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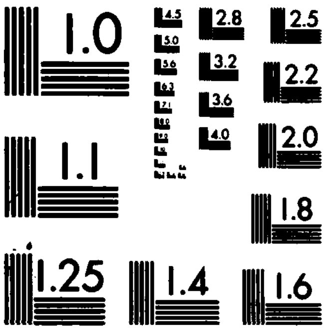

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TECHNICAL REPORT GL-84-4

EFFECTS OF MILITARY TRAFFIC ON BURIED, HIGH-PRESSURE PIPE

by

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US ARMY ENGINEER WATERWAYS EXPERIMENT STATION

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This study examines the mechanical effects of selected types of military traffic on buried, high-pressure pipe. Particularly, the study considers the section of the Colorado Interstate Gas (CIG) pipeline across Fort Carson's Pinon Canyon Maneuver Site (PCMS), which should be most susceptible to damage.

The relationship between depth of cover and steel pipe deflection is clarified experimentally, with special emphasis on dynamic effects (impact factor).

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20. ASBSTRACT (Continued).

The findings indicate that the pipeline at the PCMS (pressurized or unpressurized) is not susceptible to damage from random crossings by any anticipated military traffic, with the maximum expected impacts, as long as there is any cover over the pipeline. Further, any deflections which do occur, even at points of concentrated vehicle crossings, will be less than the recommended design deflection limit of 5 percent of the nominal pipe diameter.

PREFACE

This investigation was conducted by the Geotechnical Laboratory (GL), U. S. Army Engineer Waterways Experiment Station (WES), during the period February-September 1983. It was sponsored by Fort Carson and the 4th Infantry Division (Mech), AFZC-FE-EE, Fort Carson, Colo., as a study entitled "Effects of Military Traffic on Buried, High-Pressure Pipe." Initial consultations and investigations were funded under the Office of the Chief of Engineers Facilities Investigation and Studies Program, "Field Evaluation/ Consultation to Facility Engineer Pavement Problems."

The study was under the general supervision of Dr. W. F. Marcuson III, Chief, GL; Dr. T. D. White, Chief, Pavement Systems Division (PSD); Mr. S. G. Tucker, Chief, Engineering Analysis Group; and Mr. D. M. Ladd, Chief, Criteria Development Unit. The study was conducted by Dr. John C. Potter, PSD, who is the author of this report.

The Commander and Director of WES during this investigation was COL Tilford C. Creel, CE. Technical Director was Mr. F. R. Brown.

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CONVERSION FACTORS, NON-SI TO SI (METRIC) UNITS OF MEASUREMENT

Non-SI units of measurement can be converted to SI (metric) units as follows:

Multiply	By	To Obtain
feet	0.3048	metres
inches	2.54	centimetres
miles per hour	1.609347	kilometres per hour
pounds (mass)	0.5435924	kilograms
pounds (mass) per cubic foot	16.01846	kilograms per cubic metre
pounds (force) per square inch	6.894757	kilopascals
tons (mass)	1087.1848	kilograms

EFFECTS OF MILITARY TRAFFIC ON BURIED, HIGH-PRESSURE PIPE

PART I: INTRODUCTION

Objective

1. The mechanical effects of selected types of military traffic on buried, high-pressure steel pipe were examined in this study. Particularly considered was the section of the Colorado Interstate Gas (CIG) pipeline across Fort Carson's Pinon Canyon Maneuver Site (PCMS), which should be most susceptible to damage (see Figure 1). Preliminary analysis, as detailed in Part III, identified the 10-in.* pipe as being the most susceptible to damage from traffic. Therefore, the study focused on the behavior of the 10-in. pipe under wheeled- and tracked-vehicle loading. The relationship between depth of cover and pipe stresses/deflections was clarified experimentally, with special

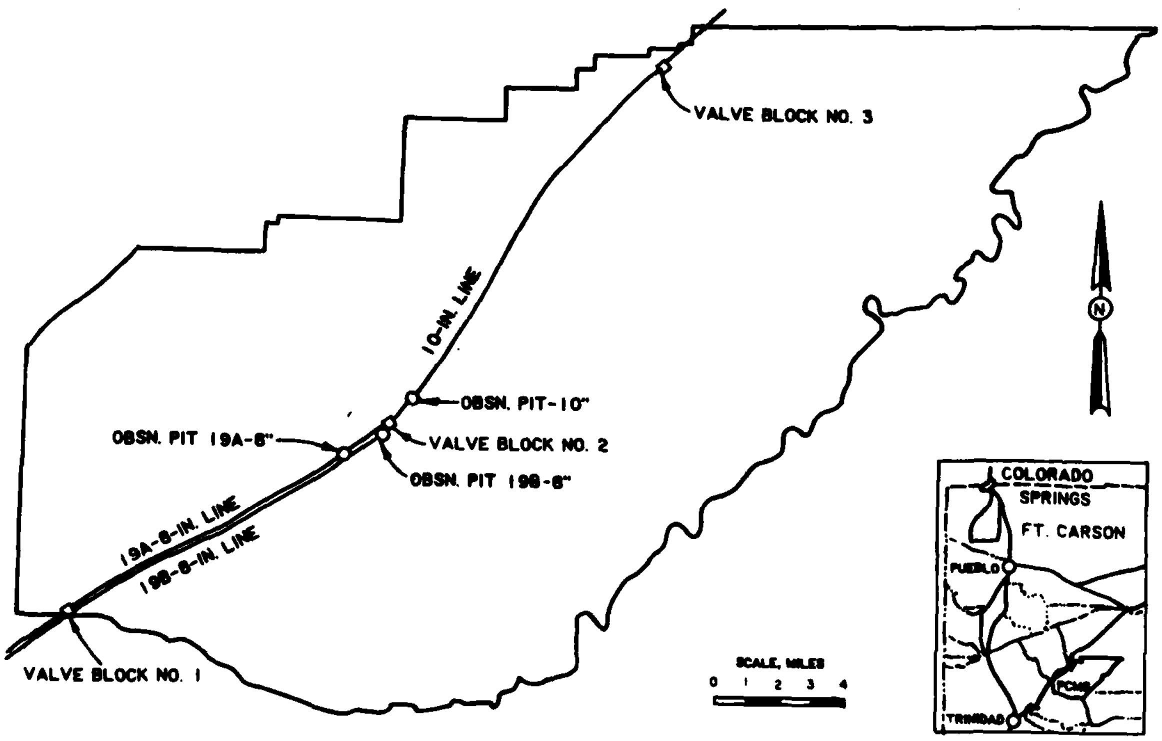


Figure 1. Pinon Canyon Maneuver Site (PCMS)

* A table of factors for converting non-SI units of measurement to SI (metric) units is presented on page 3.

emphasis on dynamic effects (impact factor). The results of this study provided the technical basis for evaluating the feasibility of CIG pipeline protection from anticipated PCMS traffic by earth cover.

