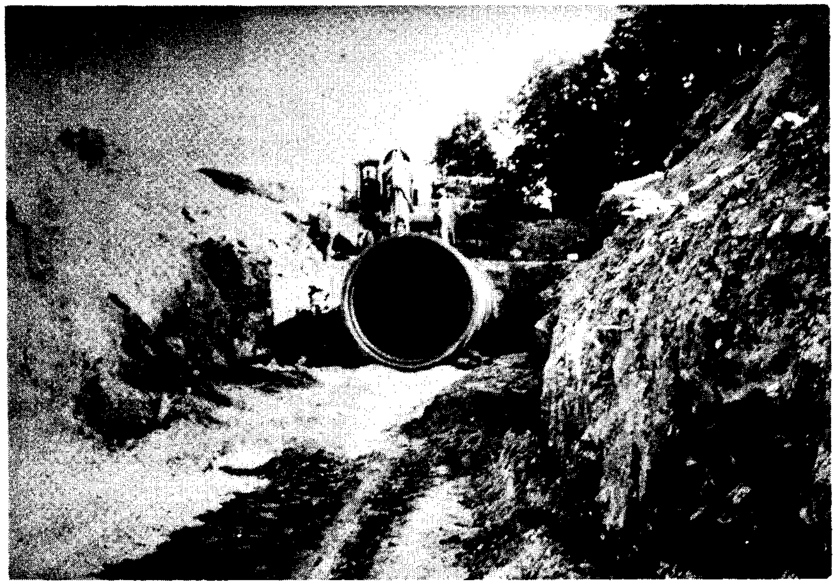


Figure 15. Backfilling and straw placement around concrete culvert.

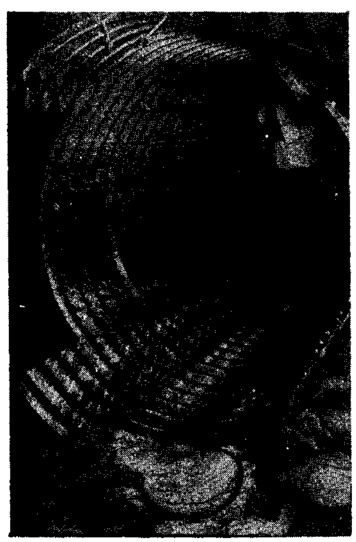


Figure 16. Steel test section being assembled.

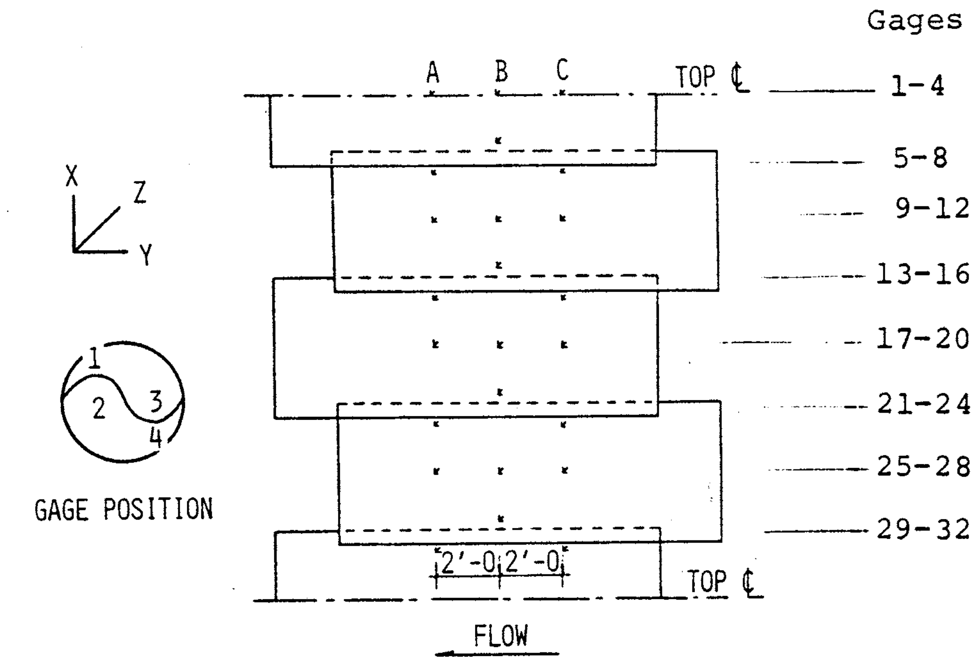


Figure 17. Strain gage plan (view from outside).
Conversion: 1 ft.= 0.3 m

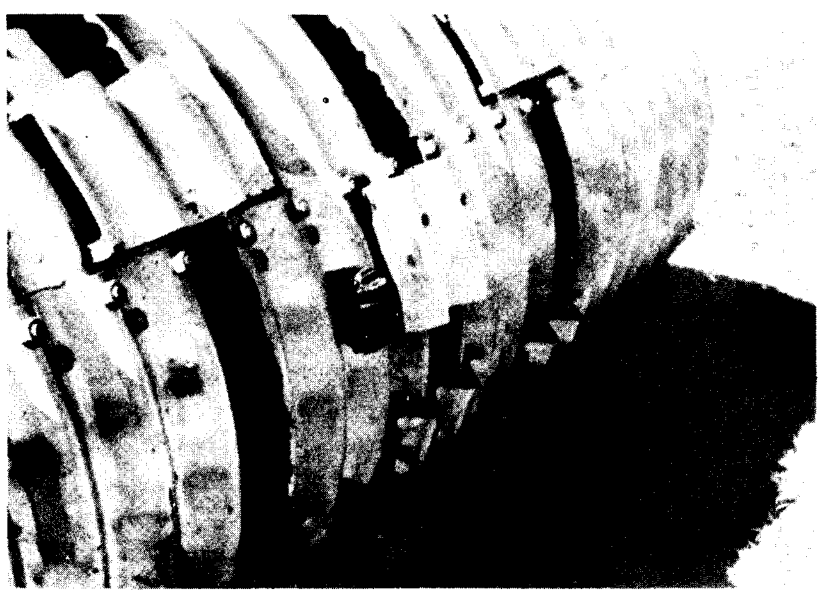


Figure 18. Enclosure for dummy gage.

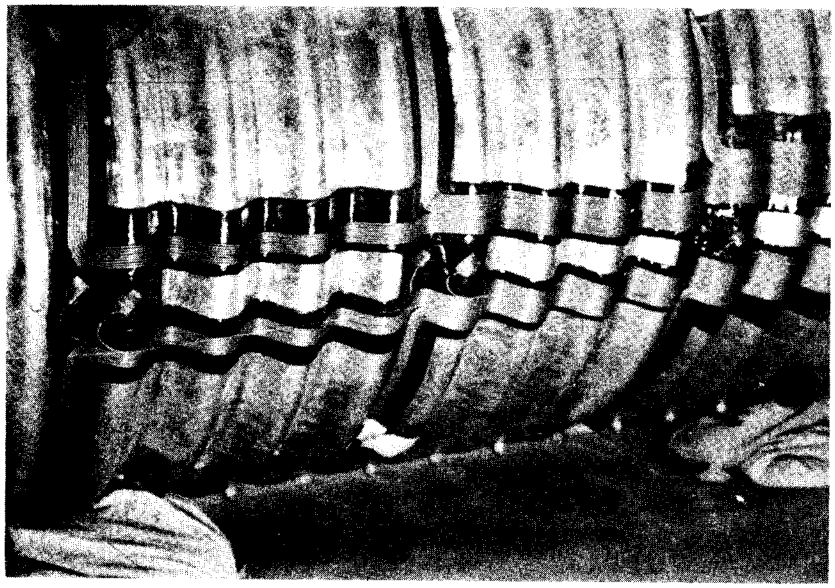


Figure 19. Rosette gages on steel pipe.

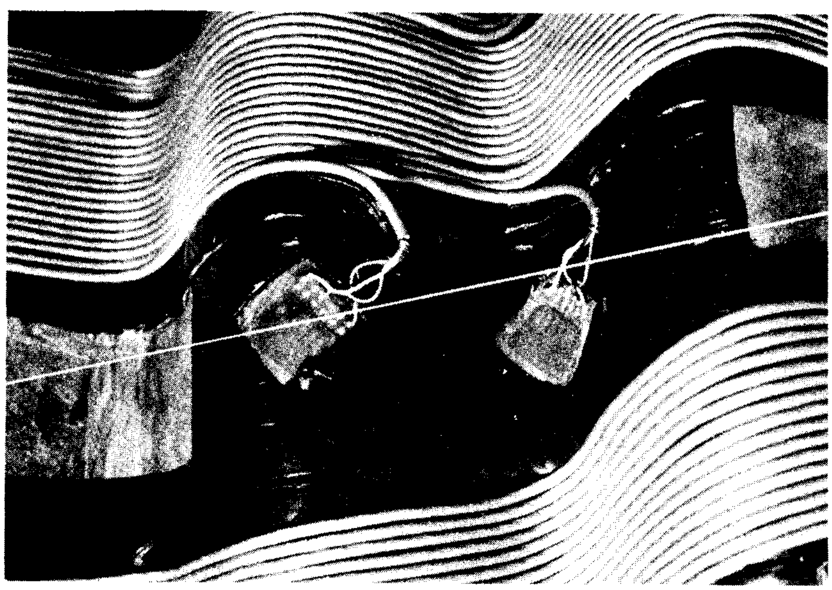


Figure 20. Completed instrumentation on steel pipe.

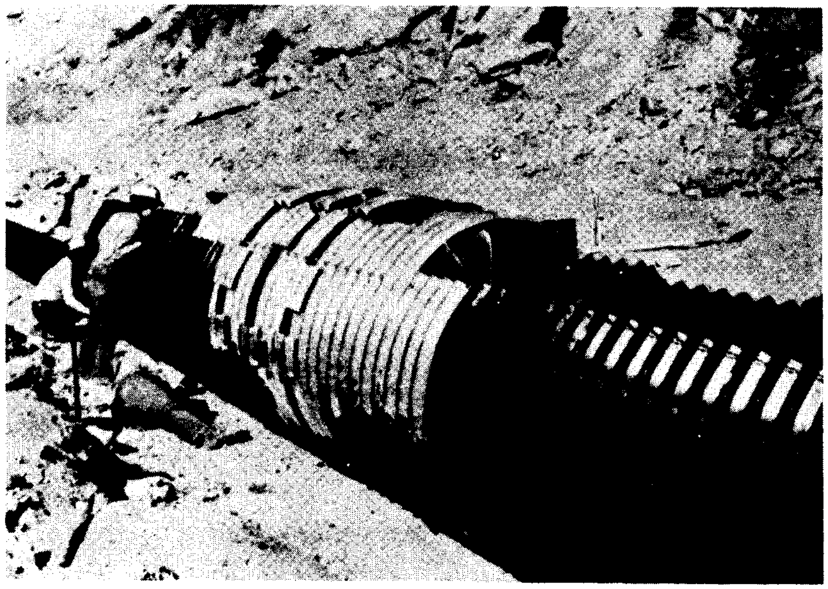


Figure 21. Completed test section in place.

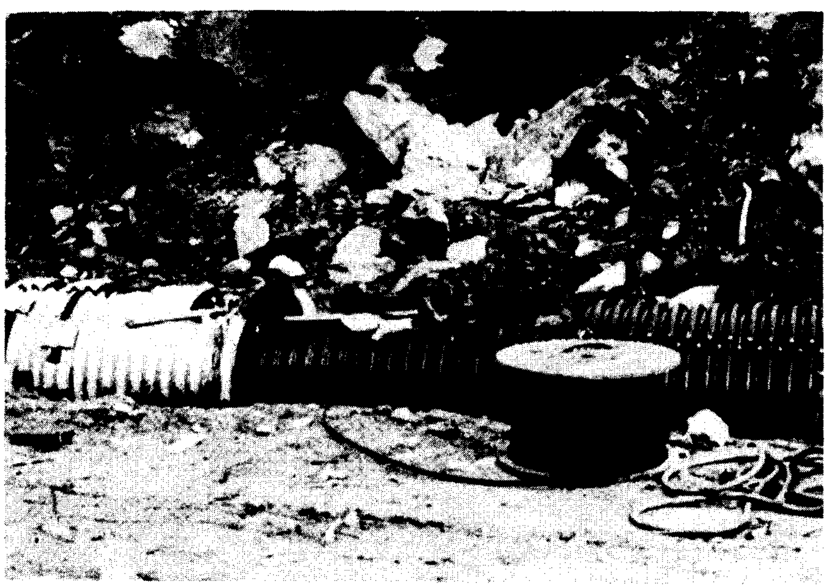


Figure 22. Wiring for instrumented test section.

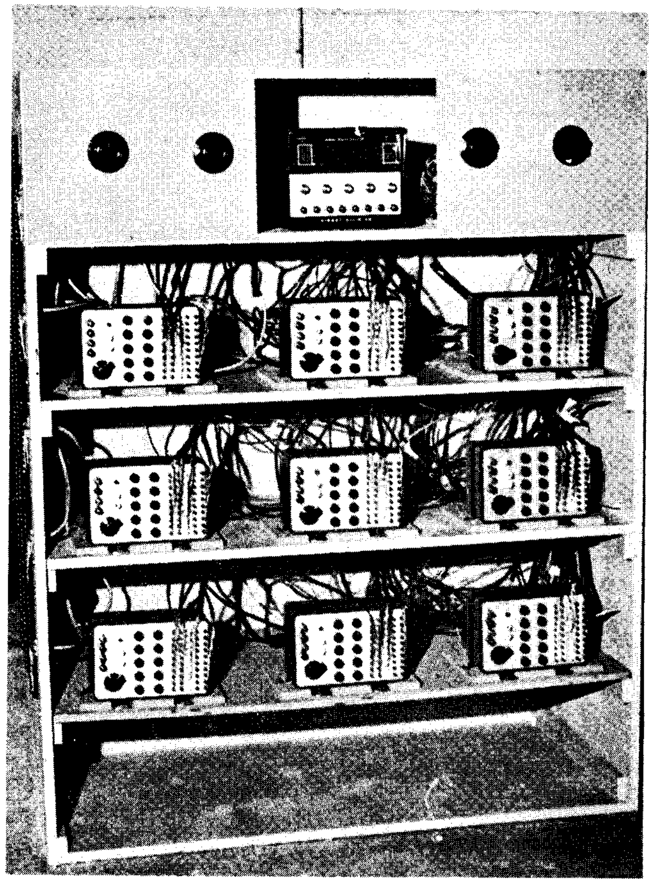


Figure 23. Instrumentation panel showing the strain indicator and switch boxes.

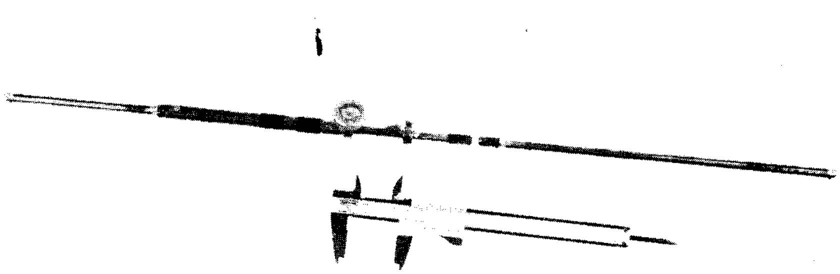


Figure 24. Typical extensometer with calipers.

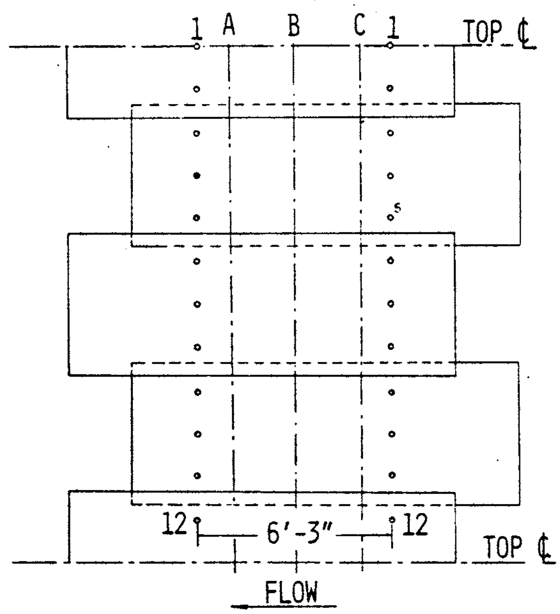


Figure 25. Deflection gage plan.
Conversion: 1 ft.= 0.3 m;
1 in.= 2.54 cm

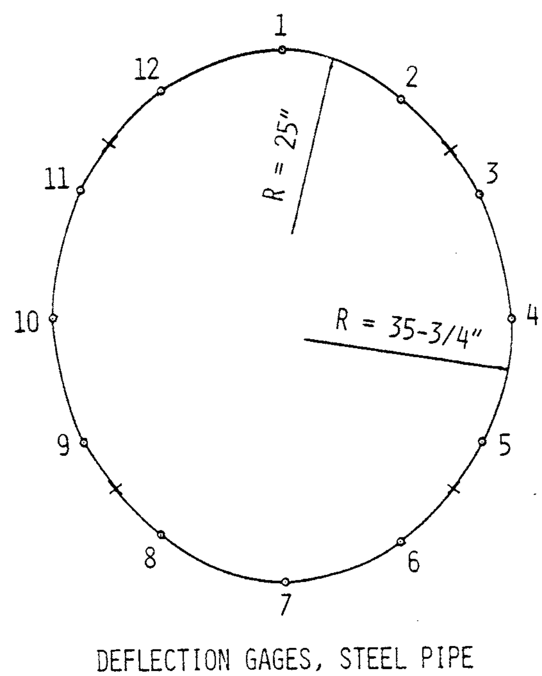


Figure 26. Deflection gage location, facing upstream.
Conversion: 1 in. = 2.54 cm

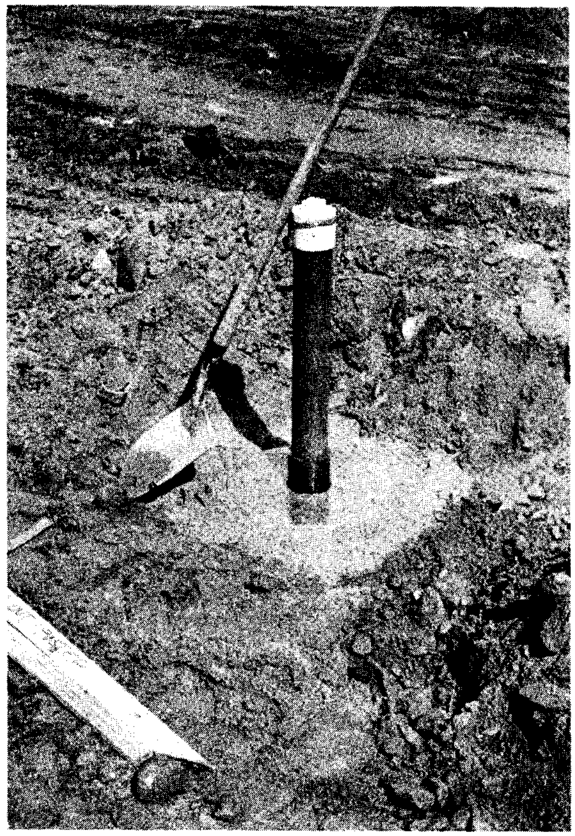


Figure 27. Settlement plate bedded in sand with plastic tube extended.

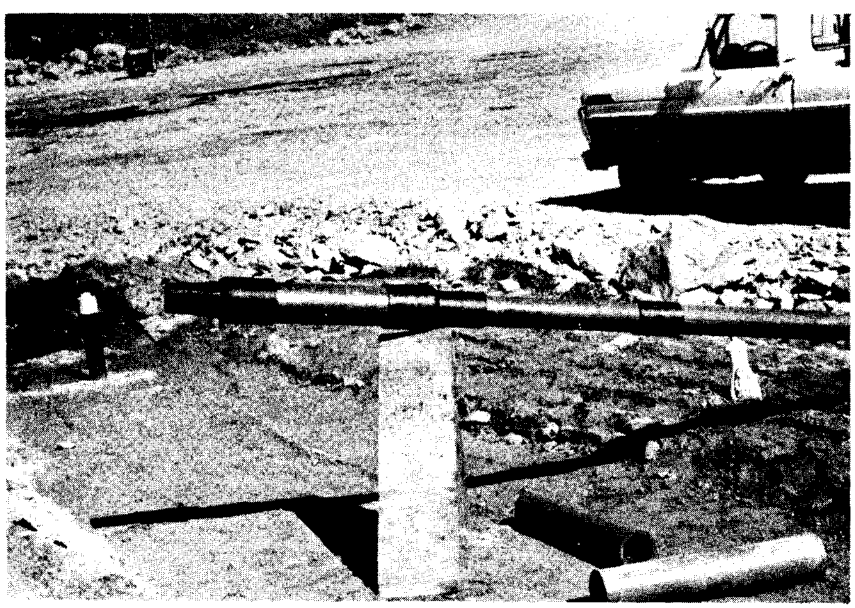


Figure 28. Plastic tubing, connecting sleeve, and rubber boots.

Figure 29. Four arrays of plastic tubes being installed.

Figure 30. Backfill around the four arrays of tubes.

Figure 31. Shot rock layers.

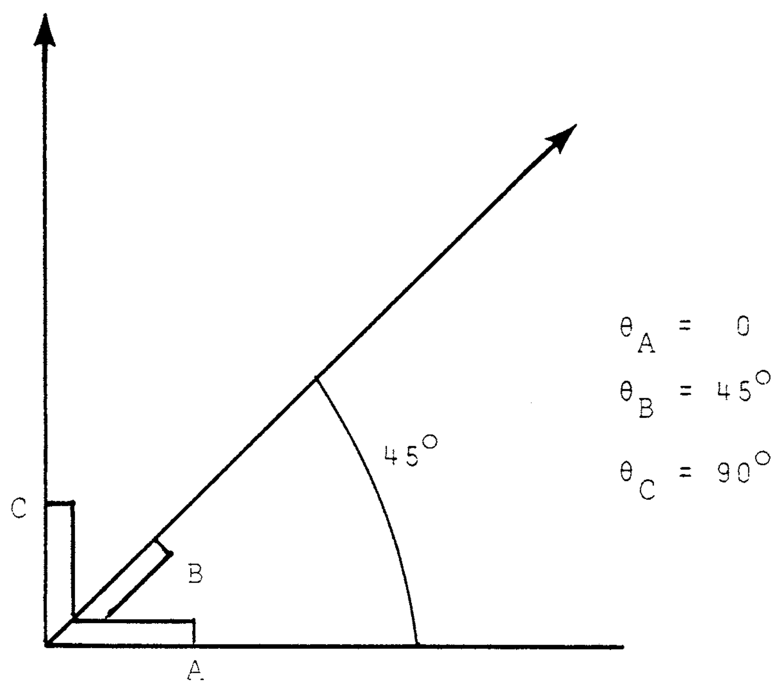


Figure 32. Gage positions in three element strain rosette. Conversion: 57.296° = 1 Rad.

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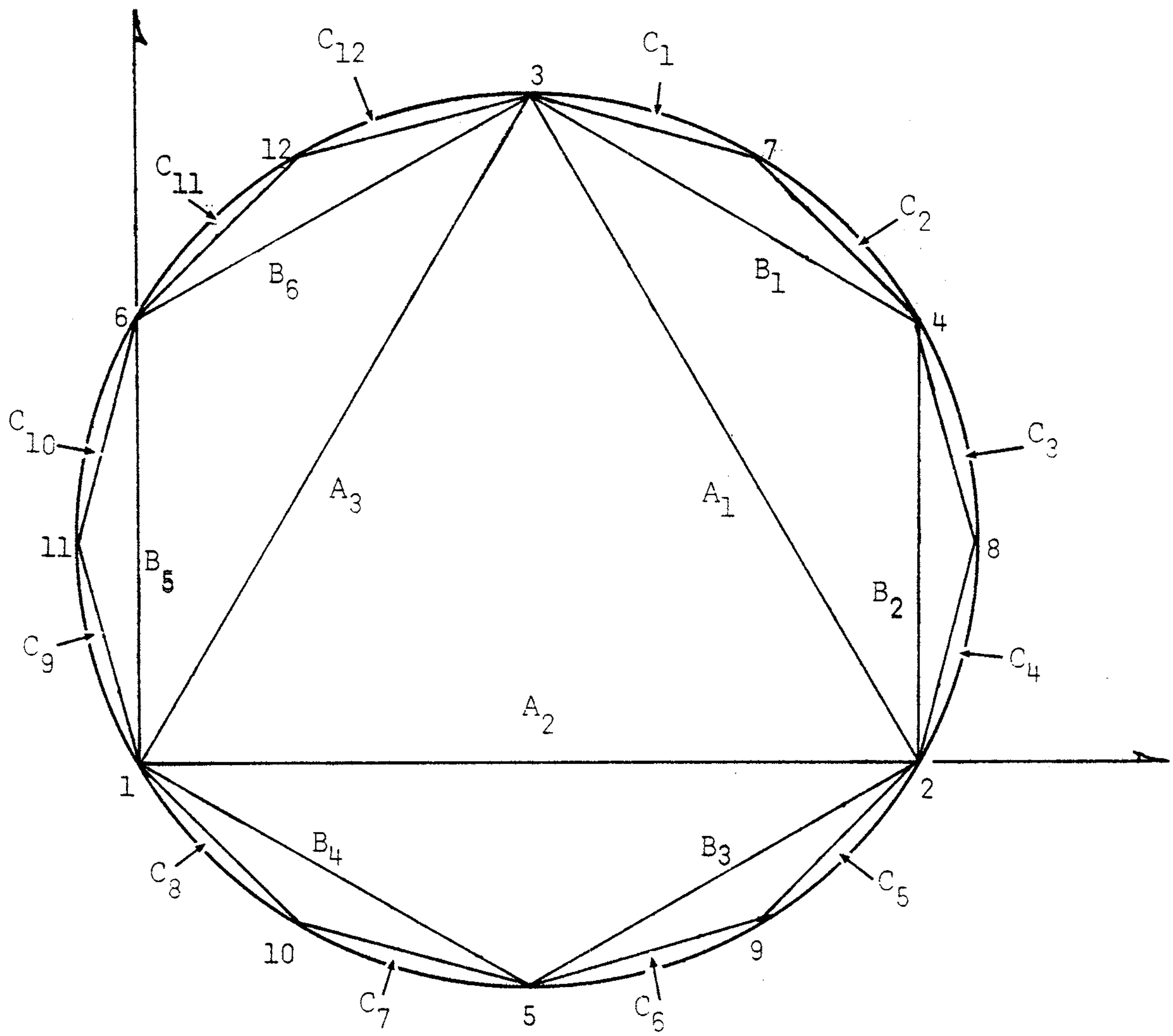


Figure 33. Labeling of gage points and chords for deformation analysis.

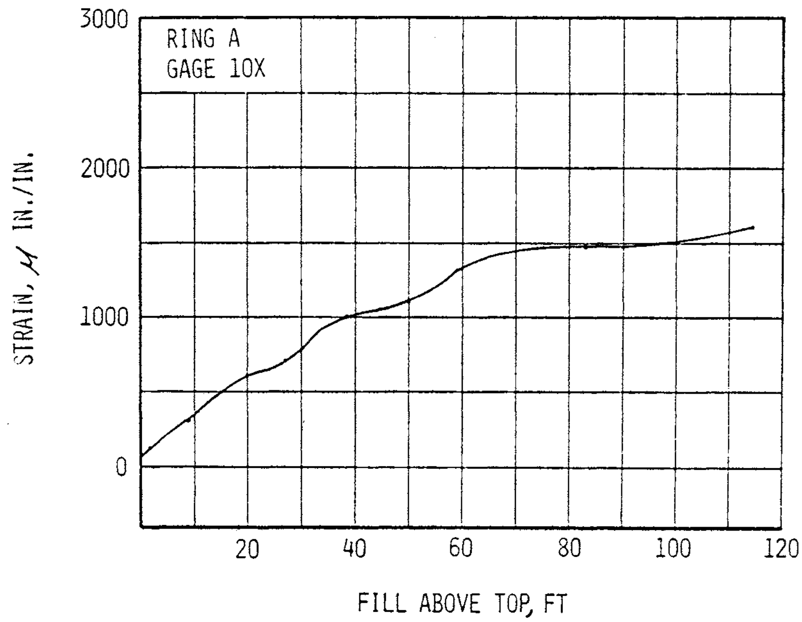


Figure 34. Strain versus fill depth (Ring A, Gage 10X). Conversion: 1 ft.= 0.3 m

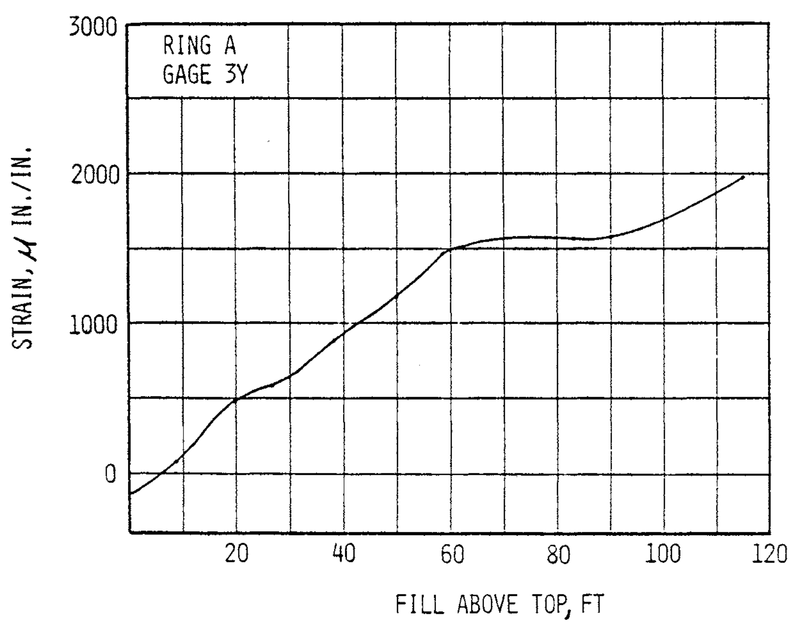


Figure 35. Strain versus fill depth (Ring A, Gage 3Y0. Conversion: 1 ft.= 0.3 m

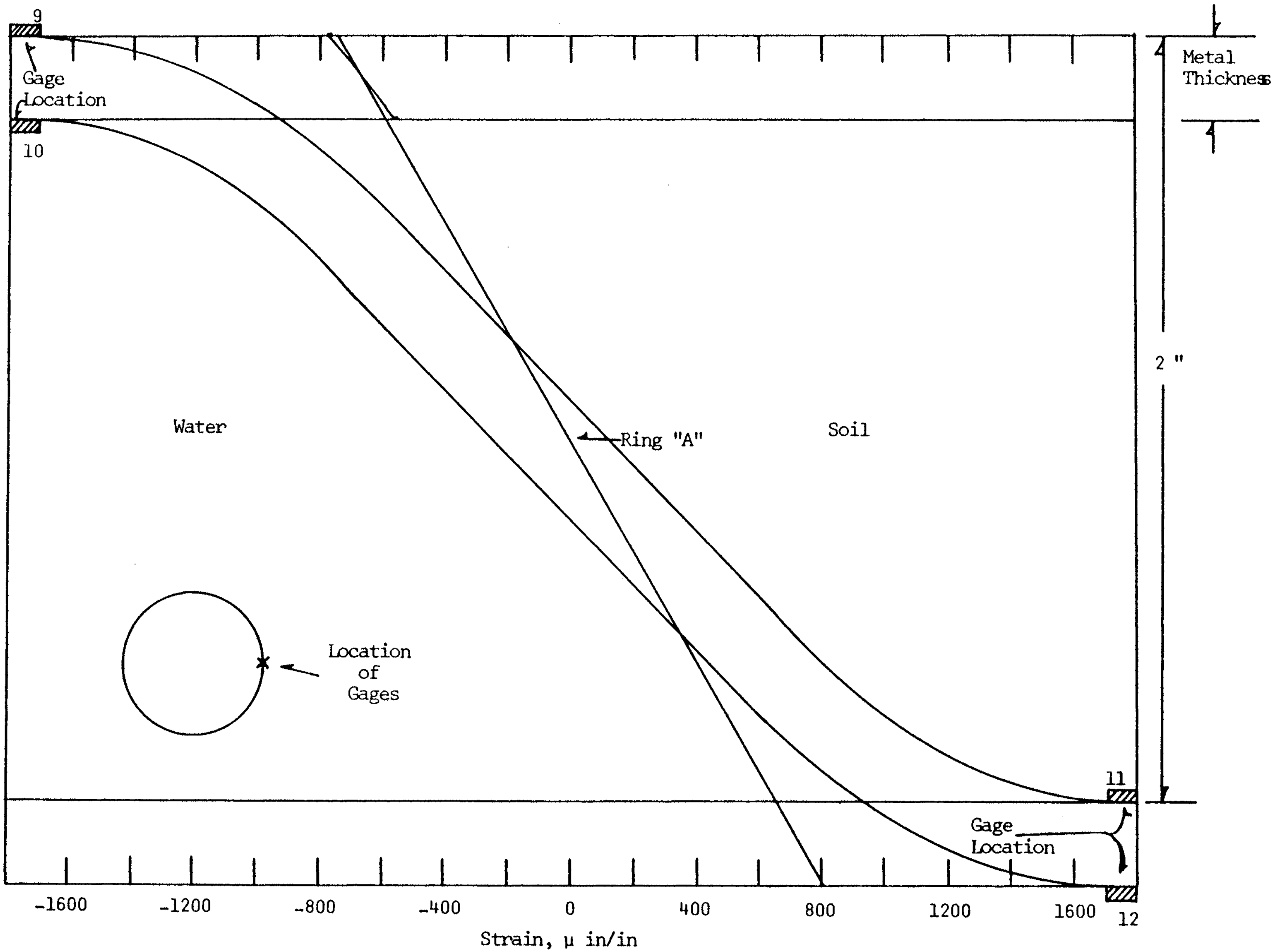


Figure 36. Circumferential strain distribution through wall thickness at 20 feet of fill. Conversion: 1 ft. = 0.3 m.

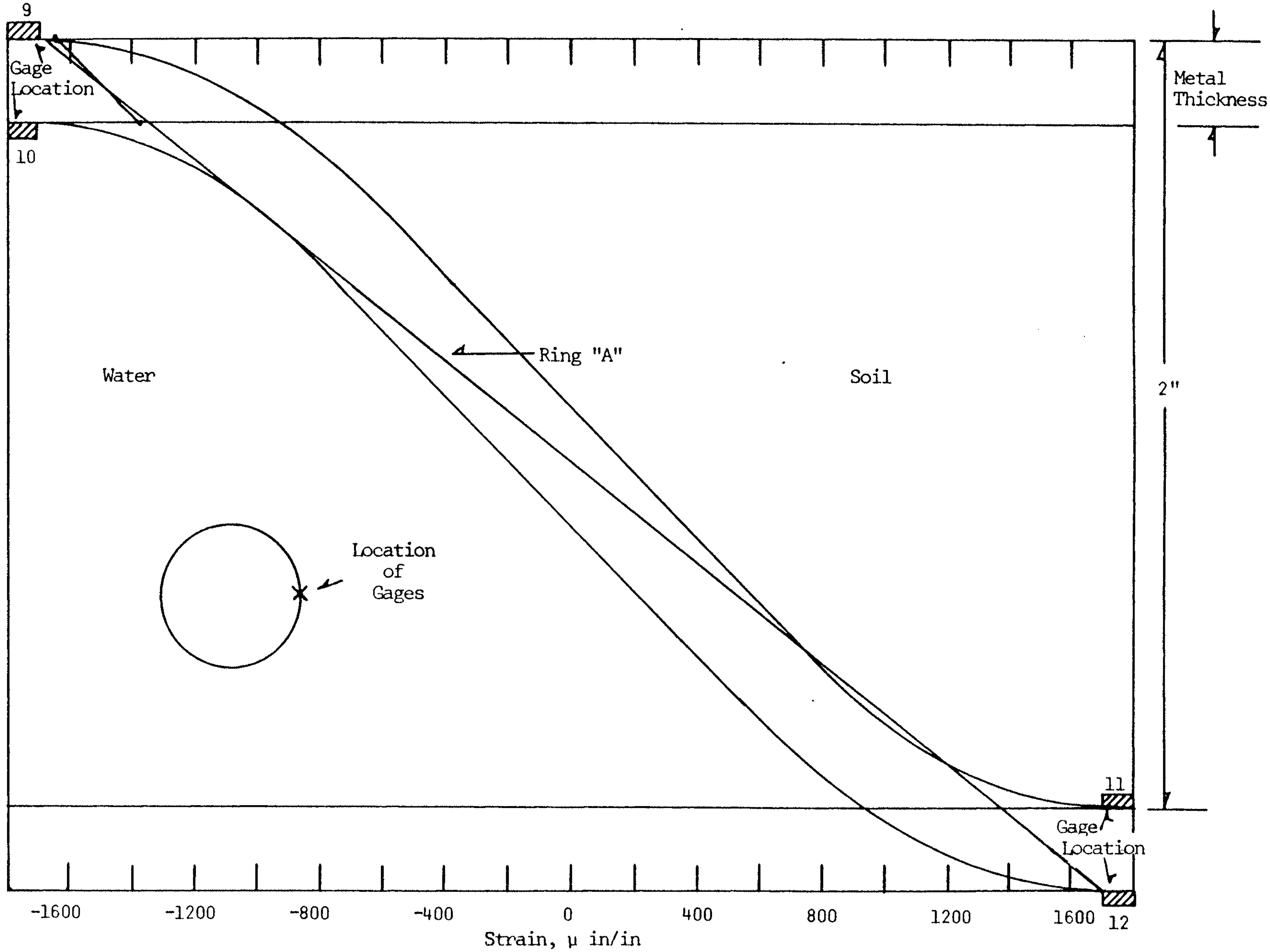


Figure 37. Circumferential strain distribution through wall thickness at gage locations 9-12 at 38.5 feet of fill.
Conversion: 1 ft. = 0.3 m

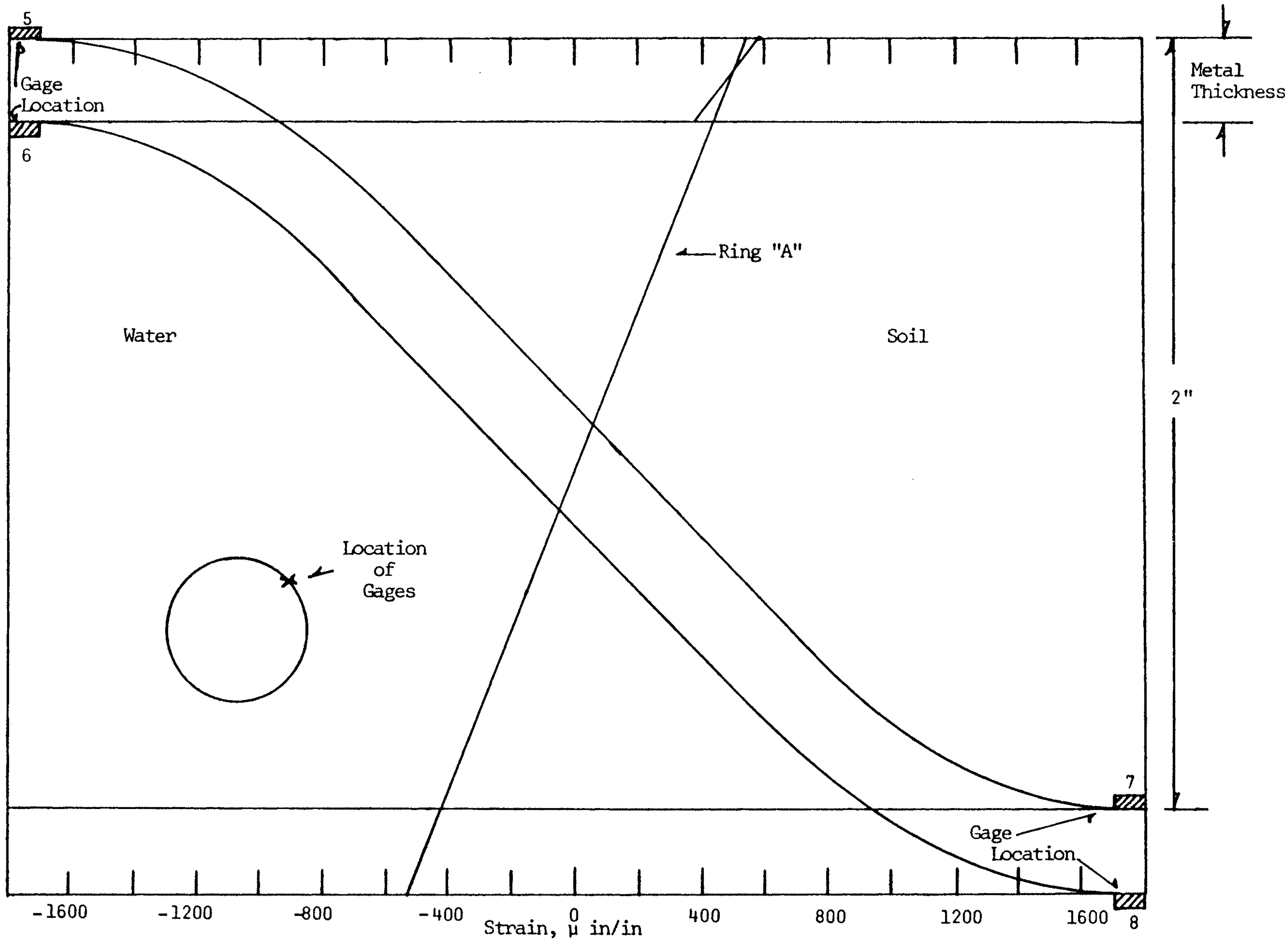


Figure 38. Circumferential strain distribution through wall thickness at gage locations 5-8 at 38.5 feet of fill.
Conversion: 1 ft. = 0.3 m

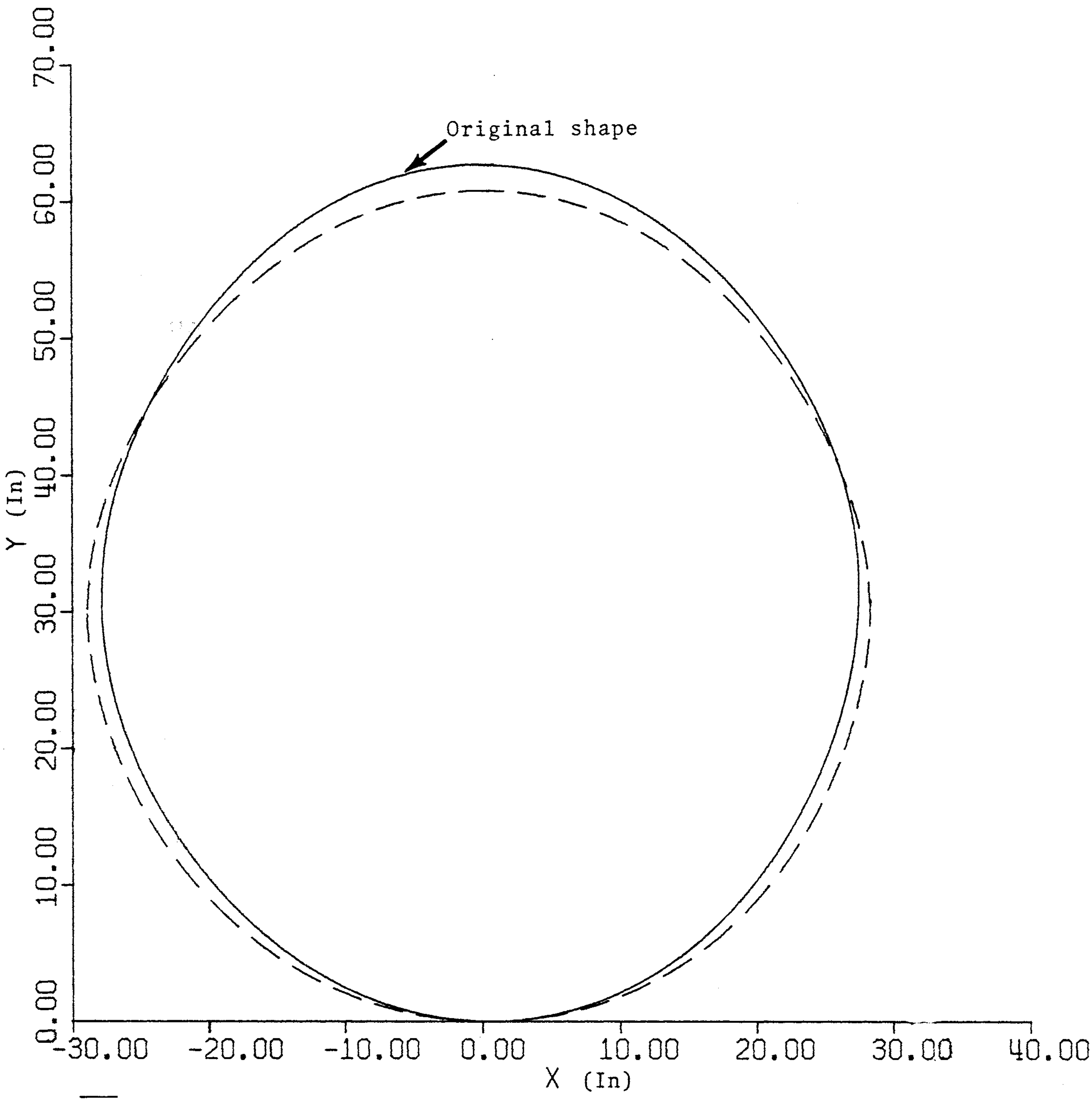


Figure 39. Cross section of steel pipe at upstream ring on June 26, 1974. Depth of fill at 50 feet. Conversion: 1 ft.= 0.3 m; 1 in.= 2.54 cm