Scope

2. The study consisted of a literature review, an office study, a field and laboratory testing program, and detailed analyses. The literature review was a survey of current theory and practice on analysis of buried conduits. Specific topics included soil mechanics and dynamics, finite element methods of computer analysis, pipe and culvert design, and case studies. The office study was an analysis of the various models for buried conduit behavior identified during the literature review. The assets and liabilities of each, to include validity of assumptions, ability to evaluate critical parameters, and reliability of results, were determined. An analytical approach was then selected for application in this study. A summary of the literature review and the results of the office study are detailed below. The testing program was planned to complement the selected analytic approach, to include measurement and/or control of parameters identified as significant. Testing included evaluation of PCMS soil and pipeline installation conditions and evaluation of traffic effects on an instrumented section of pipe. Details of the testing program are contained in Part II. The impact factors for various types of traffic and deflections of the pipe under traffic, as functions of cover depth, were determined during the detailed analyses. A description of the methods used and results obtained is presented in Part III.

Background

3. The complexity of the behavior of buried pipes goes beyond the fact that a wide variety of pipe types, loads, soils, and installation conditions are encompassed. Buried pipe behavior is a soil-structure interaction problem. Not only does pipe response depend on load conditions, but the pipe response affects load conditions. Pipe response to load conditions is primarily a function of pipe rigidity. In this respect, three classes of pipe are generally recognized (American Water Works Association (AWWA) 1964). These three classes are rigid, semirigid, and flexible. Rigid pipes exhibit a

brittle-type failure, and their cross-sectional shapes cannot be distorted enough to change their vertical or horizontal dimensions more than 0.1 percent without causing damage. Semirigid pipes can withstand changes in their vertical or horizontal dimensions of up to 3.0 percent without damage. Flexible pipes can withstand changes in their vertical or horizontal dimensions of more than 3.0 percent before damage occurs.

4. By permission, the following explanation of steel pipe behavior is reprinted from Steel Pipe Design and Installation (AWWA Manual No. 11), by the American Waterworks Association (1964):

> Steel pipe, having either light or heavy wall, because of its physical characteristics can always function as a flexible conduit....
>
> Real collapse failure of steel pipe does not occur under earth loads until a condition is reached where the vertical diameter has been decreased by about 20% of the nominal diameter and the horizontal diameter has been increased a similar amount.
>
> Although the maximum load-carrying capacity of flexible pipe depends to some extent on the wall thickness and its section modulus, the pipe, by deflecting, is able to make full use of the load carrying ability of the earth surrounding it. As the pipe may change shape without failure, it transfers part of the vertical load into a horizontal or radial thrust which is resisted by the passive pressure of the earth at its sides as they move outward. When the wall itself is rigid, this movement may not occur. It follows that the rigid pipe must carry the whole load itself, whereas the flexible pipe divides the load with the earth enclosing it. Therein lies the inherent difference between rigid and flexible behavior and the explanation of why the classical bending-moment formulas apply to the analysis of rigid pipe but not to the analysis of flexible pipe.

5. Spangler (1962) notes that flexible conduits fail by excessive deflection rather than by rupture of the pipe wall. He then recommends the investigation of flexible conduit deflections and the establishment of allowable deflection limits.

6. Returning to the AWWA manual:

> At this point, when deflection is mentioned, the engineer accustomed to thinking in terms of flexure or bending-moment formulas in rigid construction is likely to contend that permanent deflection can occur only after the yield point has been passed and that, therefore, a pipe so stressed has failed structurally and is dangerous. The simplest rebuttal to this argument is to recognize that the steel in a finished pipe has, in the manufacturing process,

been cold coiled, uncoiled, bent, curved, or twisted a number of times and has been stressed beyond the yield point each time; yet, after all these operations have been completed, the finished steel pipe is used for all manner of high-pressure work without fear or hesitation.

If the engineer still is hesitant to restress a part of the finished pipe wall beyond the yield point by slightly deflecting it underground, let him consider what happens to the test specimen by which the pipe strength is measured according to specification. Usually it is sliced as a ring from the end of a finished pipe, cut at one side, uncurled from the circle into a flat piece, and then put in a tensile-testing machine which proceeds to show that after once more passing the elastic limit, the steel still possesses the specified strength. In a way, the deflection underground is simply a finished forming operation.

Therefore, where steel pipe such as is here discussed is concerned, the word "failure" must define a state of falling short of satisfactory performance and not a state in which localized stresses appear to pass the yield point of the material as judged by the results of bending-moment formula analysis.

Whatever compressive stress due to soil action exists in the pipe must be overcome by the tensile force due to internal pressure before the pipe wall can go into tension.

For the above reason, the common practice of determining steel pipe wall thickness on the *basis of yield strength* to resist internal pressure is sound and conservative. Theoretical bending stresses and tensile stress should not be totaled for steel pipe as must be done for rigid pipe.

7. Spangler's equation for deflection (change in diameter) of a flexible pipe under an earth fill is

$$\Delta x = \frac{kDr^3\sigma_v}{EI + 0.061\ E'r^3} \tag{1}$$

where

Δx = deflection of the pipe, in.

k = bedding constant (0.108 for open trench)

D = outside diameter of pipe, in.

r = mean radius of pipe, in.

σ_v = net vertical load on pipe, psi

E = modulus of elasticity of pipe metal, psi

I = moment of inertia of cross section of pipe wall, per unit length, $in.^4$ per in.

E' = modulus of soil reaction, psi (300 psi for untamped backfill)

8. Maximum design deflection of 5 percent of the nominal pipe diameter is generally considered reasonable (Spangler 1962; American Iron and Steel Institute 1971). Since the area and hydraulic radius of a pipe deflected by 5 percent are 99.75 percent of the values for a perfect circle, the effect of a 5 percent deflection on hydraulic efficiency is negligible.