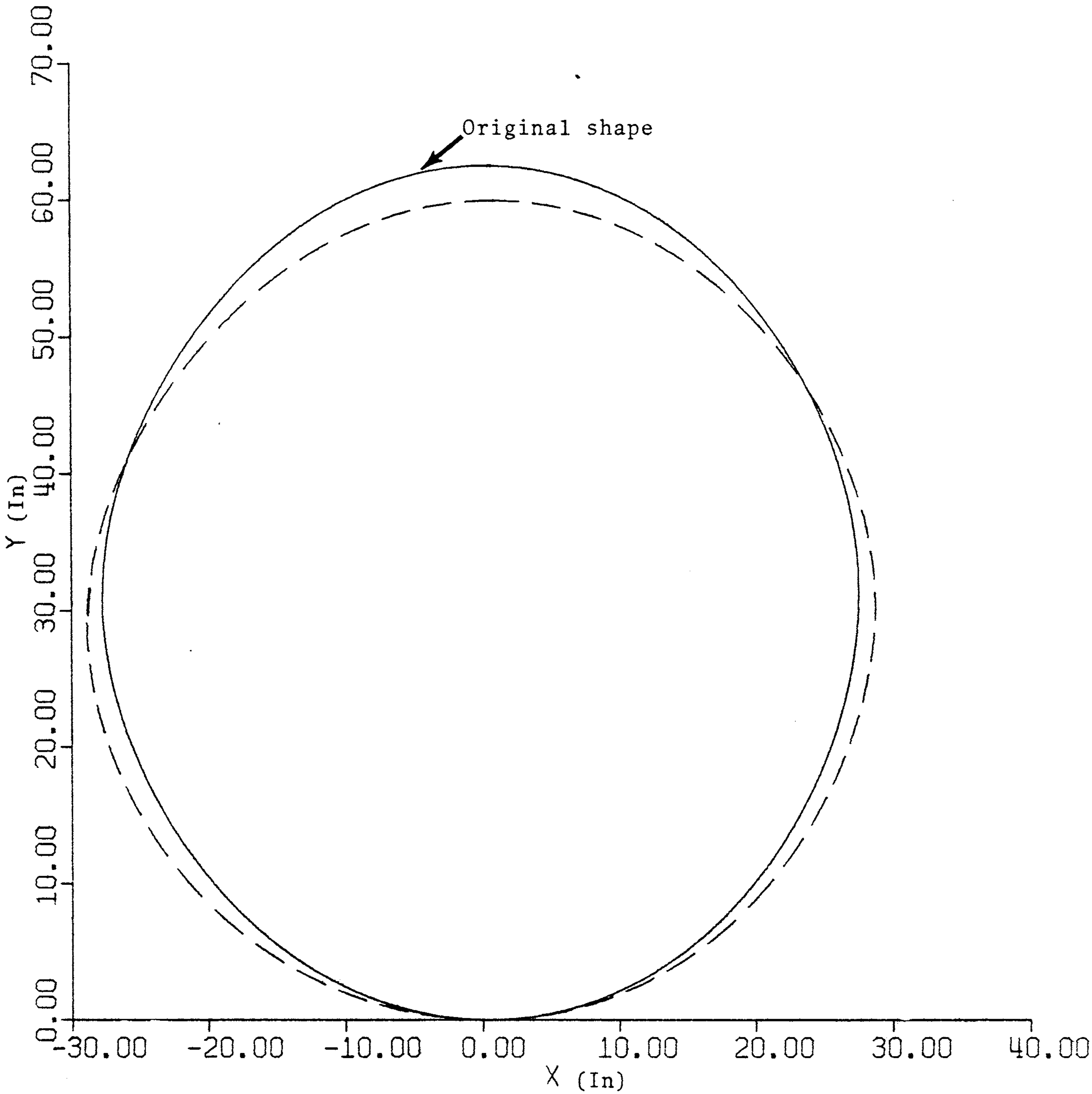


Figure 40. Cross section of steel pipe at downstream ring on June 26, 1974. Depth of fill at 50 feet. Conversion: 1 ft.= 0.3 m; 1 in.= 2.54 cm

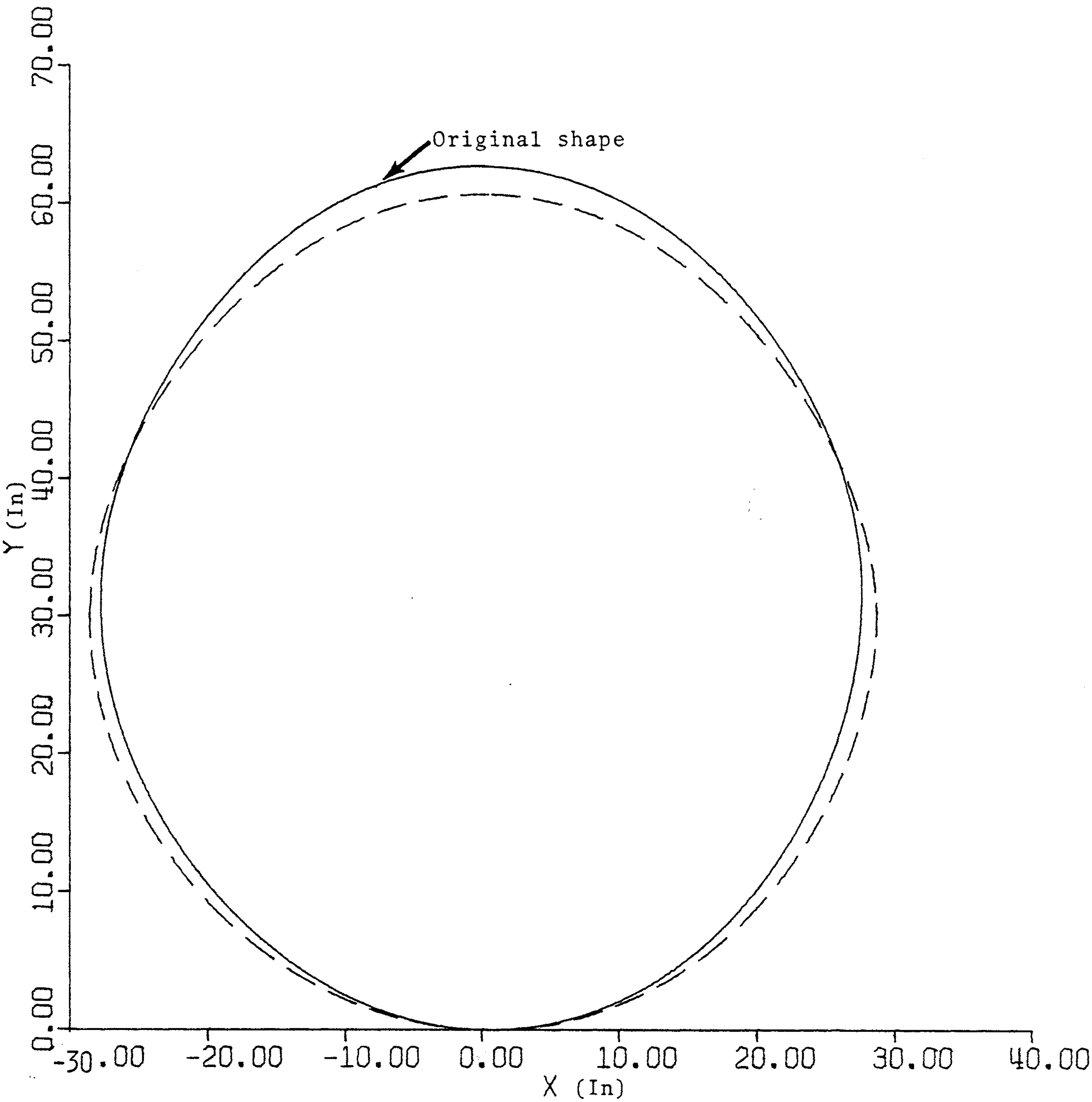


Figure 41. Cross section of steel pipe at upstream ring on October 3, 1974. Depth of fill at 100 feet. Conversion: 1 ft.= 0.3 m; 1 in.= 2.54 cm

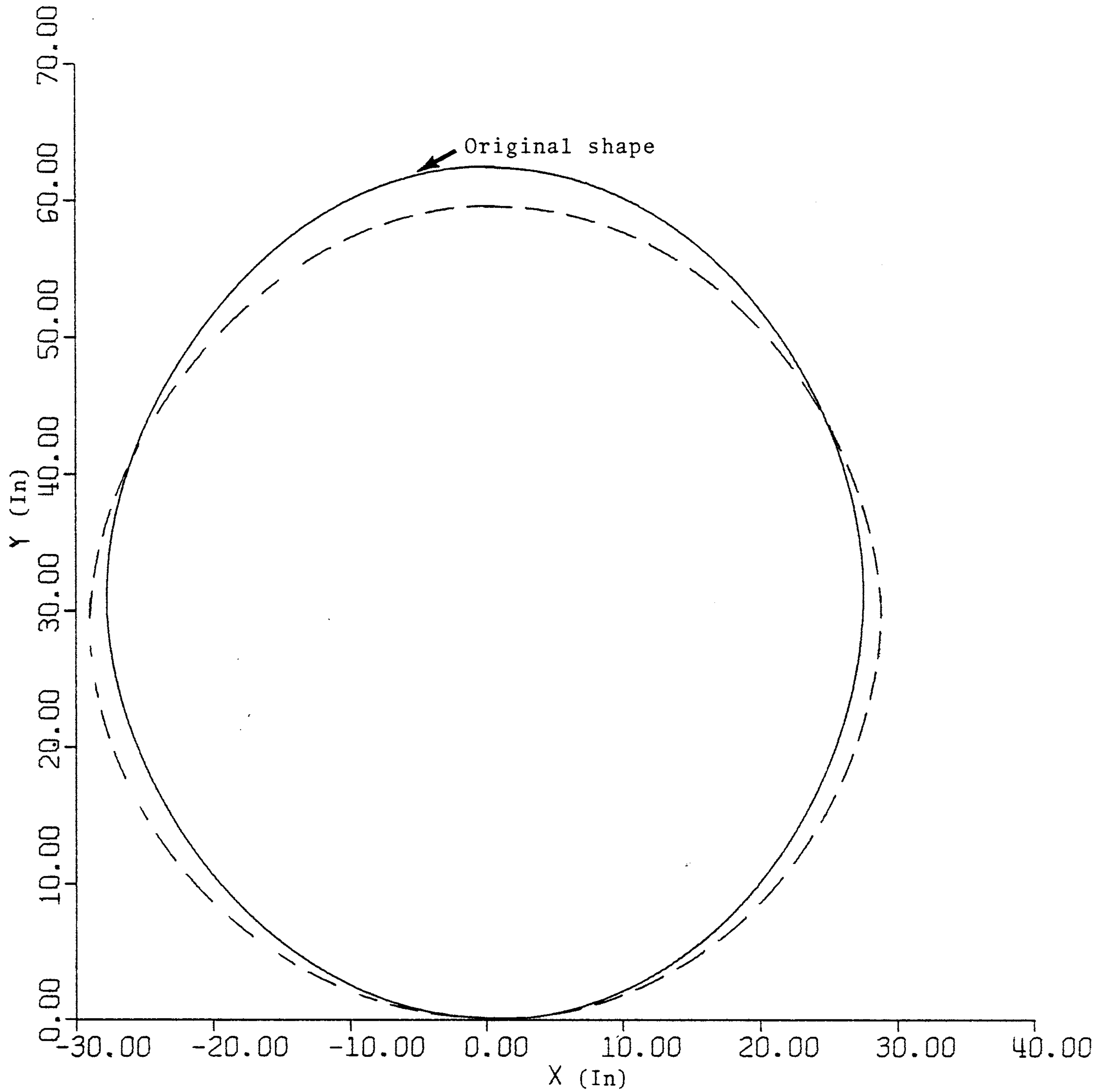


Figure 42. Cross section of steel pipe at downstream ring on October 3, 1974. Depth of fill at 100 feet. Conversion: 1 ft.= 0.3 m; 1 in.= 2.54 cm

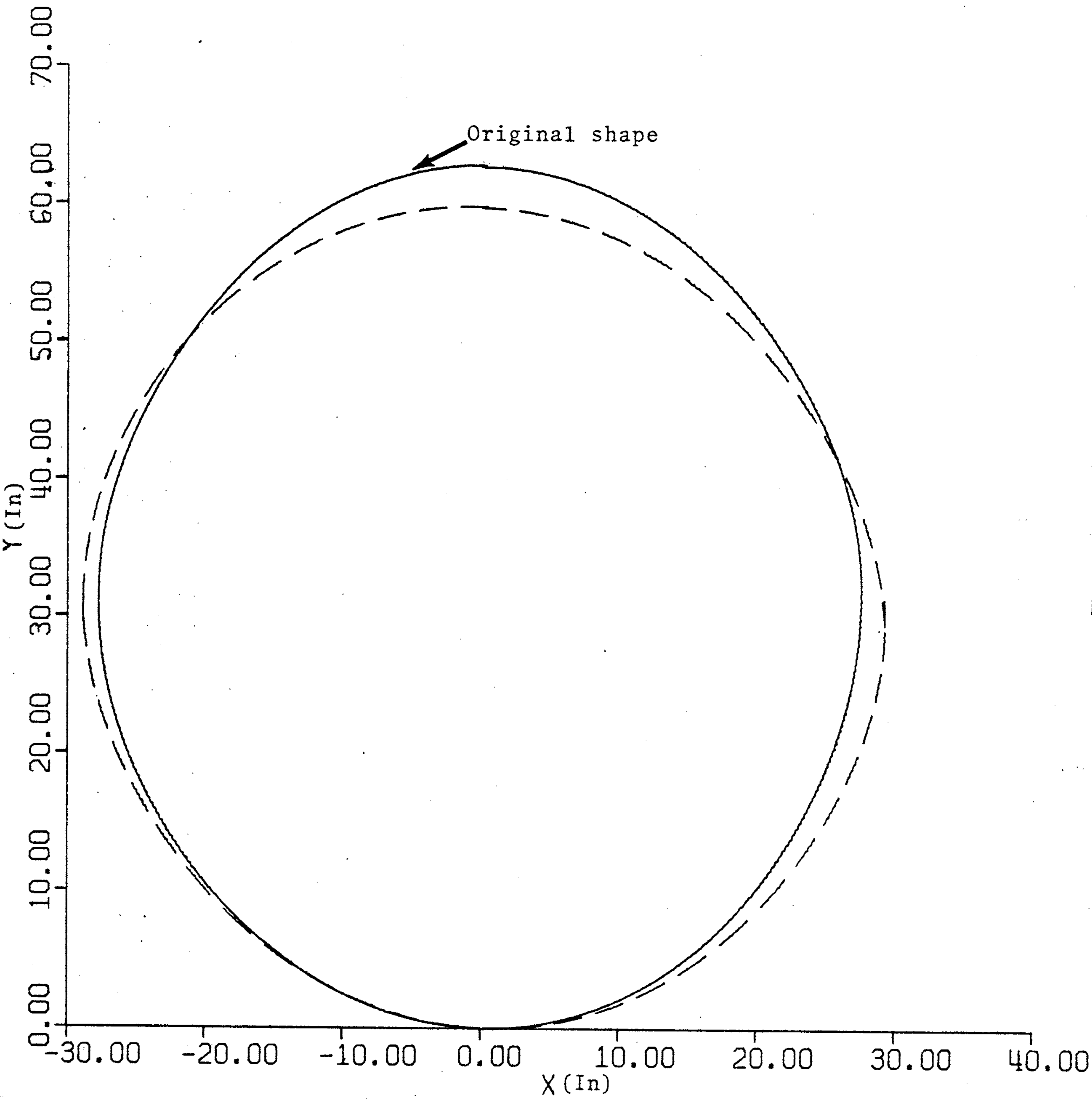


Figure 43. Cross section of steel pipe at upstream ring on February 2, 1975. Depth of fill at 156 feet. Conversion: 1 ft.= 0.3 m; 1 in.= 2.54 cm

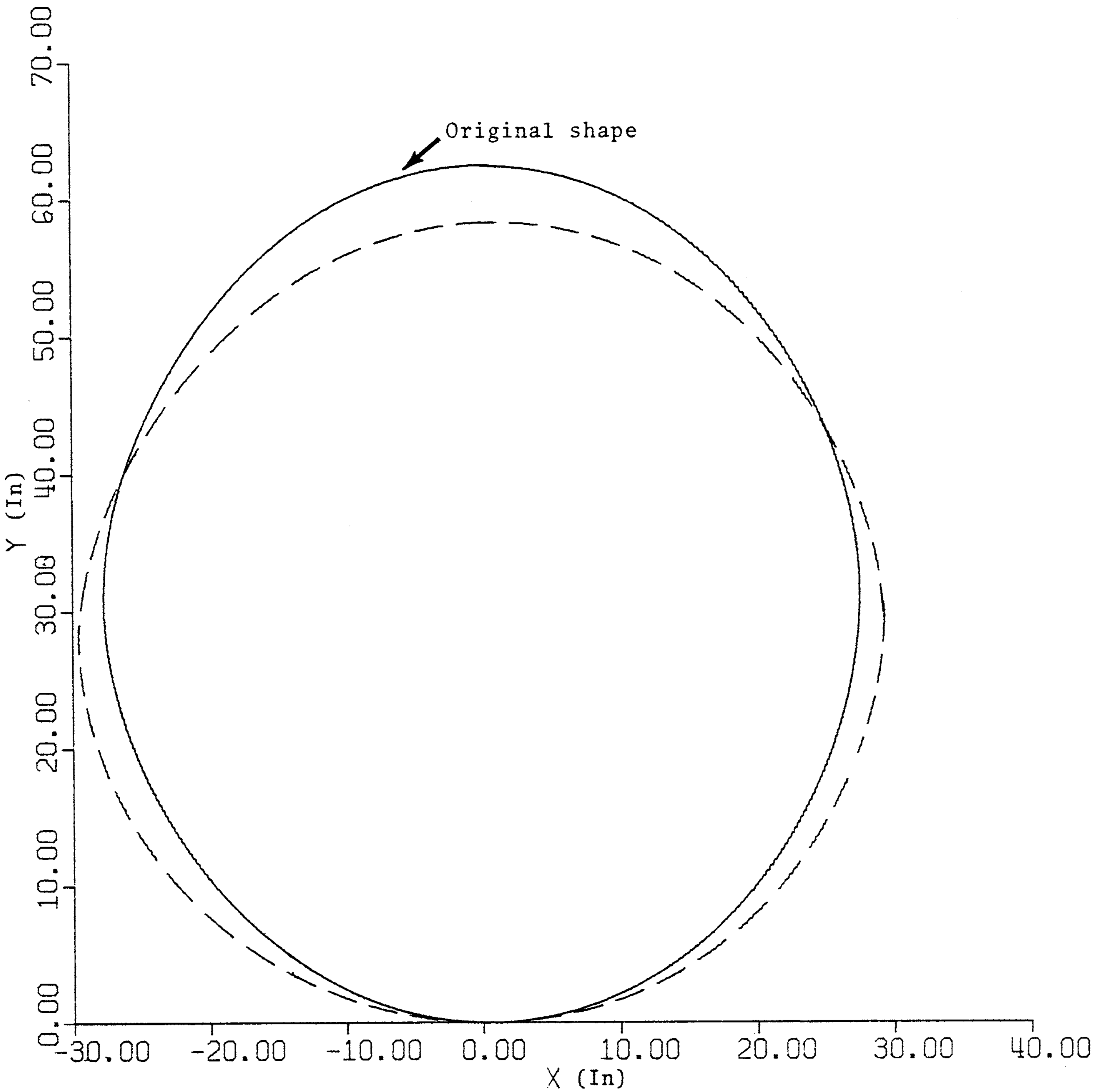


Figure 44. Cross section of steel pipe at downstream ring on February 2, 1975. Depth of fill at 156 feet. Conversion: 1 ft.= 0.3 m; 1 in.= 2.54 cm