9. It is important to note that the vertical load used in equation 1 is the net vertical load. The net vertical load is equal to the total external vertical load only in the case of an unpressurized pipe. When the pipe is pressurized, the resultant of the vertical components of the internal pressure is greater than the resultant of the horizontal components because of the elliptical shape of the deflected pipe. The excess vertical internal pressure against the upper half of the pipe counteracts the external vertical load. The result is a decrease in the net vertical load and a decrease in deflection in the case of a pressurized pipe (Figure 2).

10. The relationship between deflection and excess vertical pressure resulting from internal pressure is easily derived from simple geometric

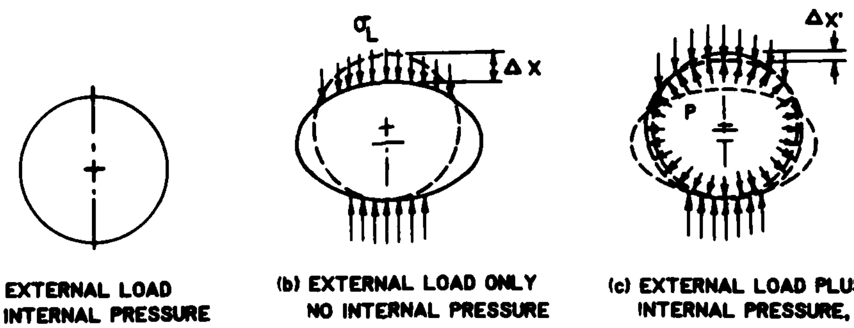


Figure 2. The effects of internal pressure

relationships. The excess vertical pressure, $\Delta\sigma_v$, is given by

$$\Delta\sigma_v = \frac{2p\Delta x}{D + \Delta x} \tag{2}$$

where p = internal pressure, psi.

11. The net vertical load used in equation 1 is then

$$\sigma_v = \sigma_L - \Delta\sigma_v \tag{3}$$

where σ_L is the total external vertical load.

12. The total external vertical load is produced by the soil overburden and by vehicles on the ground surface above the pipe. The dynamic load induced by a vehicle is generally described as the static-vehicle load multiplied by an impact factor. The total external vertical load is thus described by the expression

$$\sigma_L = \sigma_{DL} + I \cdot \sigma_{LL} \quad (4)$$

where

σ_{DL} = dead load due to soil overburden, psi

I = impact factor

σ_{LL} = static, live load due to vehicles parked on the ground surface above the pipe, psi

13. The dead load acting on the pipe may be calculated using Marston's theory of loads (Spangler 1962). Evaluation of the required parameters depends upon extensive knowledge and/or assumptions about ditch geometry, soil type, and settlement ratio. However, the upper bound for a flexible conduit is the weight of the soil prism directly above the pipe. This upper limit has also been confirmed by experiment (Braune, Cain, and Janda 1929; Barnard 1957). For design, the American Iron and Steel Institute (1971) and others thus recommend using

$$\sigma_{DL} = \frac{h\gamma}{1728} \quad (5)$$

where

h = height of soil over pipe, in.

γ = unit weight of soil over pipe, pcf

Few errors result from making this conservative simplification, especially for shallow installations.

14. A lag factor is sometimes applied to the dead load to account for long-term deformation of the backfill at the sides of the pipe. However, when the pressure in the pipe equals or exceeds the dead-load vertical pressure the lag factor is 1.00 (Barnard 1957). This is the case for the pipeline at the PCMS. The test section is not subject to long-term effects; hence a lag factor does not appear in these equations.

15. Live loads, such as static-vehicle wheel loads, are transferred through the soil to buried conduits, essentially in accordance with the Boussinesq distribution (Spangler 1962). The vertical stress due to live load can be calculated directly from the Boussinesq equation if the depth is greater than about twice the width of the load contact area. For tank tracks and dual truck wheels, this minimum depth is about 4 ft. For depths less than 4 ft, the contact pressure must be integrated over the contact area to give the stress increase at depth (Sowers and Sowers 1970). Maximum vertical stress versus depth, h , in terms of contact pressure, q , is shown in Figure 3.

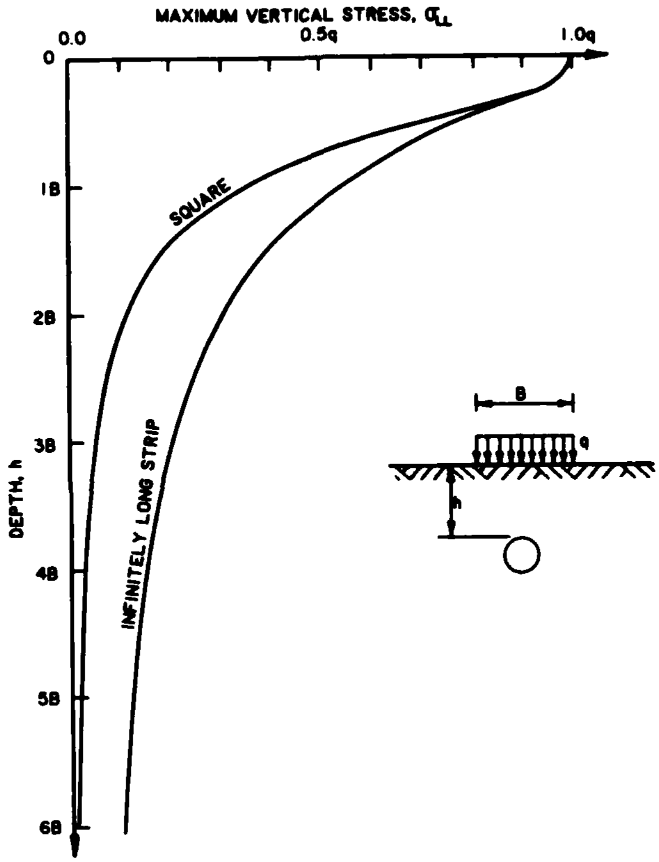


Figure 3. Maximum vertical stress beneath center of uniformly loaded square of width B , Boussinesq analysis

16. The impact factor depends upon characteristics such as vehicle type, vehicle velocity, and ground surface roughness. Precisely how these characteristics affect the impact factor is not well understood. Few empirical data are available for estimating the impact factors for military vehicles operating in a cross-country environment. Thus, the impact factors

appropriate to this special case were measured as a part of this study. Specific procedures and results are presented in Parts II and III.

PART II: TESTING

Field Conditions

17. The District Engineer (MROED-MC), U. S. Army Engineer District, Omaha (1982) conducted a subsoil survey for the proposed cantonment area at the PCMS. This survey provided a general description of PCMS soil conditions. The principal investigator also conducted subsoil investigations along the PCMS pipeline right-of-way to provide additional information.

18. The cantonment area investigation was conducted from 19 May to 3 June 1981. The program consisted of 40 borings, advanced 13.4 to 20 ft below the ground surface. Standard penetration tests were made in all holes. Both standard penetration samples and undisturbed samples were taken. Laboratory testing was conducted on these samples. Additional details of the exploration and testing program can be found in the report by the Omaha District (1982). The results pertaining to the top stratum are of interest to this investigation. The upper stratum, encountered in all borings, is a residual lean and sandy clay (CL) with an average thickness of about 10 ft. The natural moisture content was from 4 to 15 percent. Temporary saturation producing a moisture content of 27 to 37 percent can result from heavy precipitation. The liquid limit was found to be 23 to 44 percent; the plasticity index, 9 to 27 percent. Dry density ranged from 75 to 110 pcf.

19. The principal investigator's study of the pipe installation and soil conditions was conducted on 3 March 1983. An observation pit was excavated at one location along each of the three segments of pipeline crossing the PCMS. The locations of these observation pits are shown in Figure 1. Measurements taken at each pit included cover depth, ditch width, pipe bedding, pipe-wall thickness, and pipe-electrical potential. Drive tube samples of the backfill material were taken at each pit, and an undisturbed sample of the virgin subsoil was taken from the pit along the 10-in. pipeline. Pipeline right-of-way topograr [illegible] ndition of the pipeline protective coating, and characterizatior [illegible] peline bedding, ditch geometry, and subsurface soil fabric were alsc [illegible]

20. The lean anu [illegible] clay (CL) identified in the cantonment area survey was observed in all three pits. The backfill material around and above the pipe was the spoil material from the trenching operation. The

backfill was found to be firm but less dense and more weathered than the virgin soil at pipe depth. The average dry density of the backfill was 81 pcf. The moisture content of the backfill averaged 9 percent (Table 1). The virgin material had a dry density of 110 pcf and a moisture content of 9 percent.

21. Study of the two pits (Figure 4) along the 8-in. lines indicated a uniform trench width of 22 in. An inch or less of backfill material was observed between the pipe and trench bottom. The trench for the 10-in. pipe, however, was very irregular. This irregularity apparently resulted from the necessity to remove a 12-in. layer of sandstone located just below the ground surface before proceeding with pipe trench construction. The sandstone layer was removed in a 5- to 6-ft-wide swath. The remainder of the trench was then excavated. This subditch was observed to be approximately 37 in. wide at the top of the pipe. Backfill between the pipe and the trench bottom was 5 to 6 in. deep.

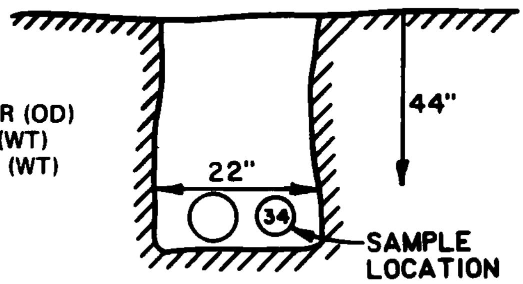
22. An ultrasonic thickness gage was used to determine pipe-wall thickness at each pit location. In each case the measured wall thickness was slightly greater than the wall thickness specified for the particular section of pipe. The protective coatings were found to be intact, and the measured electrical potentials of the pipe were within specifications for proper cathodic protection.

Test Section

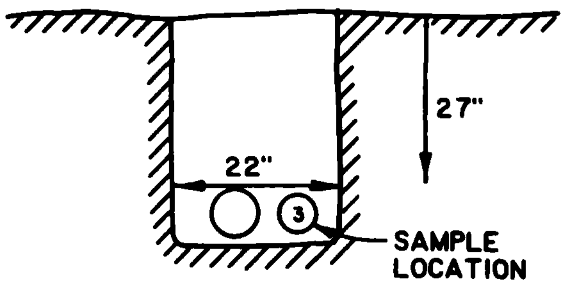
23. The test section was designed to emulate that section of the PCMS pipeline which is judged most susceptible to damage from military traffic. Instrumentation was installed to measure pipe deflections. When loaded, the pipe functions as a proving ring, with deflection being directly proportional to load. Thus, actual loads can be calculated from pipe deflections. These loads were used to verify theoretical loads and determine impact factors. As noted in the Background section, the effects of internal pressure are easily analyzed. Pressurized pipelines exhibit smaller deflections than unpressurized pipelines carrying the same load. Thus the pipe in the test section was not pressurized. Safety and speed of construction were increased while costs were decreased without reducing the validity of the test results.

24. Because of its larger span, the 10-in. pipe is the most damage-susceptible type of pipe in the PCMS. A 31-ft section of 10- by

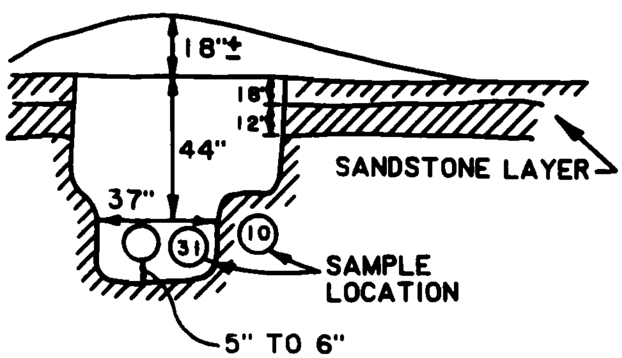
PIT 19B-8 IN. (NEW)
8-5/8 IN. SPECIFIED OUTSIDE DIAMETER (OD)
0.156 IN. SPECIFIED WALL THICKNESS (WT)
0.160 IN. MEASURED WALL THICKNESS (WT)



PIT 19A-8 IN. (OLD)
8-5/8 IN. SPECIFIED OD
0.277 IN. SPECIFIED WT
0.280 IN. MEASURED WT



PIT 10 IN.
10-3/4 SPECIFIED OD
0.188 IN. SPECIFIED WT
0.190 IN. MEASURED WT



VIRGIN SOIL IS RESIDUAL SOIL DERIVED FROM WEATHERED SHALE AND SANDSTONE. BACKFILL IS "FIRM"; A 1/2-IN. ROD CAN BE DRIVEN IN THE BACKFILL BY HAMMER; AND BACKFILL IS MUCH MORE WEATHERED THAN THE VIRGIN SOIL. TOPOGRAPHY—GENTLY ROLLING, ARID GRASSLAND.