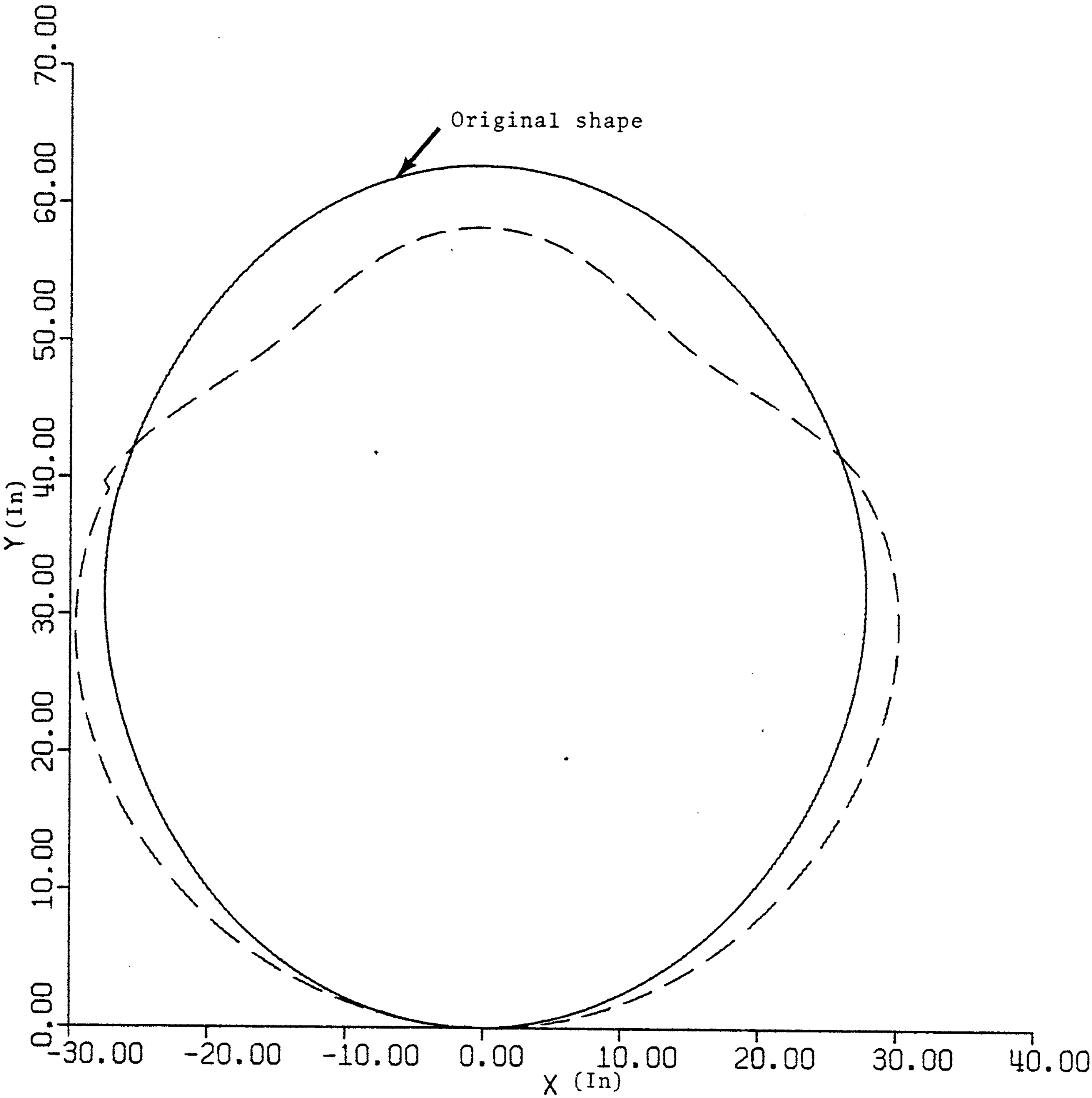


Figure 45. Cross section of steel pipe at upstream ring on February 12, 1976. Depth of fill at 256 feet. Conversion: 1 ft.= 0.3 m; 1 in.= 2.54 cm

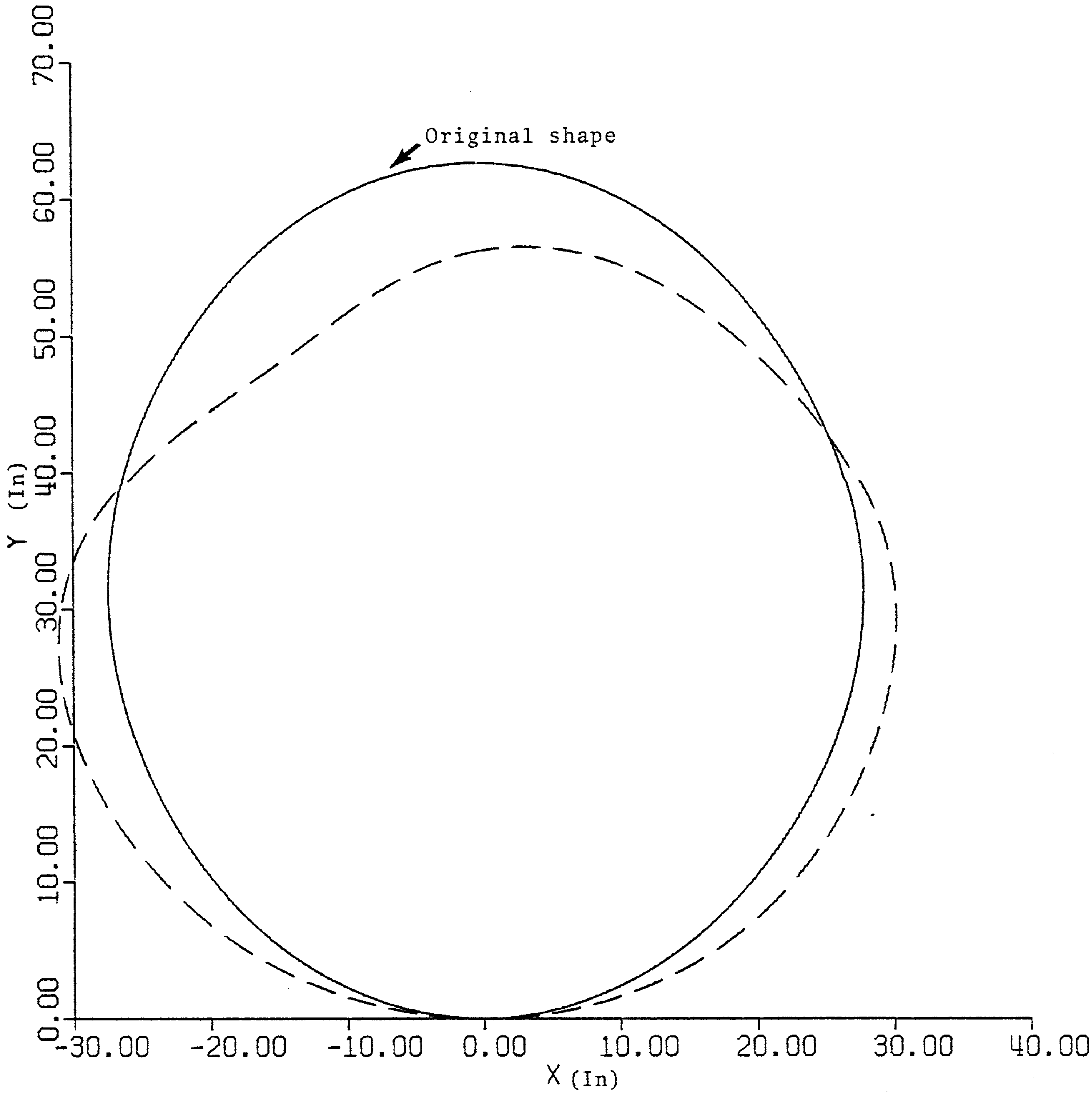


Figure 46. Cross section of steel pipe at downstream ring on February 12, 1976. Depth of fill at 256 feet. Conversion: 1 ft.= 0.3 m; 1 in.= 2.54 cm

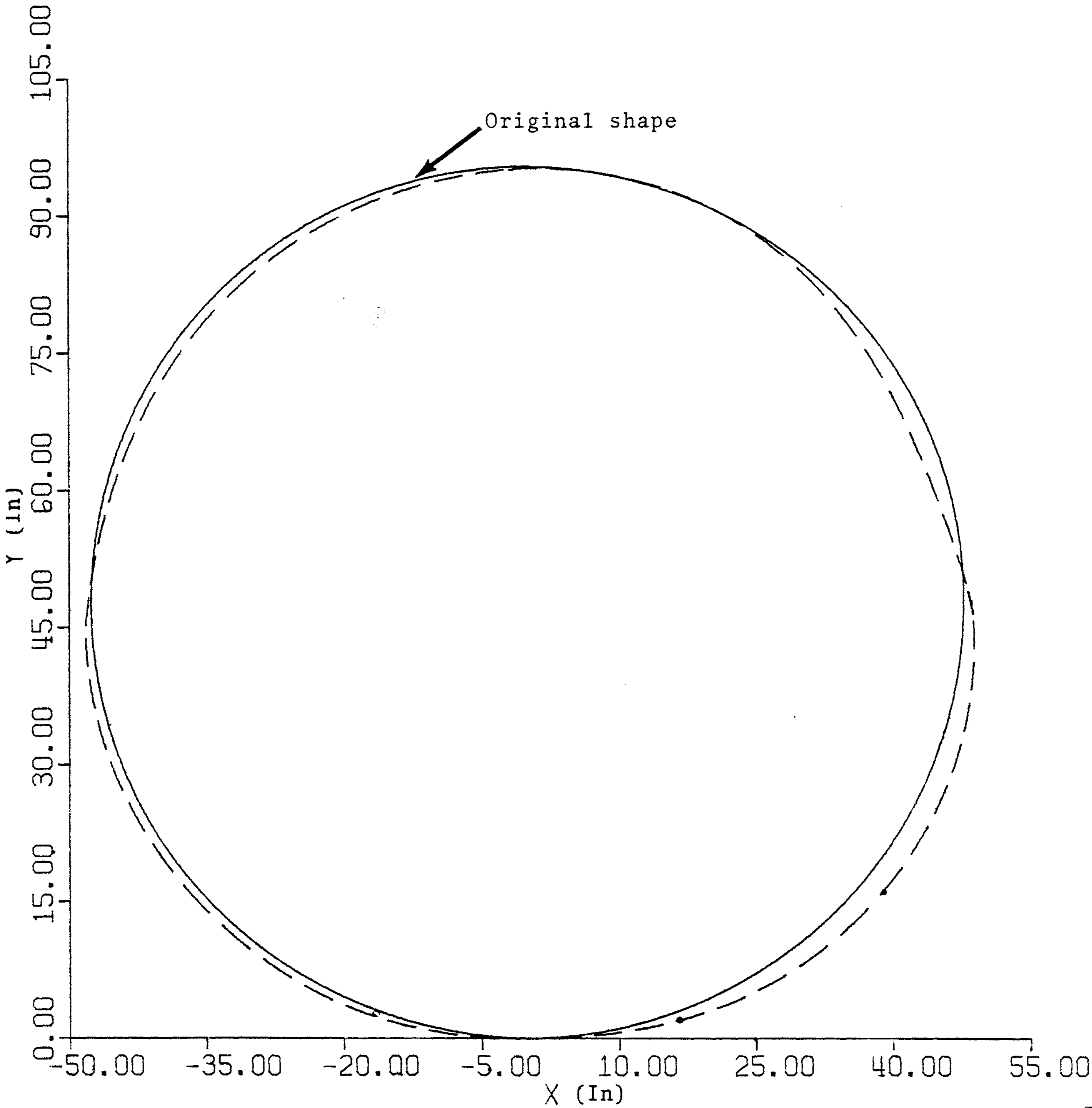


Figure 47. Cross section of concrete pipe at upstream ring on December 4, 1974. Depth of fill at 32 feet. Conversion: 1 ft.= 0.3 m; 1 in.= 2.54 cm

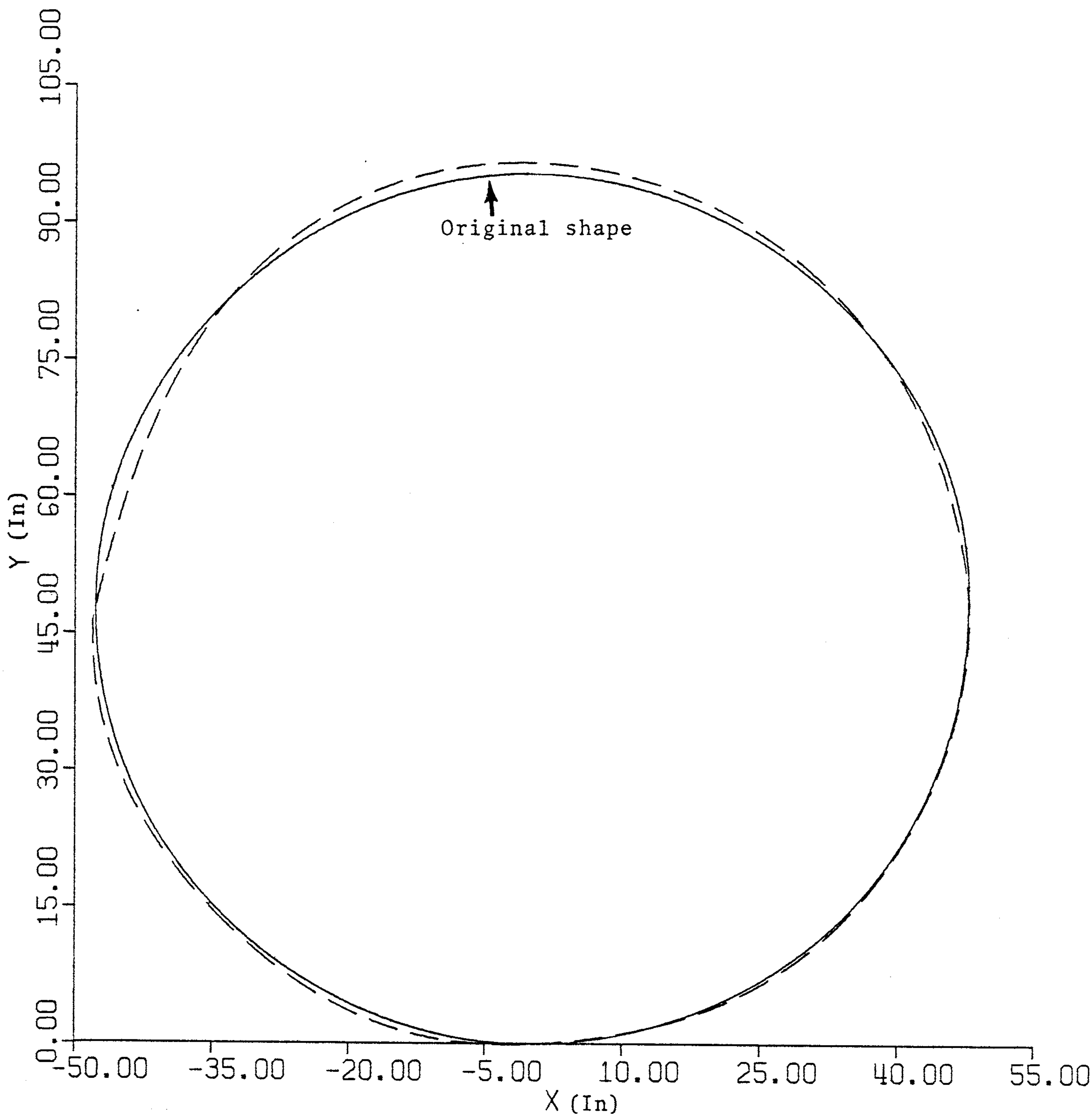


Figure 48. Cross section of concrete pipe at downstream ring on December 4, 1974. Depth of fill at 32 feet. Conversion: 1 ft.= 0.3 m; 1 in.= 2.54 cm

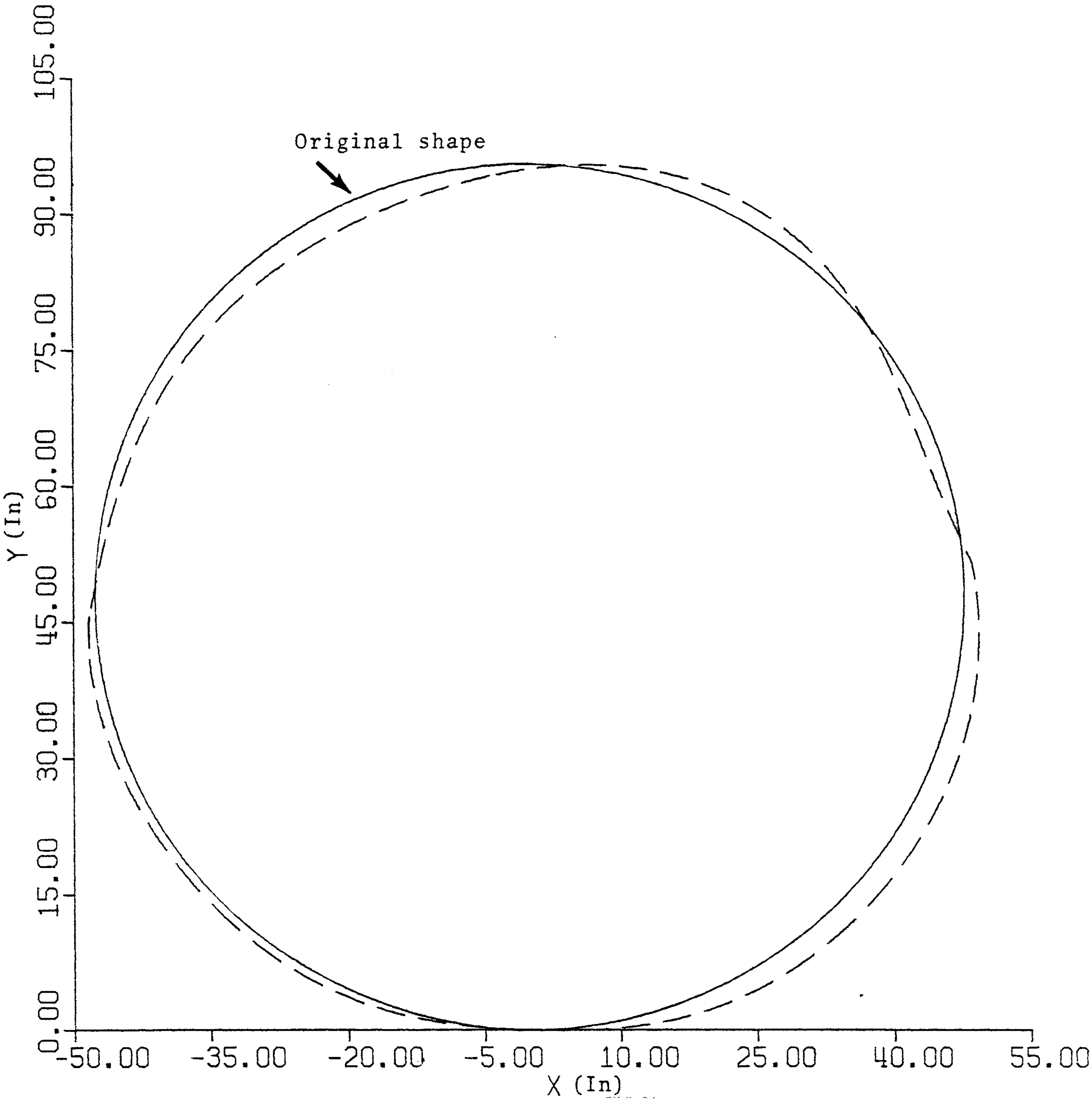


Figure 49. Cross section of concrete pipe at upstream ring on June 3, 1975. Depth of fill at 87 feet. Conversion: 1 ft.= 0.3 m; 1 in.= 2.54 cm

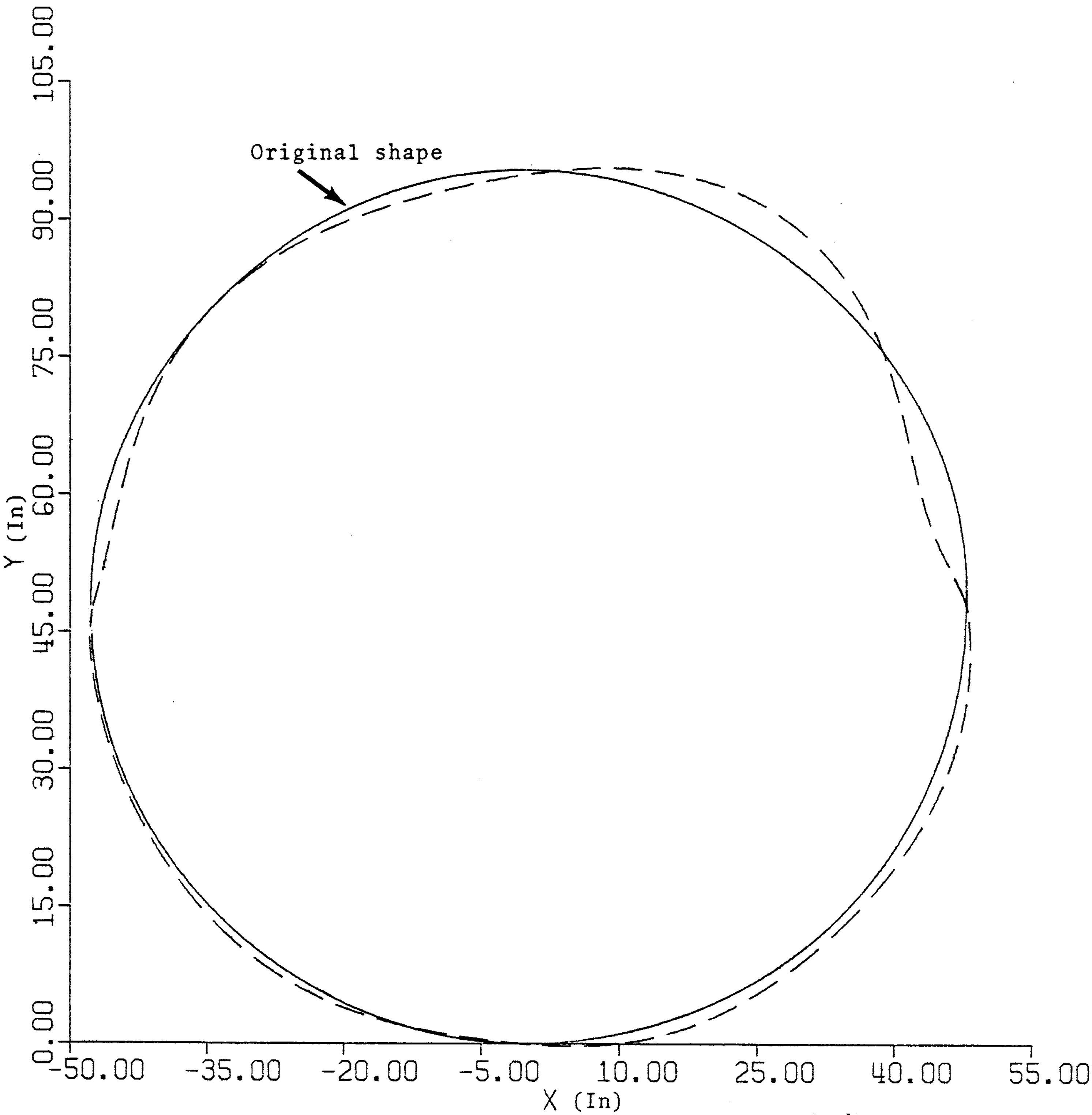


Figure 50. Cross section of concrete pipe at downstream ring on June 3, 1975. Depth of fill at 87 feet. Conversion: 1 ft.= 0.3 m; 1 in.= 2.54 cm