Figure 4. Observation pits, CIG pipeline, 3/3/83

0.188-in.-wall thickness X42 Grade EW pipe was obtained from CIG's Devine Substation for use in the test section.

25. Pipe deflection was measured by four spring-loaded Collins Direct Current Differential Transformers (DCDT's). This gage is similar to the Linear Variable Differential Transducer (LVDT) except that the driver and signal-processing electronics are located in the body of the DCDT to reduce errors and interference resulting from long lead-in cables. The DCDT's were mounted inside the pipe under the wheel path of the test vehicles. Two were mounted vertically at 68 and 72 in. from one end of the pipe. The other two were mounted horizontally at 65 and 75 in. from the same end of the pipe. One horizontal and one vertical gage are depicted in Figure 5. Leads from the DCDT's were connected through signal-conditioning cards to a power supply and oscillograph. A digital voltmeter was also used to measure steady-state gage outputs. The gages were calibrated in a laboratory-calibration fixture before installation. Zero-deflection readings were taken before test-section construction was begun.

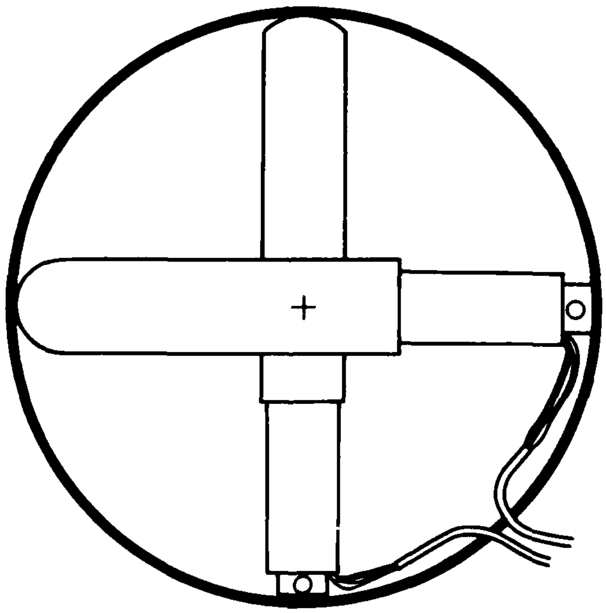


Figure 5. Deflection gages; Collins DCDT, spring-loaded, two each horizontal and vertical

26. The test section was constructed in an earth-floored shelter to facilitate maintaining the desired moisture content of 6 to 10 percent. The instrumented pipe was placed directly on a leveled section of shelter floor. This placement duplicated the bedding condition corresponding to a flat-trench

bottom, where special bedding materials or methods are not used. This is conservative since some ditch spoil was found between the pipe and trench bottom in the PCMS observation pits. However, insufficient evidence of special bedding treatment was detected to justify a less stringent bedding condition. Actual trench width at the PCMS was observed to be variable. This variability and the possibility of future, additional protective cover over the pipeline led to selection of embankment (positive projection) construction for the test section. This ensures the worst-case behavior of the test section. Test section layout is shown in Figure 6 and test section densities and moisture measurements are given in Table 2.

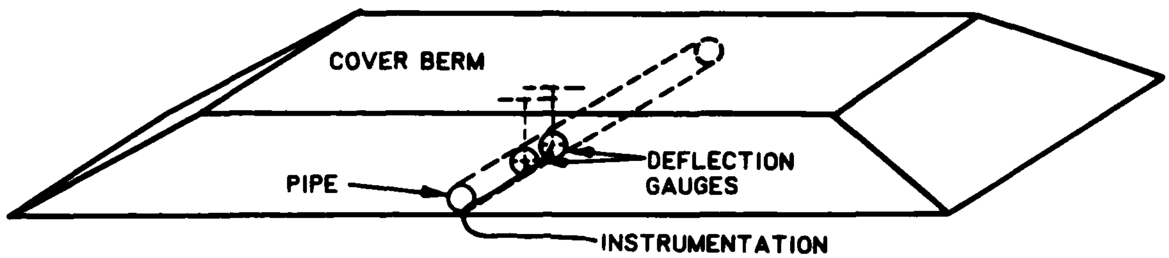


Figure 6. Test section layout

27. The test section was constructed with a processed sandy clay (CL) blend similar to the sandy clay of the PCMS. A gradation curve is depicted in Figure 7. The blended material had a liquid limit of 23 and a plasticity index of 6. The material was dried to an average moisture content of less than 10 percent before placement. In-place moisture contents of 5 to 9 percent were recorded during testing. The embankment was constructed without deliberate compaction to simulate the untamped backfill around the pipeline at the PCMS. However, some compaction did result from earthmoving and test vehicle traffic. In-place dry densities of 110 to 120 pcf were recorded. These densities correspond to expected densities in areas of concentrated maneuver traffic. For most areas of the PCMS, however, densities are less than those of the test section. Thus, the expected soil-load deflections along the PCMS pipeline should be less than those observed in the test section.

28. Traffic loads were applied with both wheeled and tracked vehicles. Vehicles and payloads were selected to simulate the severest vehicle-loading configurations anticipated at the PCMS. The principal wheeled test vehicle was an M51 5-ton dump truck with a maximum highway payload of 20,000 lb and a gross weight of 42,000 lb. Since the ground contact pressure is