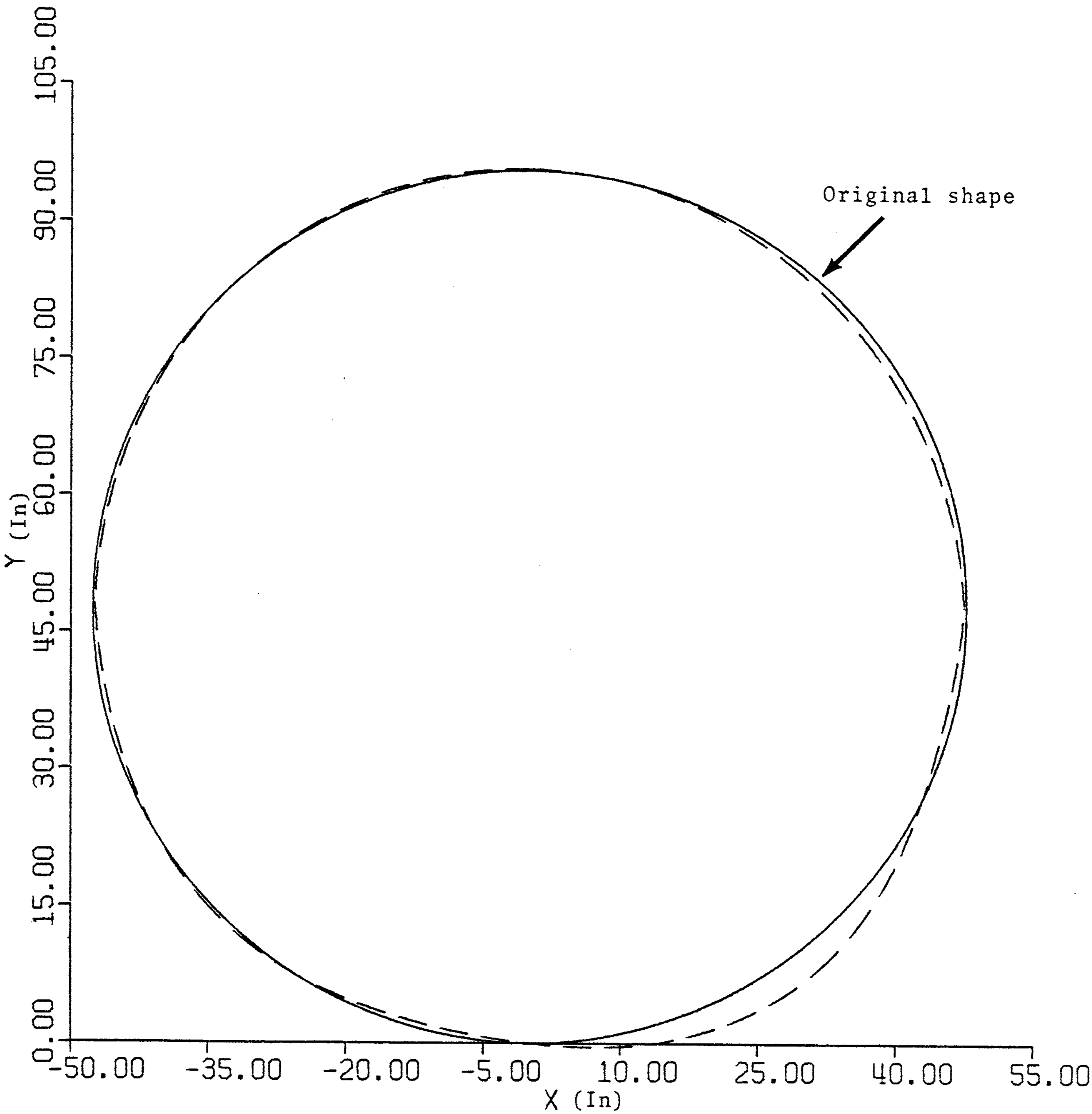


Figure 51. Cross section of concrete pipe at upstream ring on February 12, 1976. Depth of fill at 125 feet. Conversion: 1 ft.= 0.3 m; 1 in.= 2.54 cm

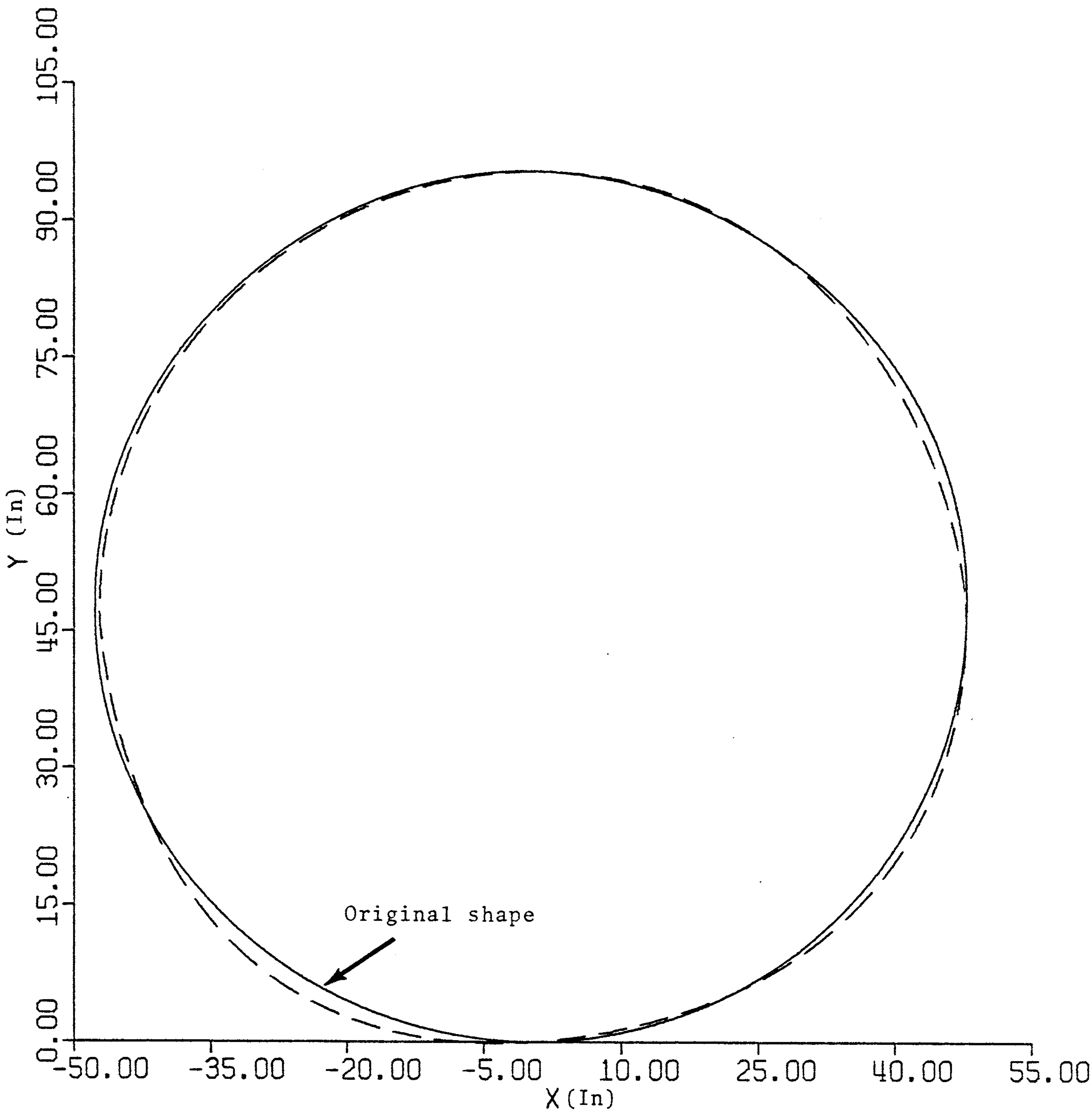


Figure 52. Cross section of concrete pipe at downstream ring on February 12, 1976. Depth of fill at 125 feet. Conversion: 1 ft.= 0.3 m; 1 in.= 2.54 cm

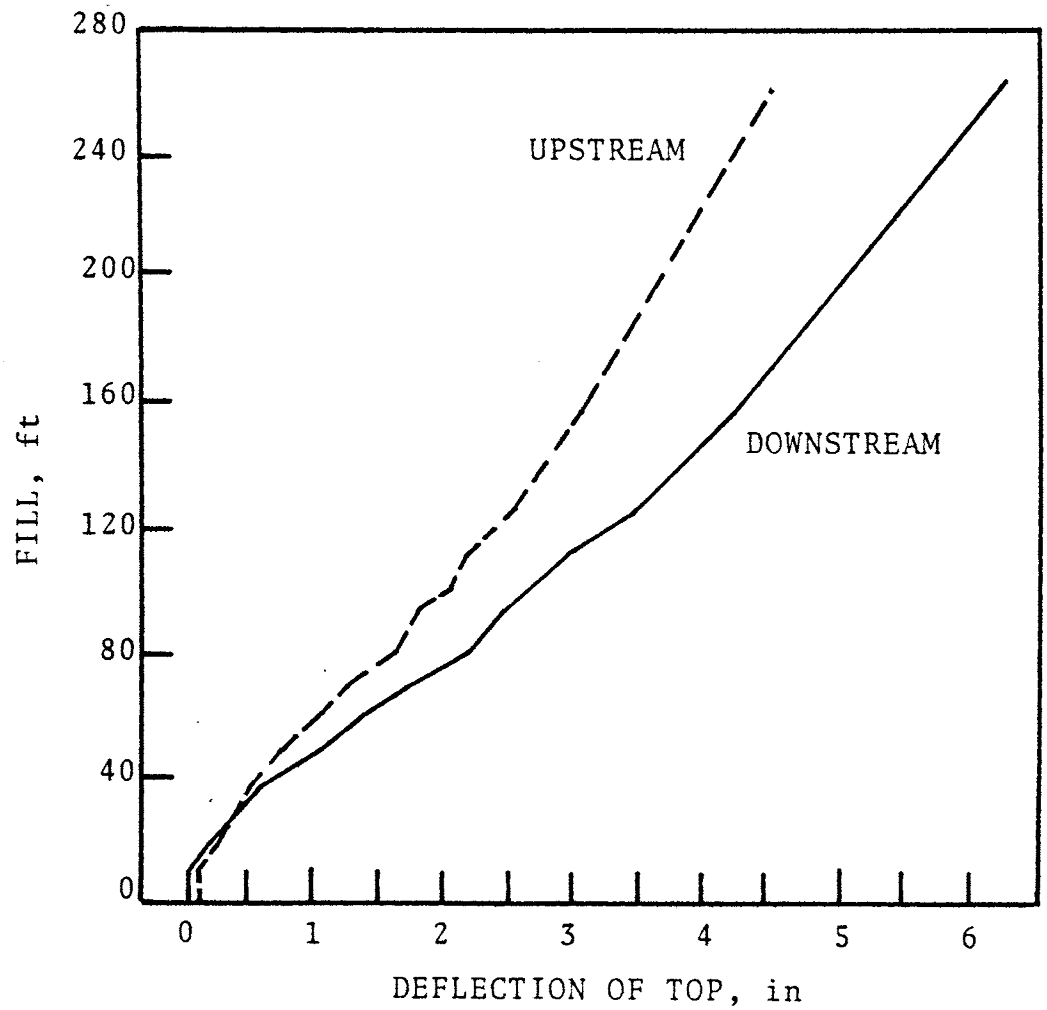
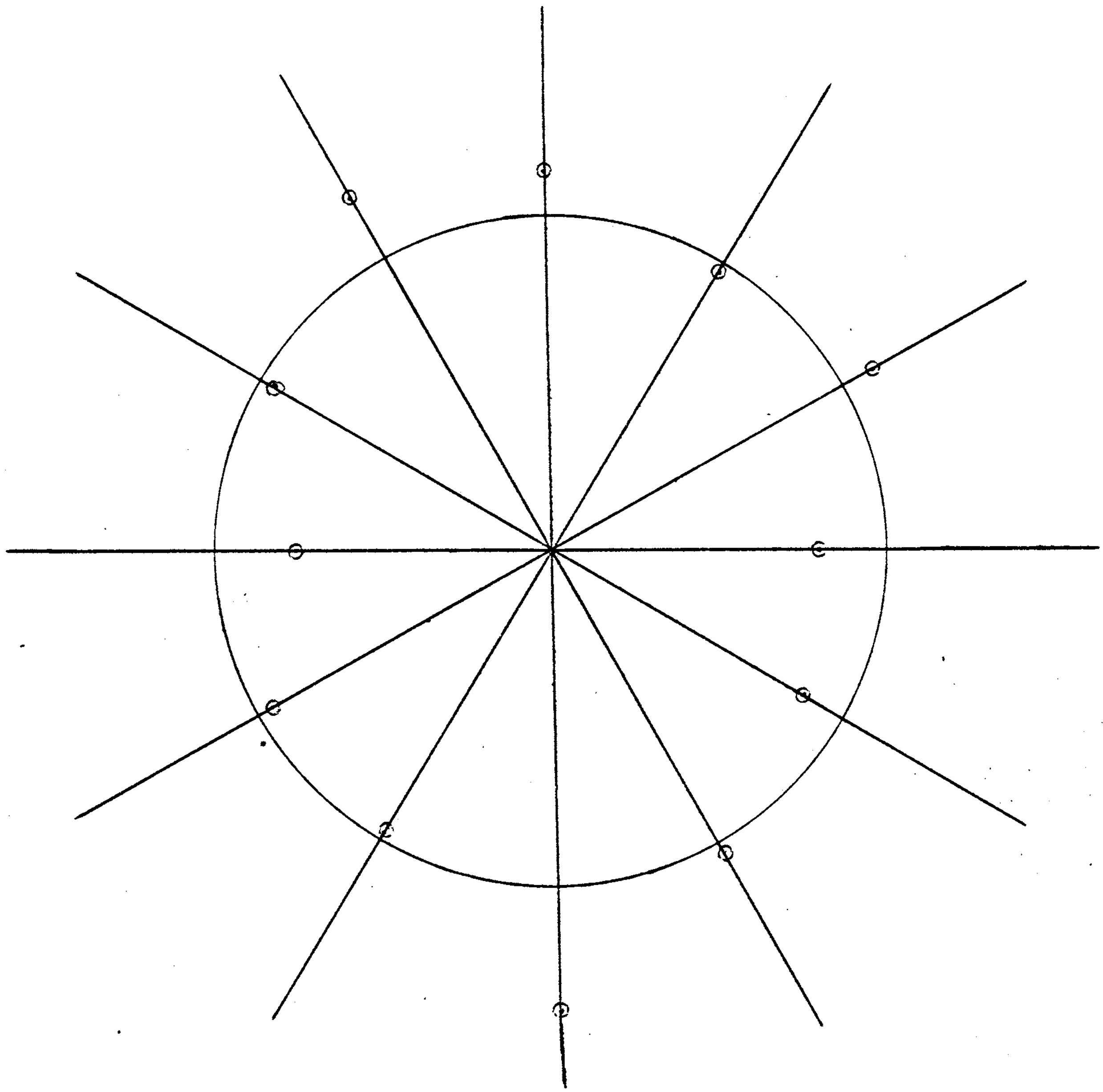


Figure 53. Crown deflection versus fill depth.
Conversion: 1 ft = 0.3 m;
1 in = 2.54 cm



Scale: 1" = 4 x 10^{-3} in.$^{-1}$

Figure 54. Distribution of normalized bending moments (M/EI) at upstream ring of steel pipe. Depth of fill at 50 feet. Conversion: 1 ft.= 0.3 m; 1 in. 2.54 cm

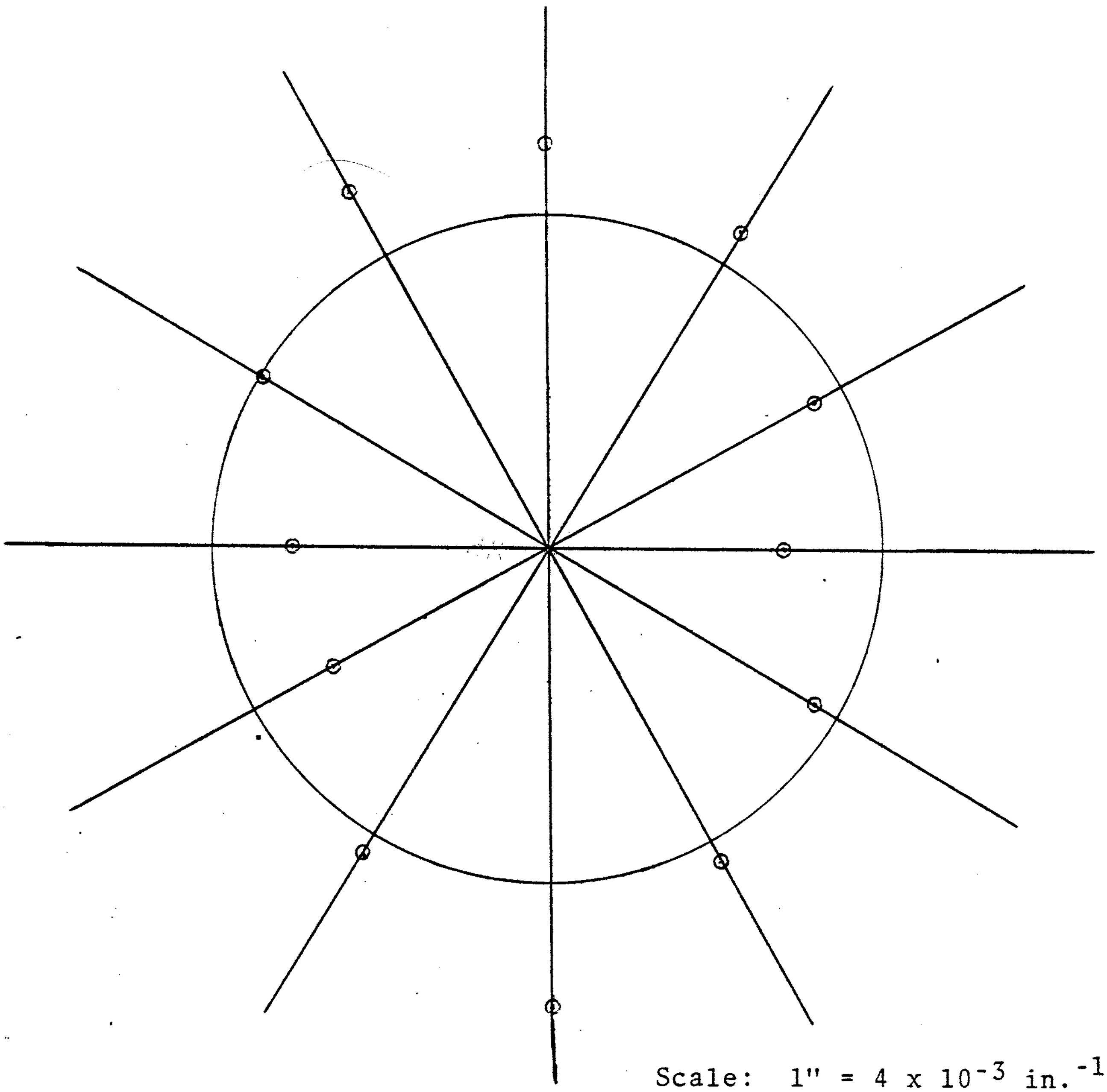


Figure 55. Distribution of normalized bending moments (M/EI) at downstream ring of steel pipe. Depth of fill at 50 feet. Conversion: 1 ft.= 0.3 m; 1 in.= 2.54 cm