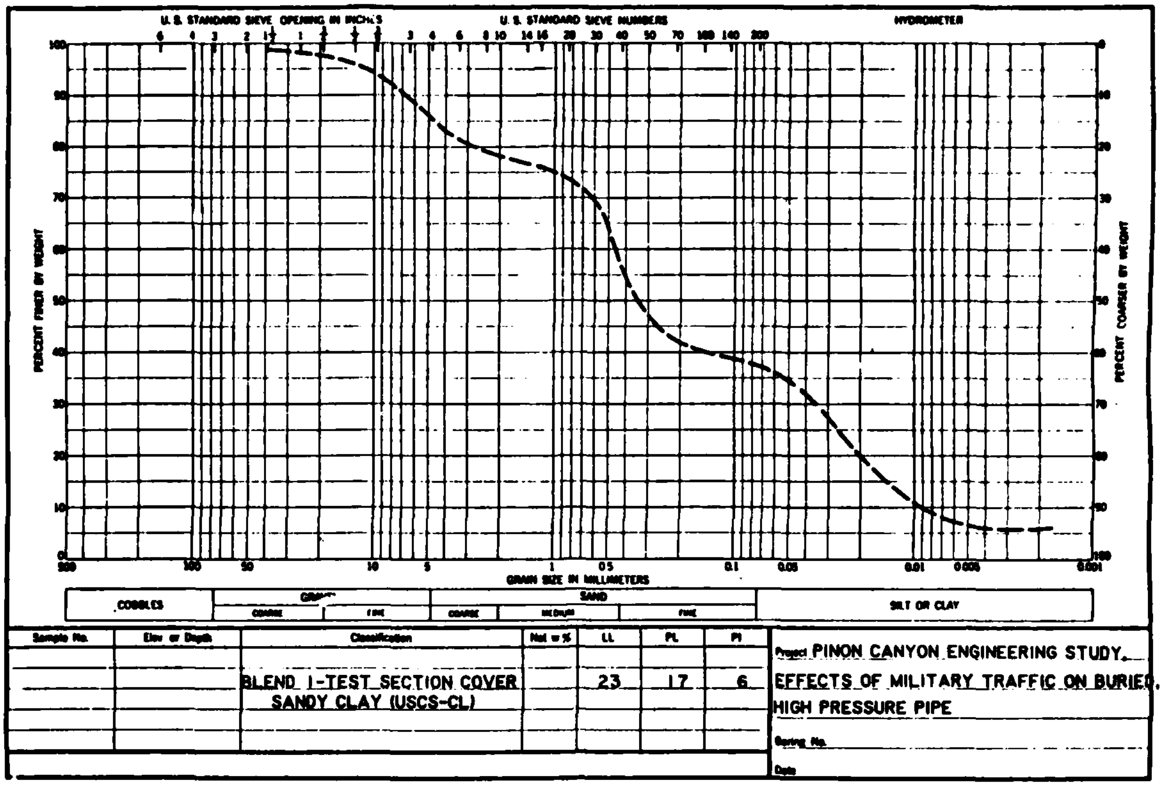


Figure 7. Gradation curves

approximately equal to tire pressure, a maximum inflation pressure was chosen. All tires were inflated to 70 psi. The principal tracked test vehicle was a modified M48 tank chassis. In lieu of the turret assembly, special ballast weights were used to achieve a gross weight of 140,000 lb. The hard-surface ground contact pressure of this vehicle exceeds 80 psi. The soft-surface ground contact pressure is about 13 psi. The M520E1 GOER was not included in testing because of its relatively low ground contact pressure. Typical GOER tire pressure, and hence ground contact pressure, is approximately 35 psi.

29. For each depth of soil cover over the test section pipe, three types of deflections were recorded: dead-load deflection, static-vehicle deflection, and dynamic-vehicle deflection. Dead-load deflections were deflections due solely to the weight of the soil covering the pipe. Static-vehicle or live-load deflections were the maximum deflections resulting from the weight of a vehicle either parked or rolled slowly across the top of the test section. Dynamic-vehicle deflections were the maximum deflections due to moving vehicles. Dynamic deflections were recorded while operating at high speeds, moving over obstructions or humps, and while stopping or accelerating.

30. Two techniques (multiple passes and gage-reading interpolation)

were used to ensure that maximum deflections were recorded. Multiple passes increased the probability of maximum dynamic vehicle loads being generated over the axis of the pipe. For these maximum loads, the maximum pipe deflections were determined by interpolation between deflections recorded by the gages spaced along the pipe axis. Thus, large deflections, not occurring exactly at a gage location, could be estimated. Pipe deflection along the axis of the pipe and in the neighborhood of the gages was modeled by a second-degree polynomial. The coefficients of this polynomial were determined using least squares regression through the points determined from the deflection readings and from the locations of the gages along the pipe axis. The maximum deflection of the pipe was then taken as the maximum of this polynomial. This technique also compensates for small errors in deflection readings. Deflections caused by loads occurring outside of the neighborhood of the gages can also be rapidly identified. Recorded deflections, along with averages for soil (dead) loads and interpolated maximums for vehicle (live) loads, are listed in Tables 3-7.

PART III: ANALYSIS

Test Section

31. Test data, contained in Tables 3-7, were used to determine the relationship between pipe deflection and cover depth. As shown in Figure 8, the average deflection of the pipe was directly proportional to cover depth.

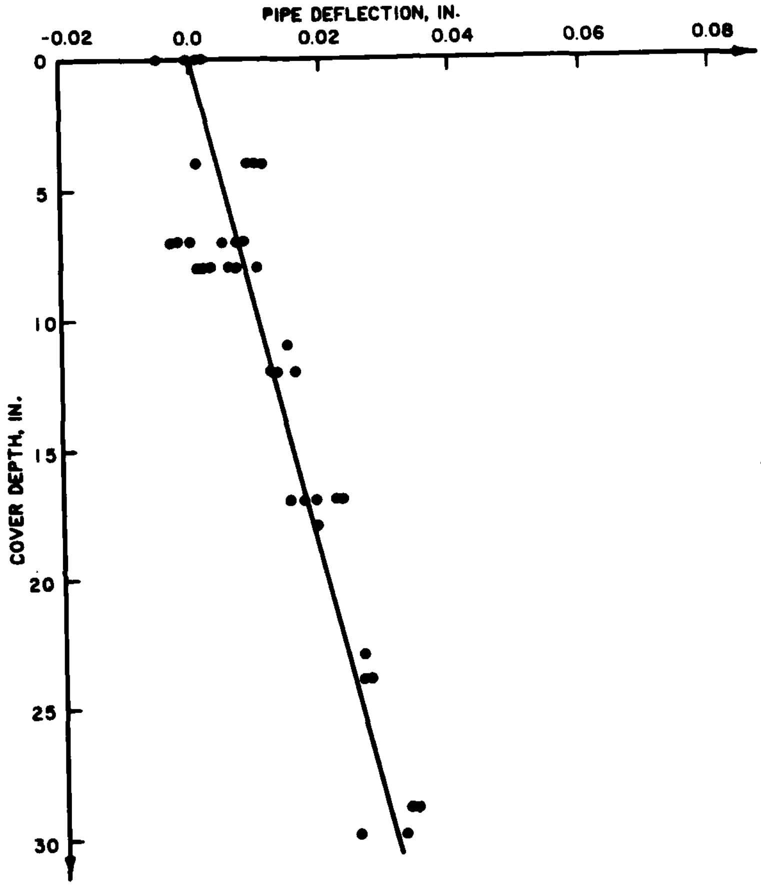


Figure 8. Average soil-load deflection versus cover depth

32. The relationships between maximum static live load and cover depth for both wheeled and tracked test vehicles are shown in Figure 9. The shape of these curves is similar to that of the Boussinesq stress distribution. Static deflections caused by the tracked test vehicle are larger than those caused by the wheeled test vehicle for all cover depths (0 to 30 in.).

33. Pipe deflection is directly proportional to load (Equation 1). Therefore, test section impact factors can be computed from the ratio of

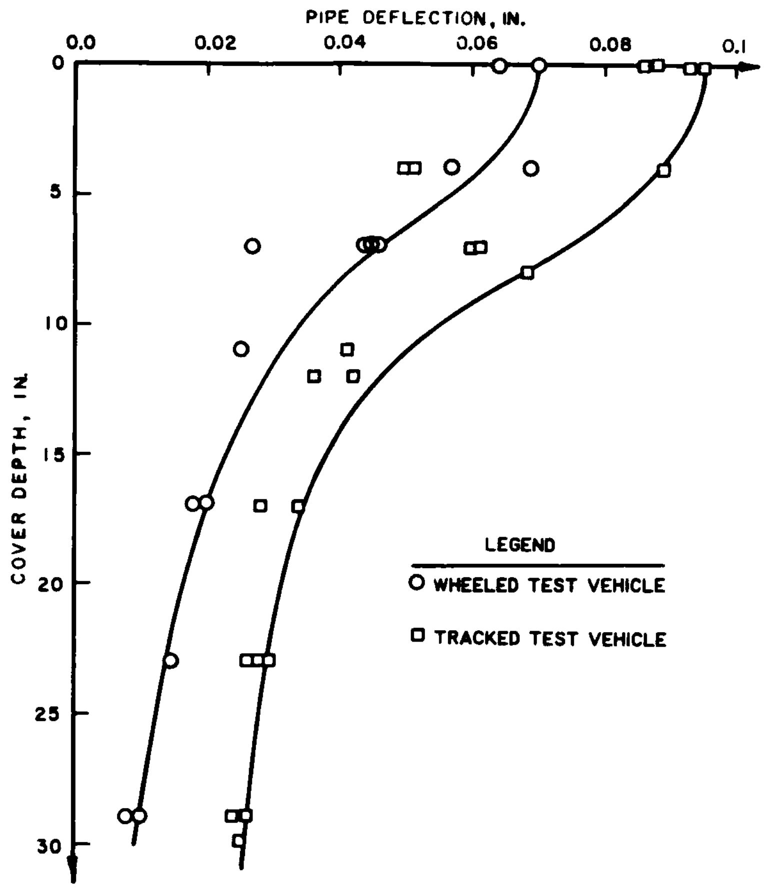


Figure 9. Maximum static vehicle deflections versus cover depth

the dynamic vehicle deflections to the static vehicle deflections. Maximum impact factors computed for the wheeled- and tracked-test vehicles are listed in Tables 8 and 9, respectively.

34. For normal modes of vehicle operation (for example high-speed travel, stopping, and accelerating) the impact factors are less than or equal to 1.3. In fact, impact factors less than 1.0 were common. For moving vehicles, load durations are too short to allow mobilization of full static deflections. This phenomenon is well documented in pavement-design literature (Yoder and Witczak 1975; Horonjeff 1962; Yoder 1959) and is reflected in current Corps of Engineers pavement design procedures. Impact factors greater than 1.3 were observed under special conditions such as obstacle crossing. Impact factors as high as 4.1 were observed. Under similar conditions, the impact factors generated by the tracked test vehicle were larger than those generated by the wheeled test vehicle.

35. The test data suggest an upper limit of 5 for the impact factor. This agrees with recommendations made by Gemperline (1982), who measured impact factors generated by loaded scrapers operating on very rough haul roads. Recorded impact factors were as high as 3.0, and he estimated the maximum impact factor to be near 5. This upper bound is also consistent with the results of ride tests and shock tests conducted with various vehicles by the Mobility Systems Division of the WES Geotechnical Laboratory (Schreiner 1981; Schreiner and Green 1982; Schreiner and Randolph 1979). They recorded total chassis accelerations up to 5 g's (gravity) while maneuvering vehicles over abrupt obstacles. They also observed that vehicle operators' physical tolerance levels dictate restricting chassis accelerations to under 5 g's. In uncontrolled environments, operators usually choose vehicle speeds which restrict chassis accelerations to 3.5 g's or less. A maximum chassis acceleration of 3.5 g's is used as the speed-limiting criterion for cross-country traffic in the Army Mobility Model (Schreiner and Green 1982). All of these maximum impact factors were recorded at vehicle speeds of 5 to 15 mph. Lower or higher speeds resulted in reduced impact factors.

36. The live-load deflections for the tracked test vehicle will be used for the analysis of pipe performance, since the tracked test vehicle produced both the largest static live-load deflections and the largest impact factors. The test section pipe is subject to a maximum dynamic live-load deflection up to five times the static live-load deflection produced by the tracked test vehicle. The maximum total deflection is found by adding the maximum dynamic live-load deflection to the dead-load deflection for each cover depth. The relationship of maximum deflection to cover depth is shown in Figure 10. This curve was derived by adding the average soil-load deflection (from Figure 8) to five times the maximum static vehicle deflections for the tracked test vehicle (from Figure 9).

37. For cover depths in the range of 0 to 30 in., the maximum total deflection occurs at zero cover depth and is less than 5 percent of the nominal pipe diameter. For cover depths greater than 30 in. the maximum live load decreases asymptotically to zero, while the dead load continues to increase linearly with depth. Hence, the total deflection is expected to exceed the 5 percent design deflection at some large depth. By extrapolating from the curves in Figures 8 and 9, this depth is estimated to be approximately 40 ft.