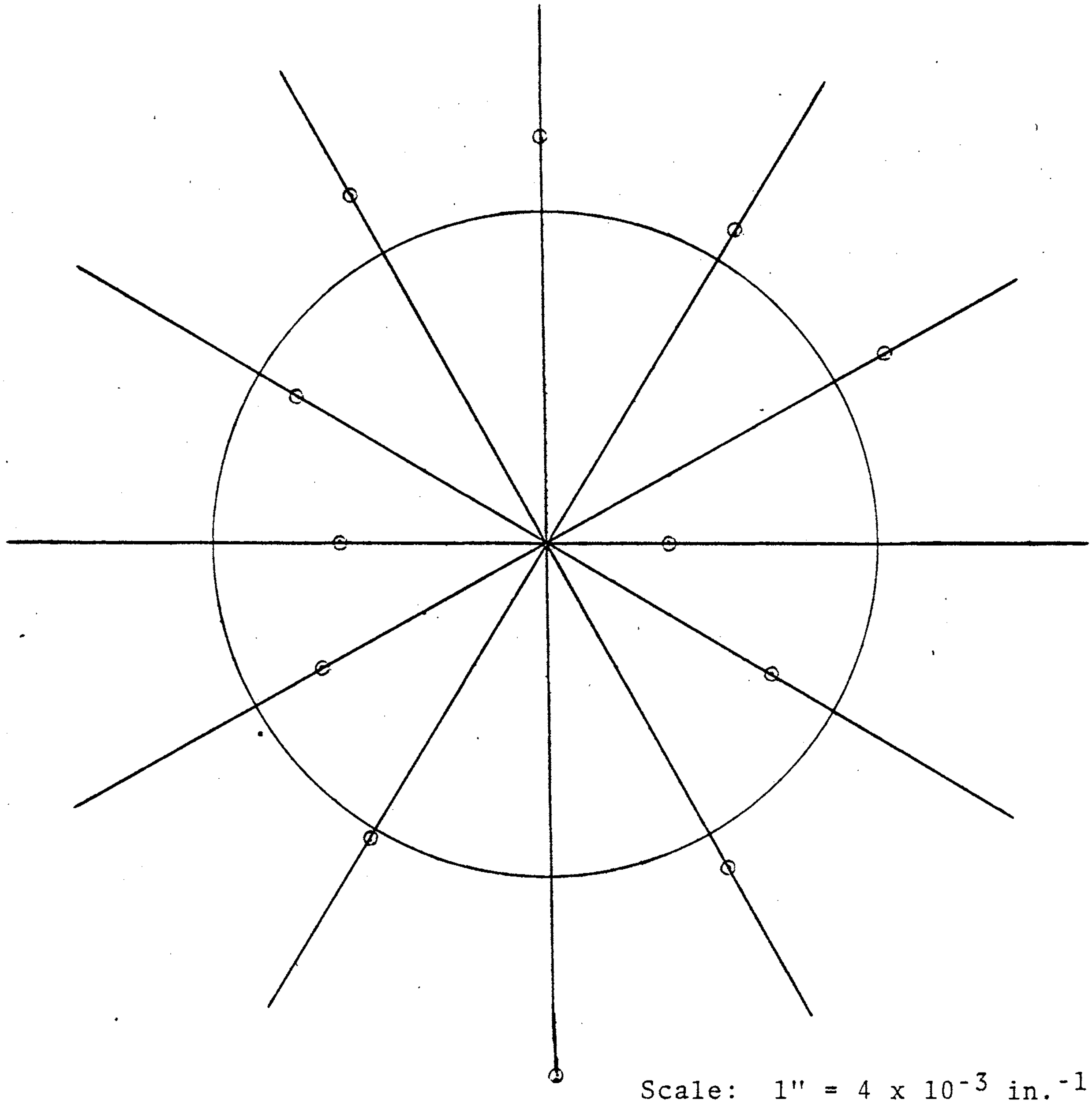
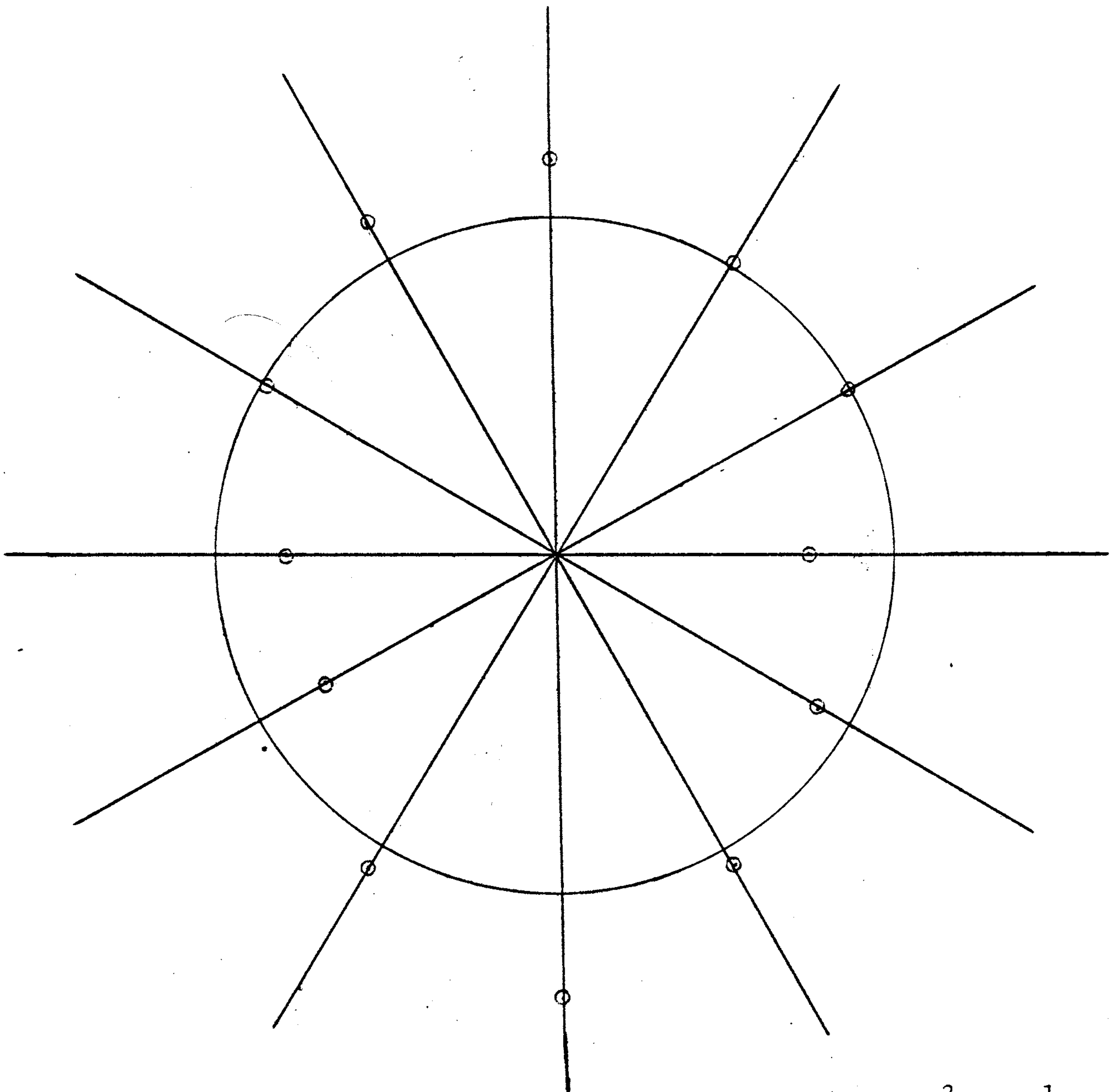


Figure 56. Distribution of normalized bending moments (M/EI) at upstream ring of steel pipe. Depth of fill at 100 feet. Conversion: 1 ft.= 0.3 m; 1 in.= 2.54 cm



Scale: 1" = 1 x 10^{-2} in.$^{-1}$

Figure 57. Distribution of normalized bending moments (M/EI) at downstream ring of steel pipe. Depth of fill at 100 feet. Conversion: 1 ft.= 0.3 m; 1 in.= 2.54 cm

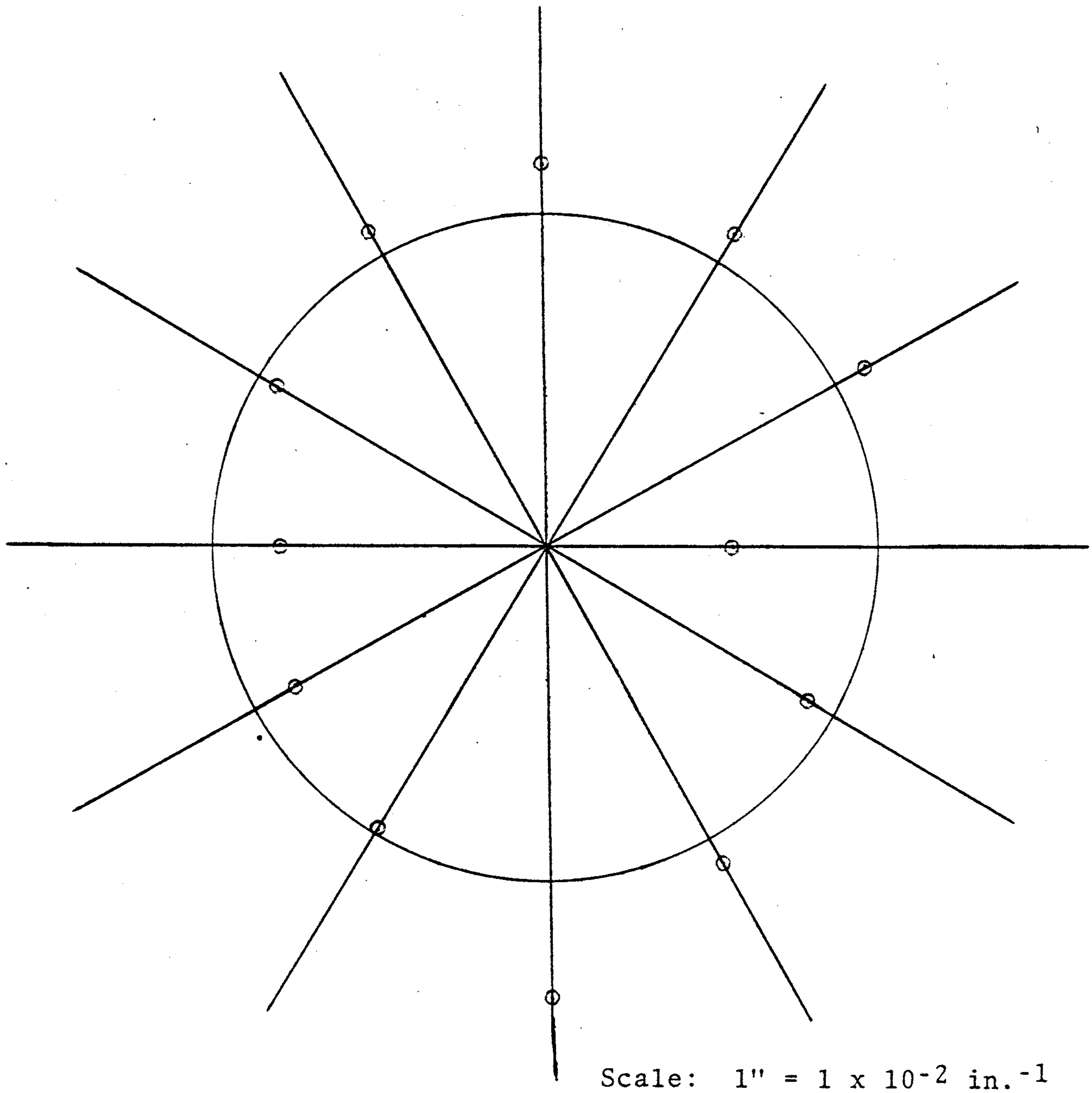
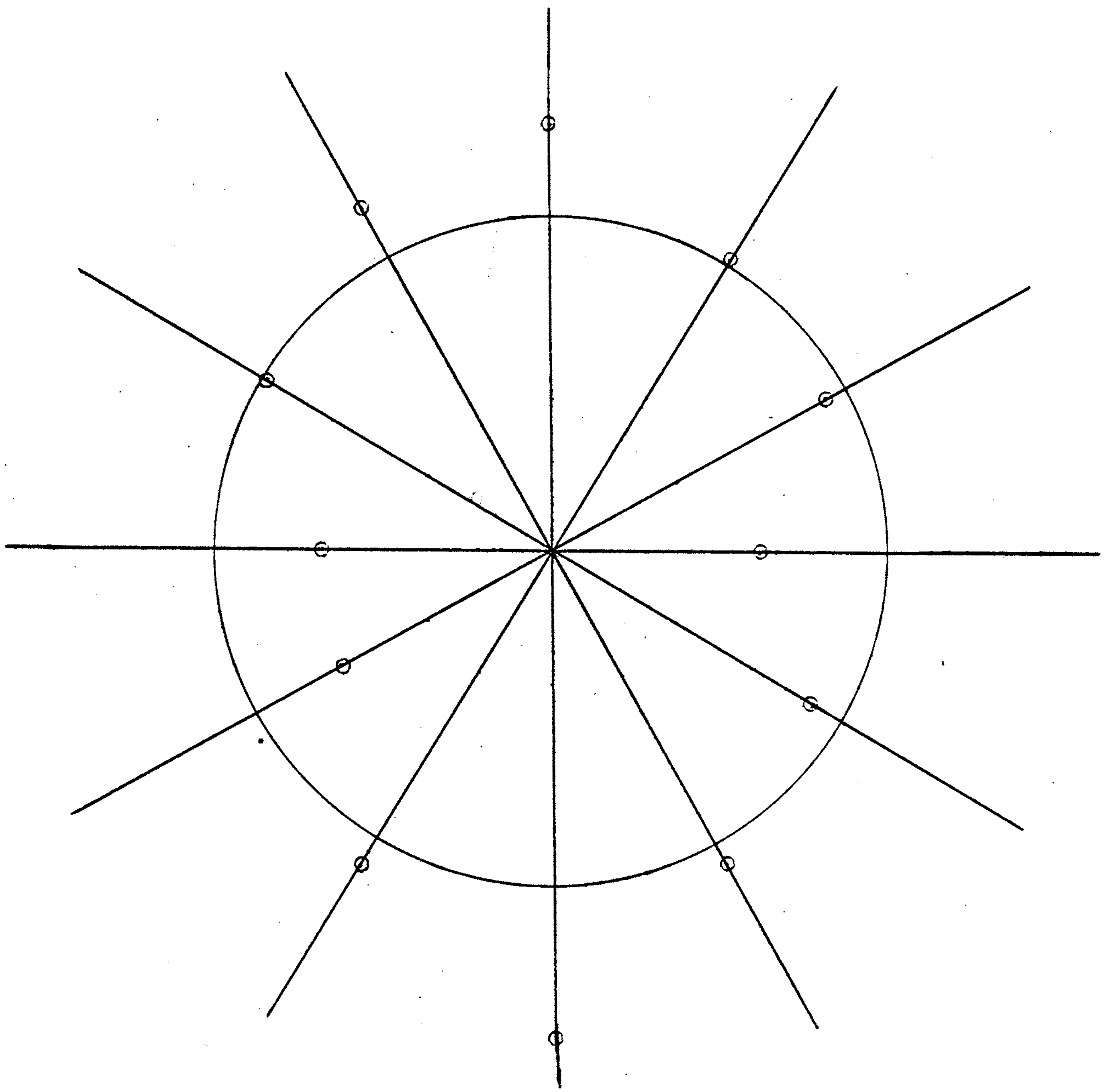
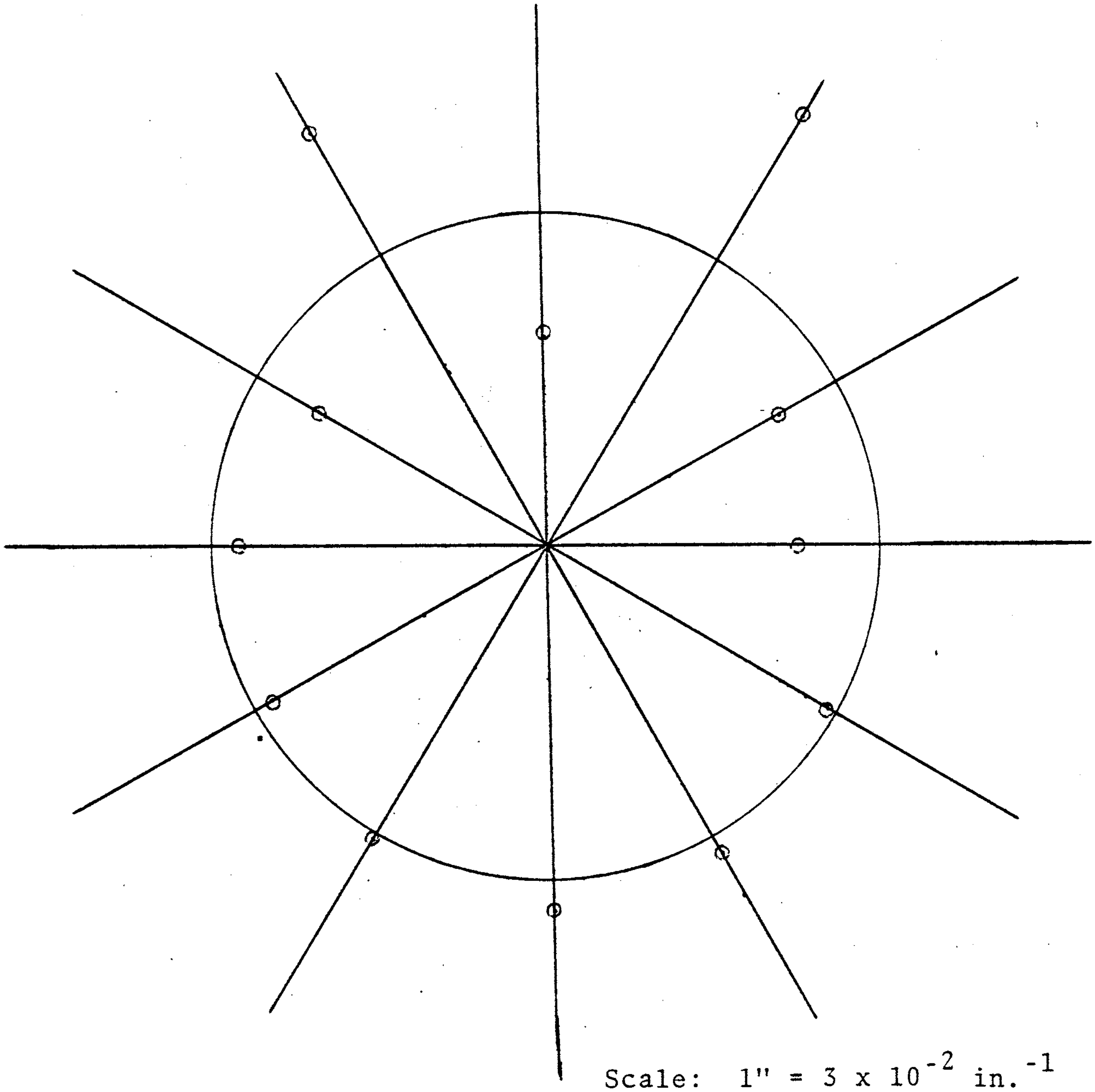


Figure 58. Distribution of normalized bending moments (M/EI) at upstream ring of steel pipe. Depth of fill at 156 feet. Conversion: 1 ft.= 0.3 m;
1 in.= 2.54 cm



Scale: 1" = 1 x 10^{-2} in.$^{-1}$

Figure 59. Distribution of normalized bending moments (M/EI) at downstream ring of steel pipe. Depth of fill at 156 feet. Conversion: 1 ft.= 0.3 m; 1 in.= 2.54 cm



Scale: 1" = 3 x 10^{-2} in.$^{-1}$

Figure 60. Distribution of normalized bending moments (M/EI) at upstream ring of steel pipe. Depth of fill at 256 feet. Conversion: 1 ft.= 0.3 m; 1 in.= 2.54 cm

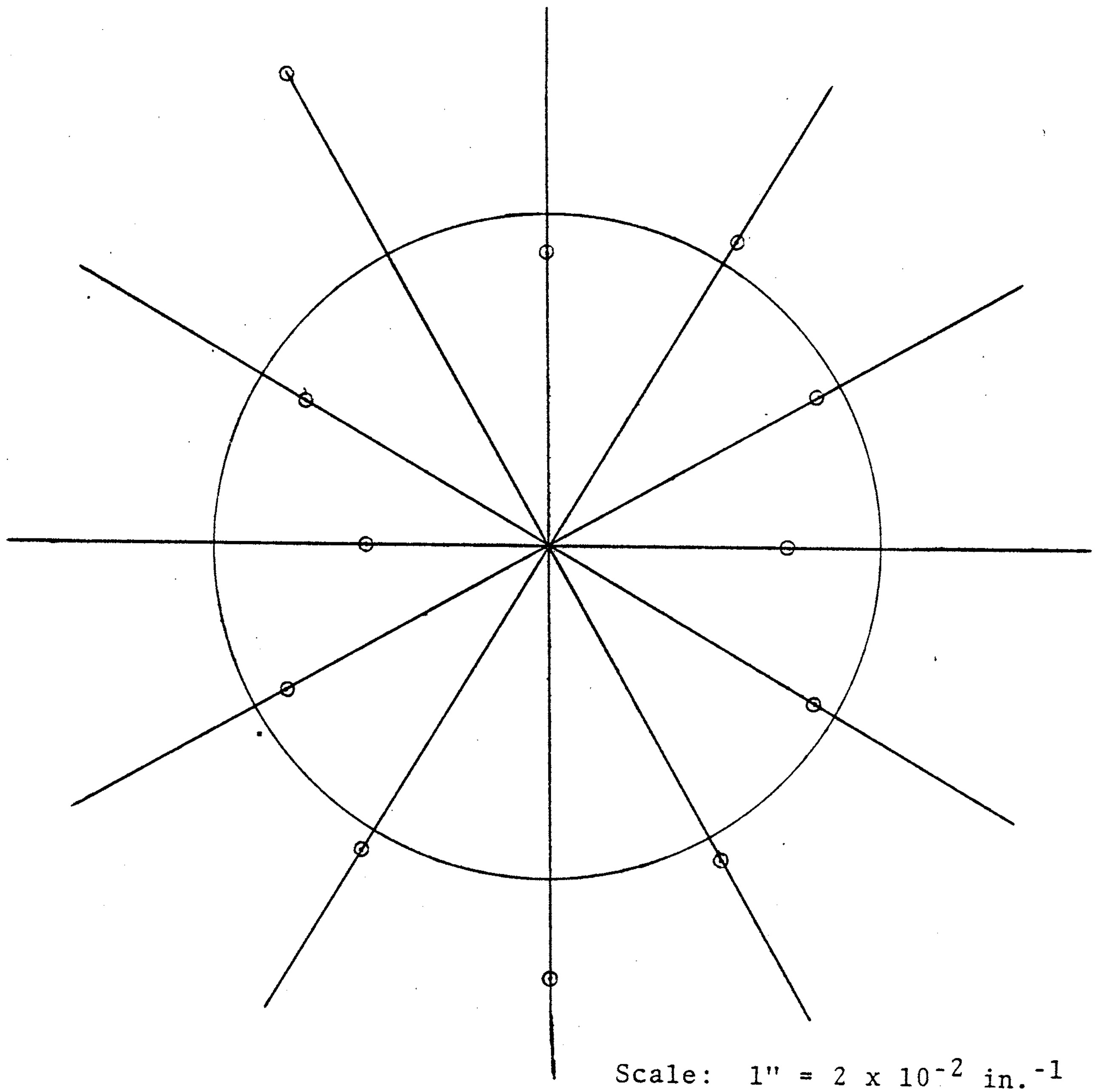
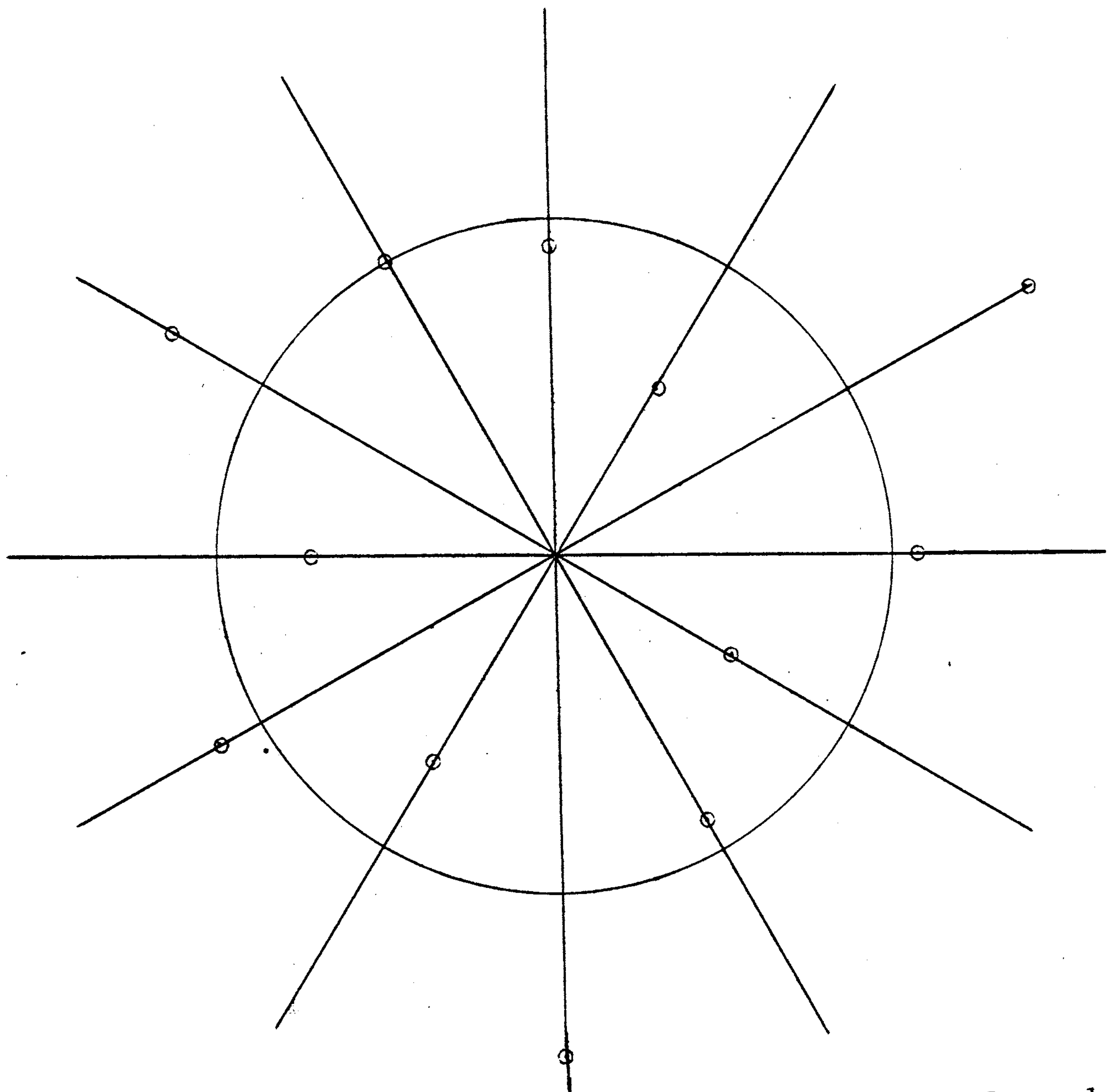


Figure 61. Distribution of normalized bending moments (M/EI) at downstream ring of steel pipe. Depth of fill at 256 feet. Conversion: 1 ft.= 0.3 m; 1 in.= 2.54 cm



Scale: 1" = 2 x 10^{-3} in.$^{-1}$

Figure 62. Distribution of normalized bending moments (M/EI) at upstream ring of concrete pipe. Depth of fill at 32 feet. Conversion: 1 ft.= 0.3 m; 1 in.= 2.54 cm

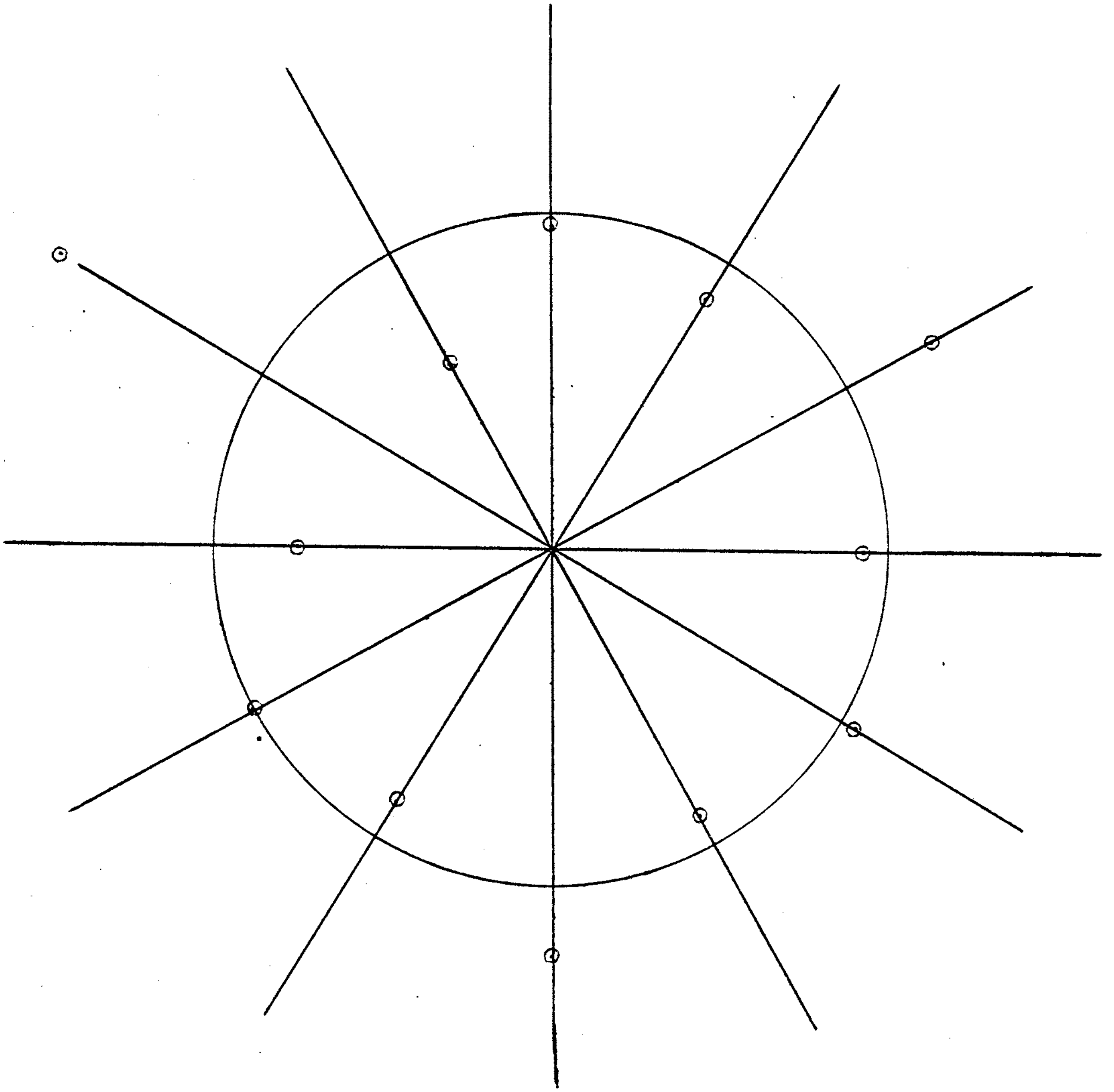
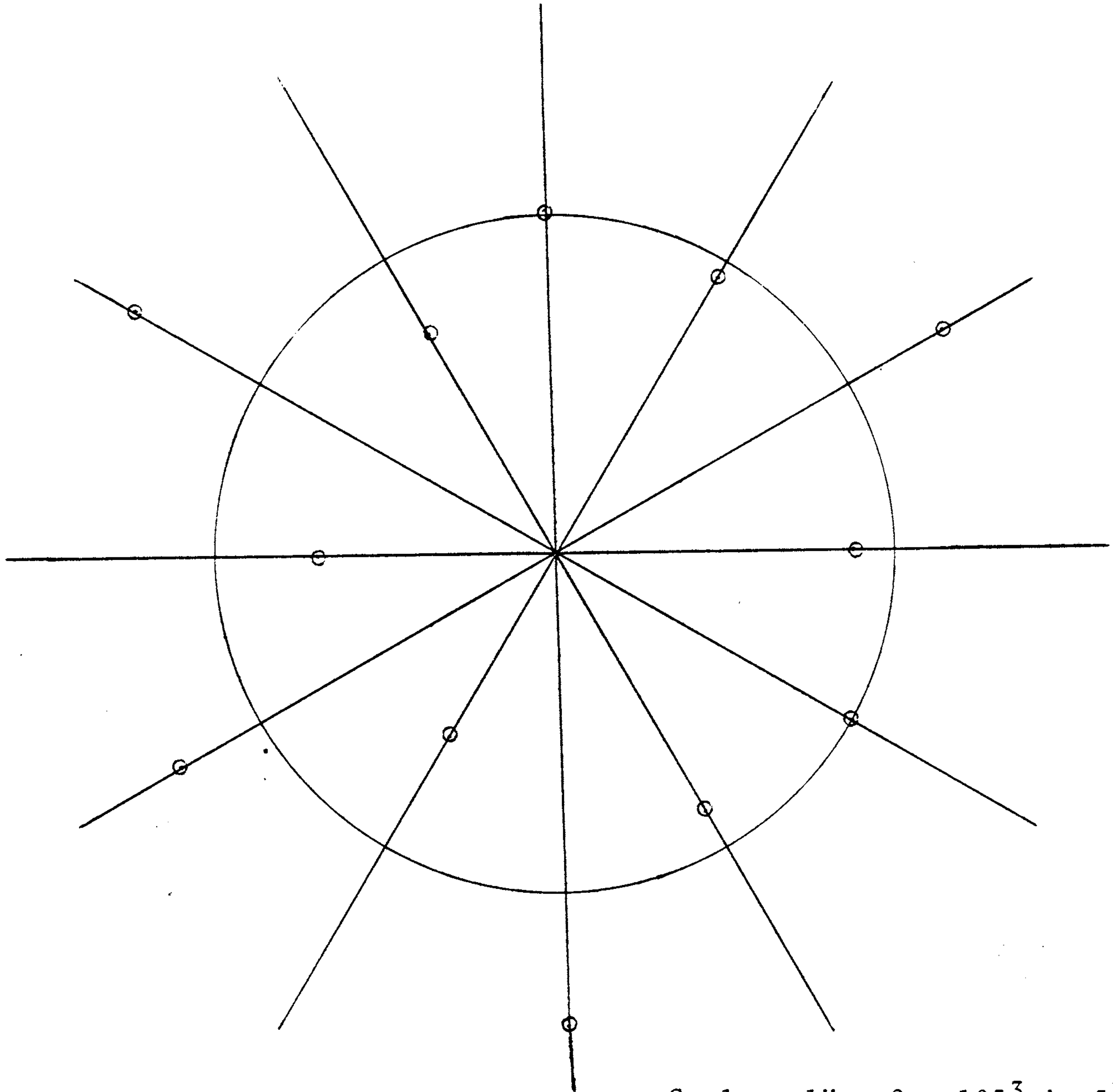


Figure 63. Distribution of normalized bending moments (M/EI) at downstream ring of concrete pipe. Depth of fill at 32 feet. Conversion: 1 ft.= 0.3 m; 1 in.= 2.54 cm



Scale: 1" = 2 x 10^{-3} in.$^{-1}$

Figure 64. Distribution of normalized bending moments (M/EI) at upstream ring of concrete pipe. Depth of fill at 87 feet. Conversion: 1 ft.= 0.3 m; 1 in.= 2.54 cm

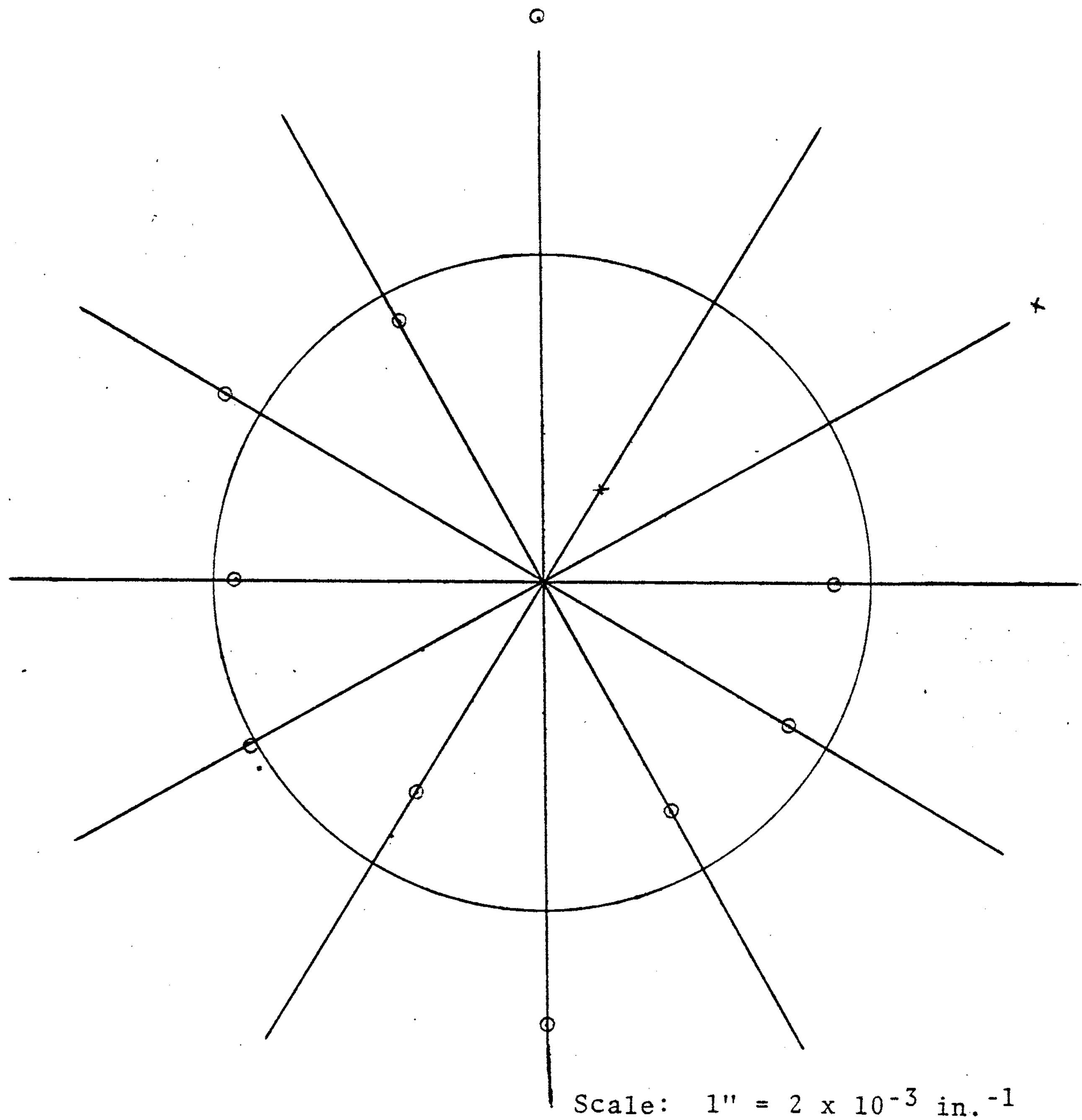


Figure 65. Distribution of normalized bending moments (M/EI) at downstream ring of concrete pipe. Depth of fill at 87 feet. Conversion: 1 ft.= 0.3 m; 1 in.= 2.54 cm

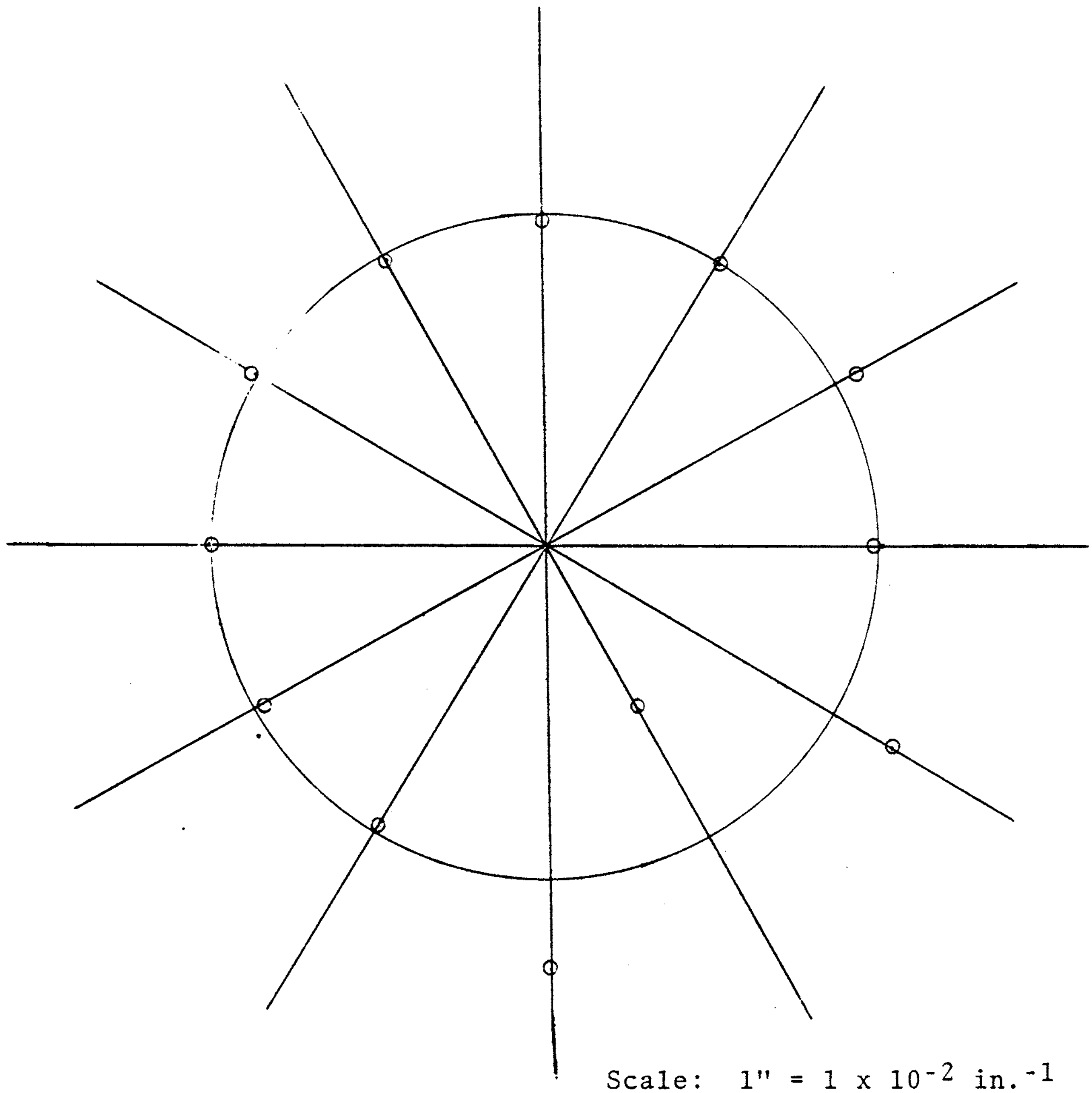


Figure 66. Distribution of normalized bending moments (M/EI) at upstream ring of concrete pipe. Depth of fill at 125 feet. Conversion: 1 ft. = 0.3 m; 1 in. = 2.54 cm

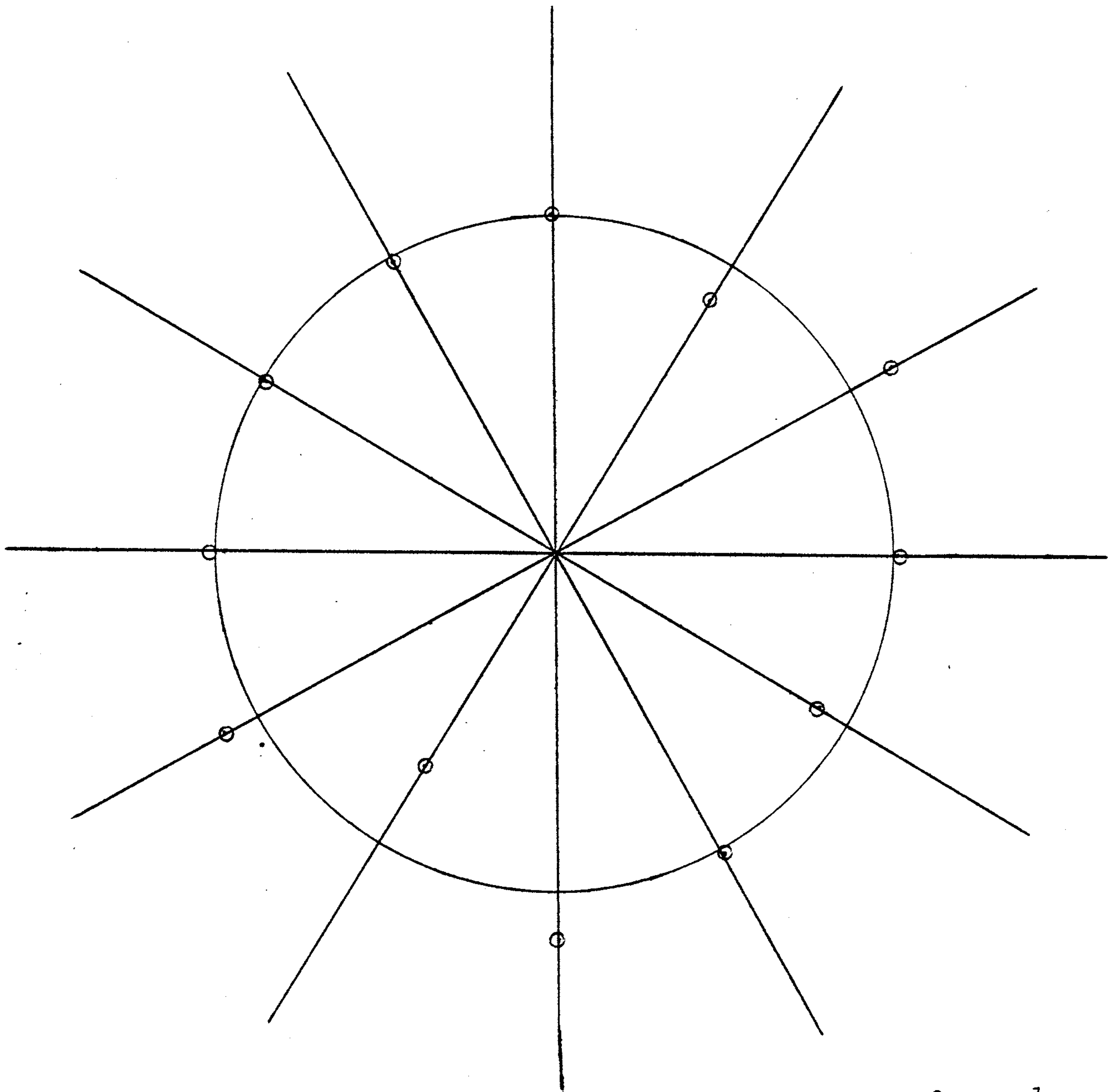


Figure 67. Distribution of normalized bending moments (M/EI) at downstream ring of concrete pipe. Depth of fill at 125 feet. Conversion: 1 ft.= 0.3 m; 1 in.= 2.54 cm

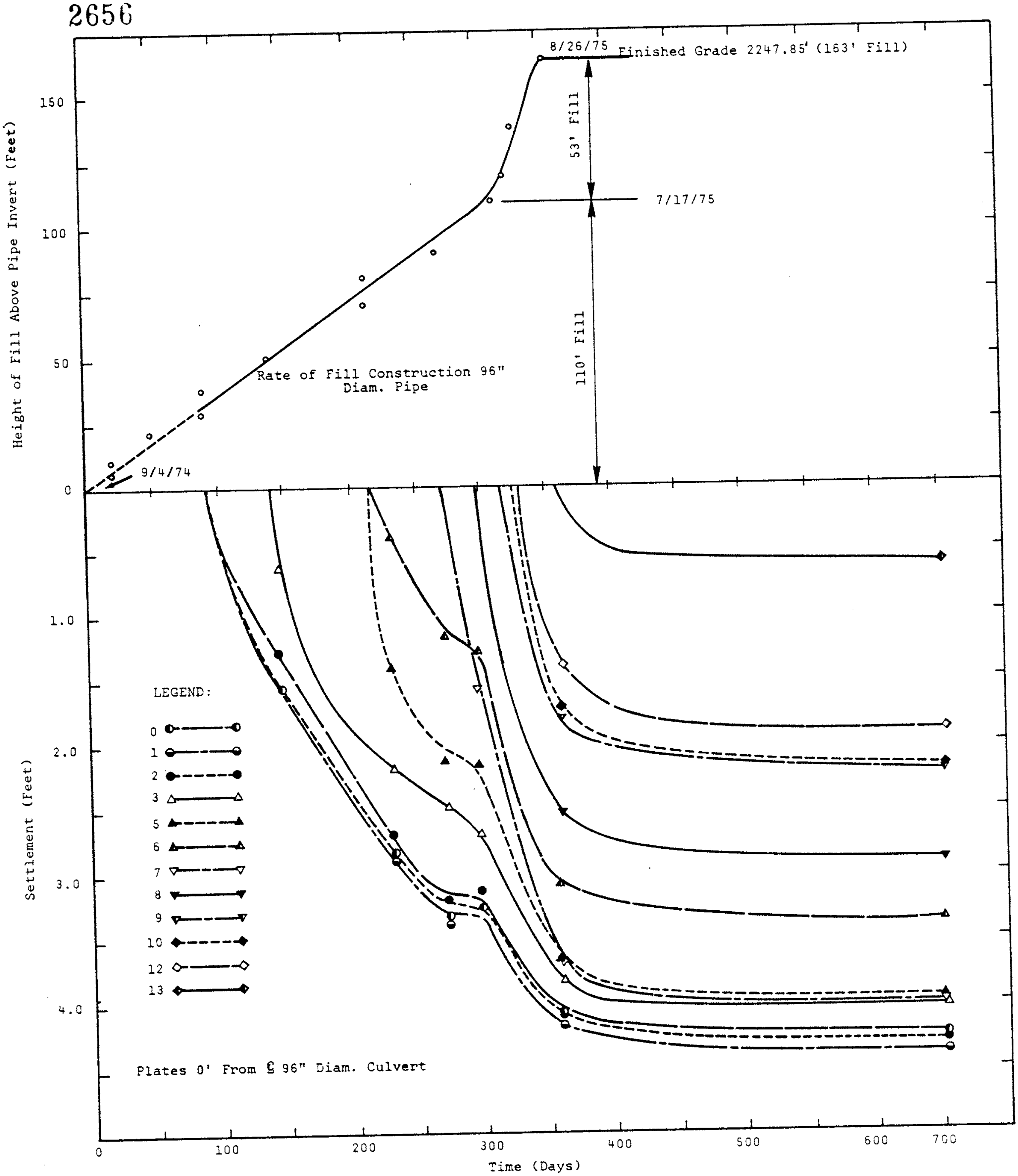


Figure 68. Rate of fill construction and settlement of plates above centerline of pipe.
Conversion: 1 ft. = 0.3 m;
1 in. = 2.54 cm

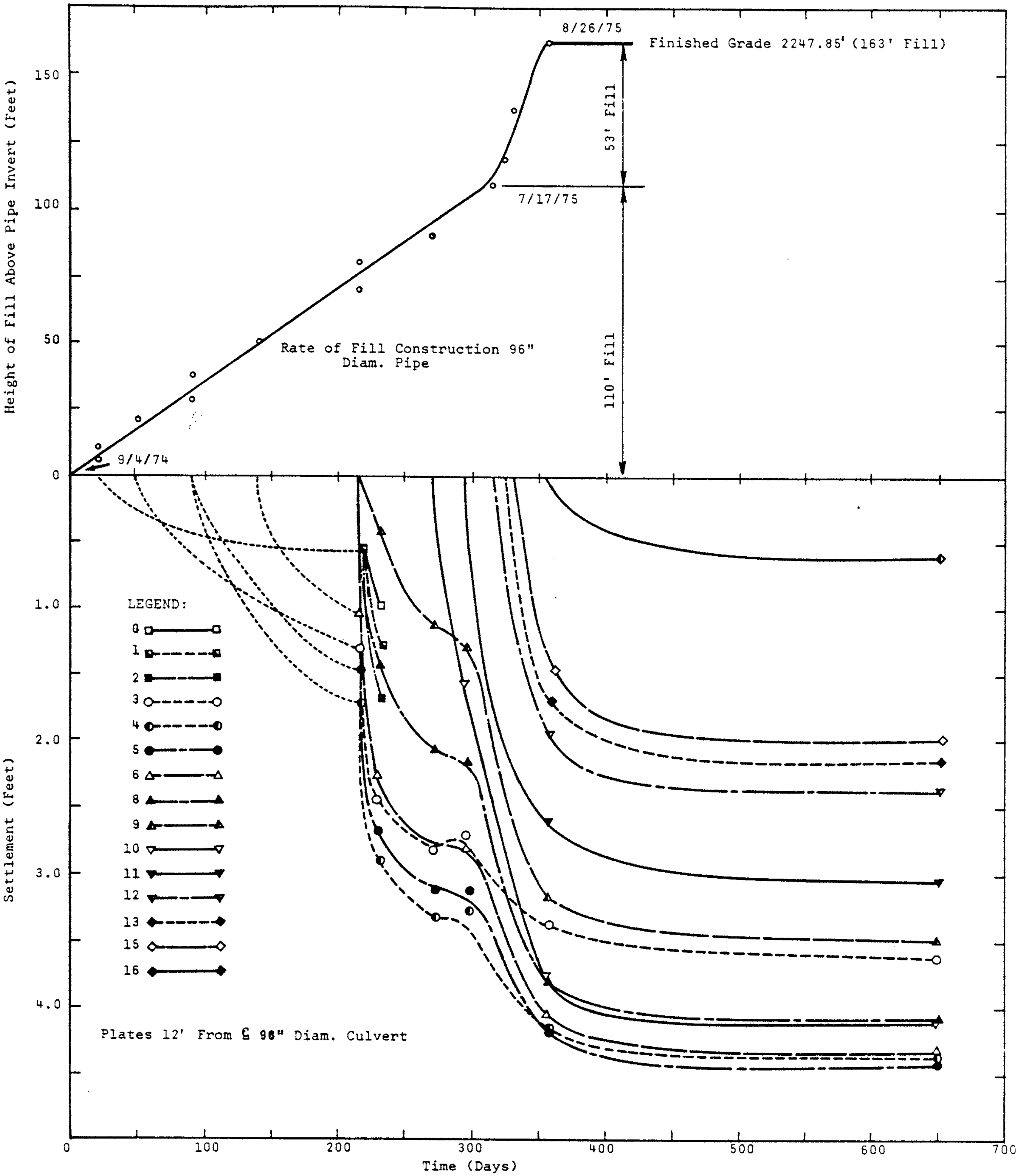


Figure 69. Rate of fill construction and settlement of plates 12 feet from centerline of pipe.
Conversion: 1 ft. = 0.3 m;
1 in. = 2.54 cm

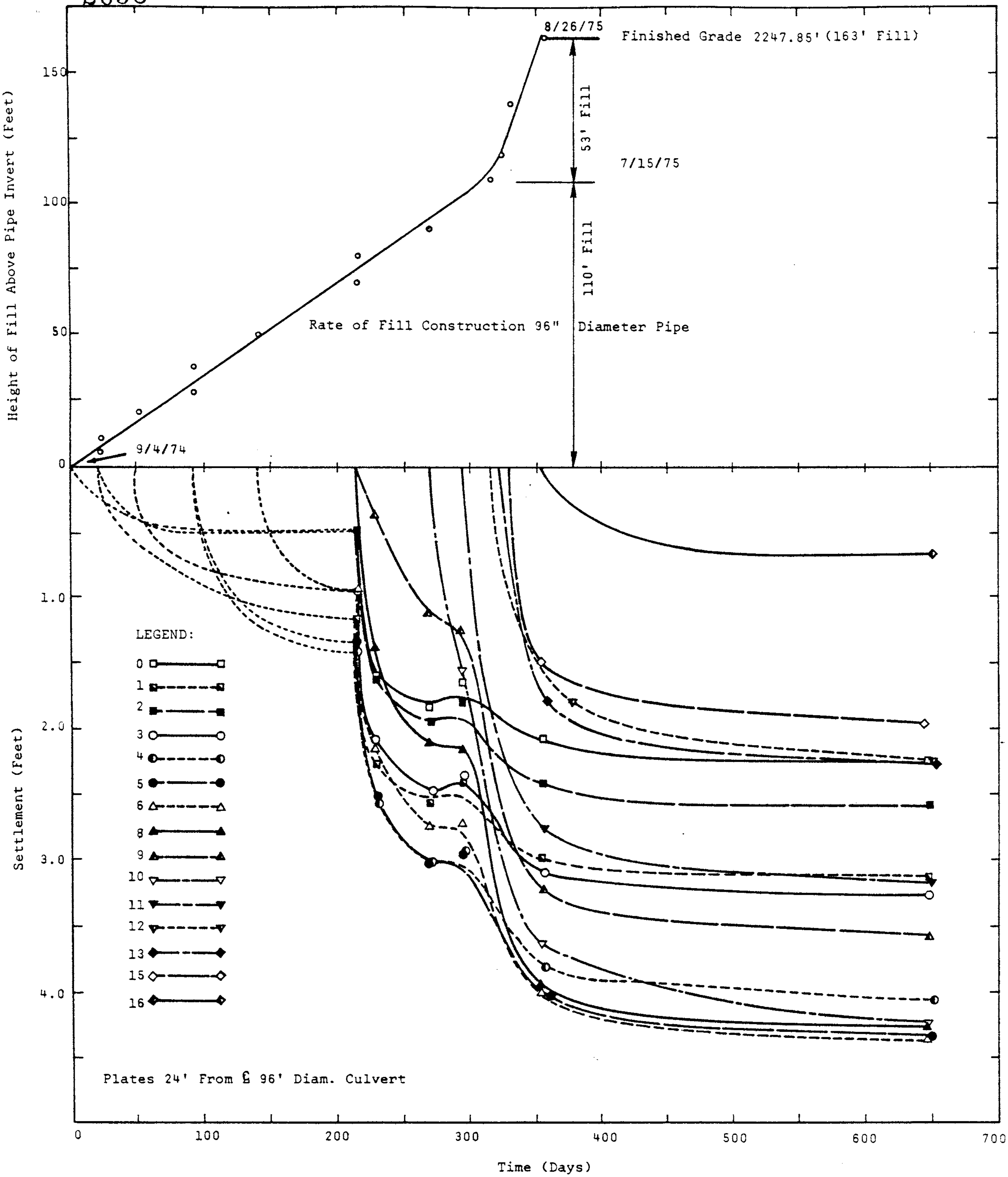


Figure 70. Rate of fill construction and settlement of plates 24 feet from centerline of pipe.
Conversion: 1 ft. = 0.3 m;
1 in. = 2.54 cm

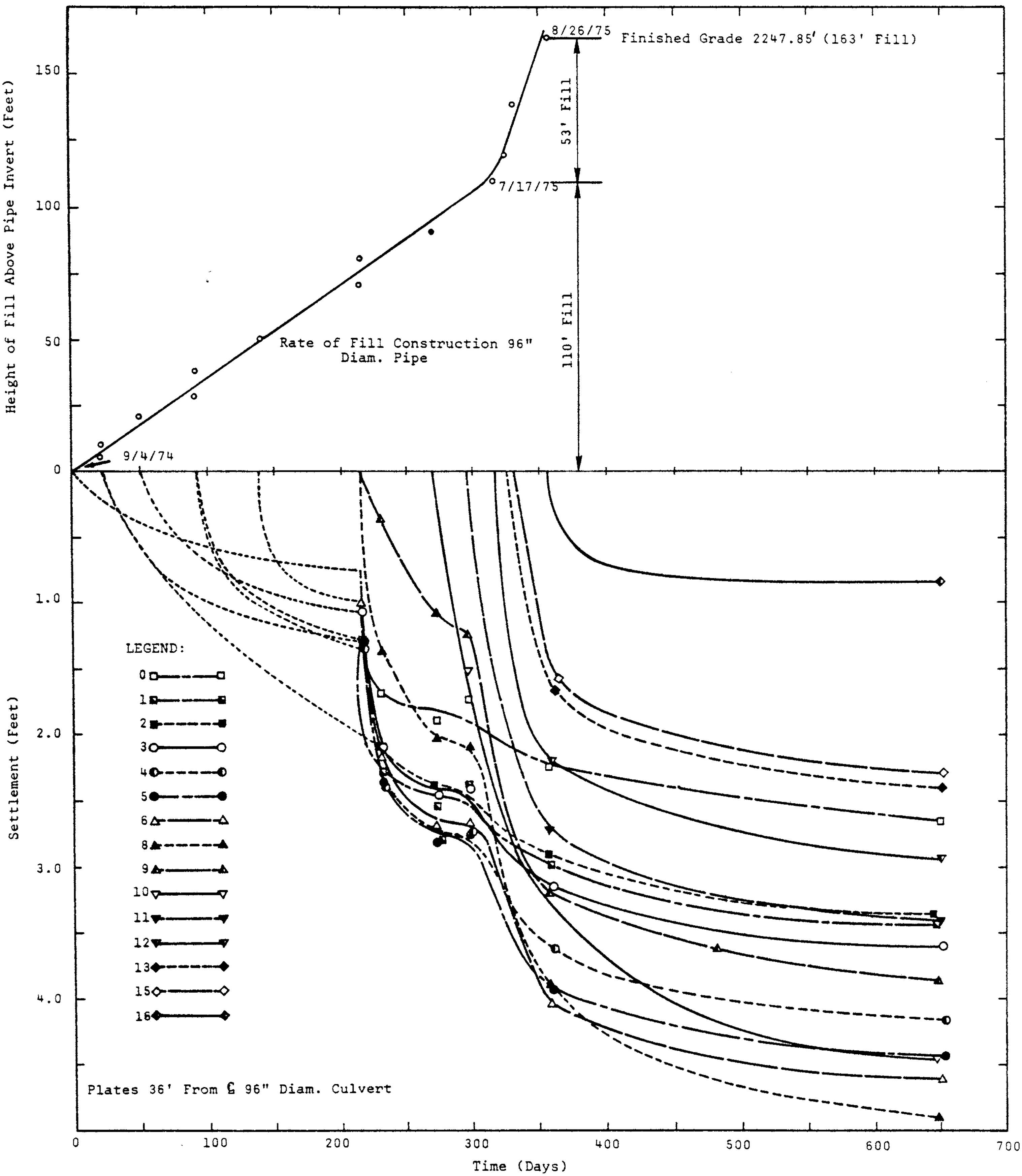


Figure 71. Rate of fill construction and settlement of plates 36 feet from centerline of pipe.
Conversion: 1 ft. = 0.3 m;
1 in. = 2.54 cm

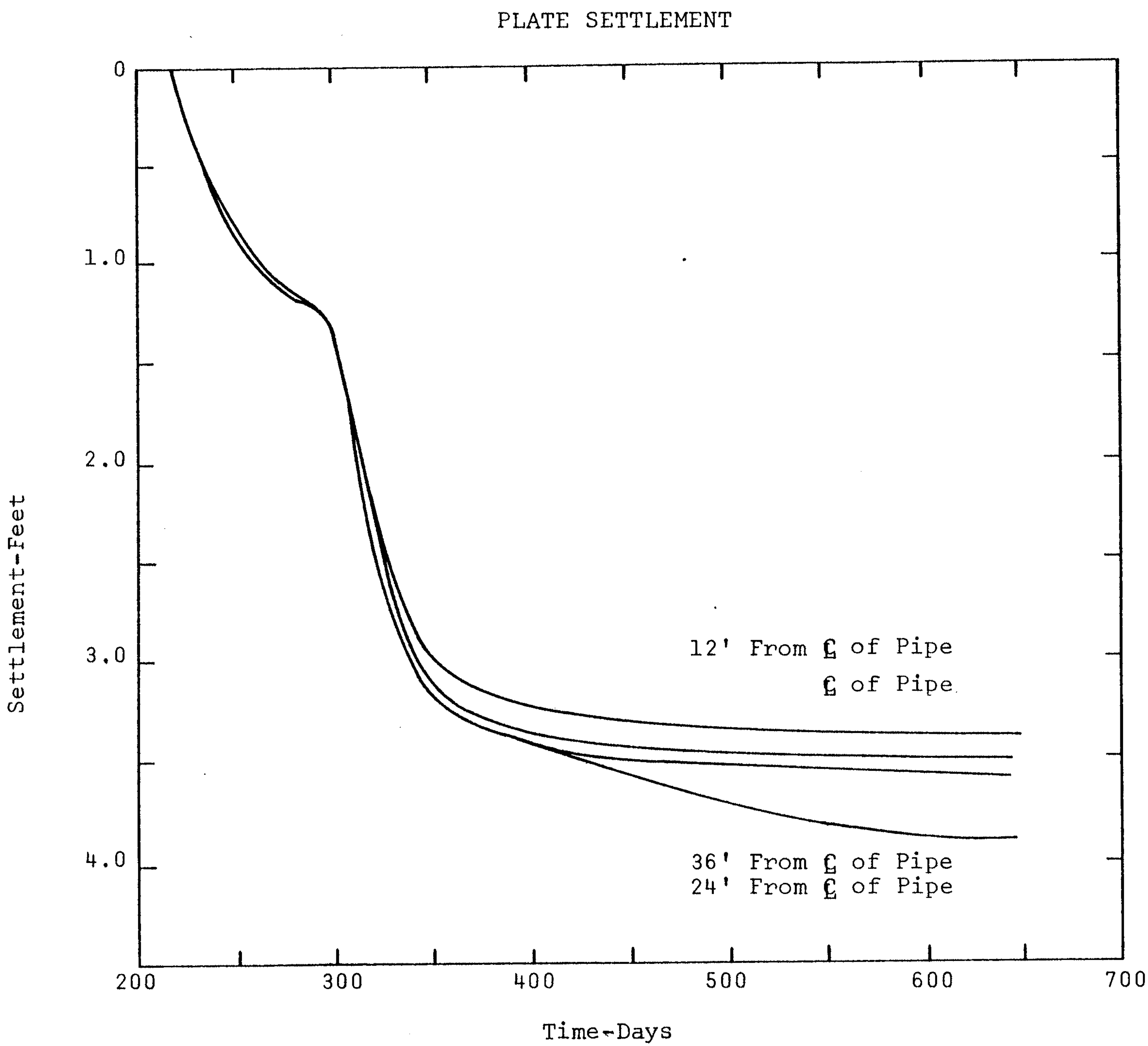


Figure 72. Typical settlement of 4 plates at the same original elevation but 0, 12, 24, and 36 feet from centerline of the pipe. Conversion: 1 ft. = 0.3 m