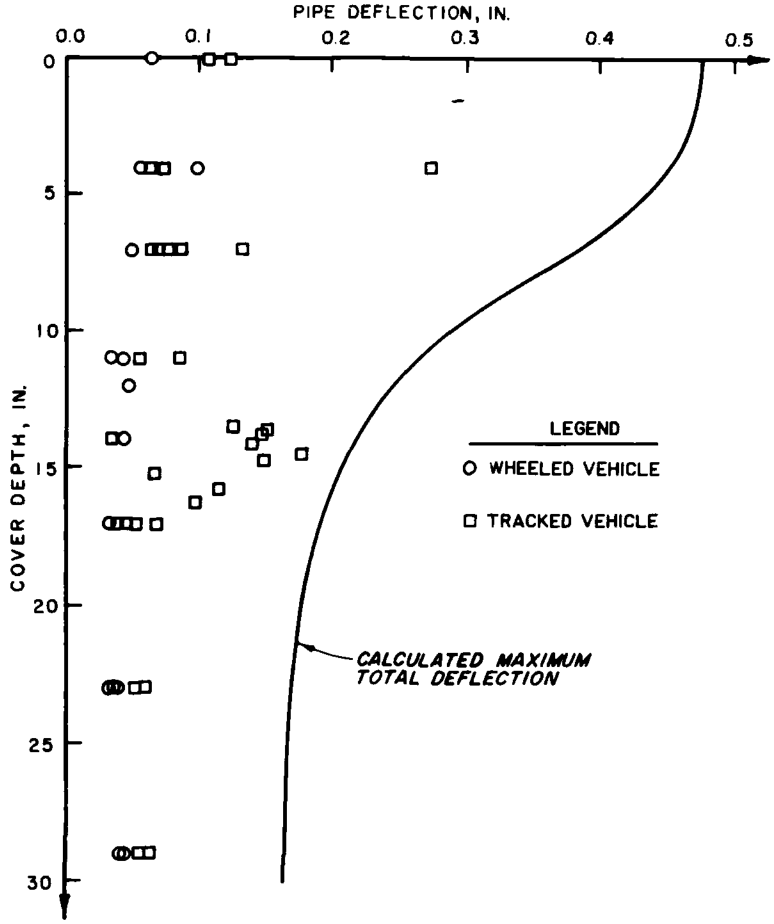


Figure 10. Actual total deflection readings from the test section and calculated maximum total deflection versus cover depth

Pinon Canyon Maneuver Site

38. From Equation 3 it is seen that pressurizing the pipe will result in smaller deflections for a given external vertical load. Hence, deflections in the 10-in. pressurized pipeline crossing the PCMS should be smaller than those of the test section pipe. Negative projection ditch conditions, pipe bedding, and lighter overburden along the PCMS pipeline will also contribute to smaller deflections in the 10-in. pipeline crossing the PCMS.

39. The net vertical pressure is proportional to the deflection. It can be computed from the deflection using Spangler's deflection formula (Equation 1). For the 10-in. pipe, one might estimate:

$k = 0.108 \qquad E = 30 \times 10^6$ psi

$D = 10.75$ in. $\qquad I = 5.5 \times 10^{-4}$ in.^{4}/in.

$r = 5.28$ in. $\qquad E' = 300$ psi

The deflection of the 10-in. pipe at the PCMS should then be

$$\Delta X_{10} = 0.0089 \ \sigma_v \tag{6}$$

The deflection, expressed as a percentage of nominal diameter, is

$$\Delta \%_{10} = 0.089 \ \sigma_v \tag{7}$$

For the line designated as 19A-8 in. on the site map (Figure 1), one might use

$k = 0.108 \qquad E = 30 \times 10^6$ psi

$D = 8.625$ in. $\qquad I = 1.77 \times 10^{-3}$ in.^{4}/in.

$r = 4.174$ in. $\qquad E' = 300$ psi

then

$$\Delta X_{19A-8} = 0.00124 \ \sigma_v \tag{8}$$

and

$$\Delta \%_{19A-8} = 0.0156 \ \sigma_v \tag{9}$$

For the 19B-8in. line (shown on the site map in Figure 1),

$k = 0.108 \qquad E = 30 \times 10^6$ psi

$D = 8.625$ in. $\qquad I = 3.2 \times 10^{-4}$ in.^{4}/in.

$r = 4.2345$ in. $\qquad E' = 300$ psi

then

$$\Delta X_{19B-8} = 0.00644 \ \sigma_v \tag{10}$$

and

$$\Delta\%_{19B-8} = 0.0804\ \sigma_v \tag{11}$$

Hence, the prediction that deflections in both sections of 8-in. pipe at the PCMS will be less than those of the 10-in. section for any given net vertical load, σ_v . Percentage deflections for the 8-in. sections will also be smaller.

40. The external vertical load, σ_v , acting on the test section pipe can be estimated from the test section deflection data and Equation 6. From Figure 9, the maximum static deflection is 0.095 in. at zero cover depth. Using Equation 6, the corresponding external vertical load is computed to be 10.7 psi. This vertical load is substantially less than the vehicle contact pressure because of the small size of the vehicle footprint relative to the area of the pipe carrying the load. From Figure 10, the maximum predicted deflection is 0.475 in. at zero cover depth. Using Equation 6, the corresponding external vertical load is computed to be 53.4 psi. Using this load and Equations 2, 3, and 6-11, the maximum zero cover deflections of the pressurized pipes can be predicted. These deflections are shown below for a typical operating pressure of 735 psi.

	ΔX	
Pipe	in.	Percent
10 in.	0.217	2.17
19A-8 in.	0.055	0.68
19B-8 in.	0.166	2.07

PART IV: CONCLUSIONS

41. The type of steel pipe used in the 10-in. section of gas pipeline crossing the PCMS was instrumented and installed in an embankment test section. This test section was designed and constructed to emulate soil and pipe installation conditions at the PCMS. Each test section detail was tailored in an attempt to match the worst anticipated conditions along the pipeline. Test vehicles were also selected to produce more severe loads than those of anticipated PCMS traffic. Hence, observed test section deflections were greater than the actual deflections anticipated along the 10-in. section of the PCMS pipeline.

42. For cover depths of 0 to 30 in., the test section pipe was found to behave in the manner predicted by flexible-pipe theory. Rigid pipe theory, including load prediction and failure mode, is not appropriate. The deflection of an unpressurized pipe was shown, by theory, to be directly proportional to the external vertical load. This external vertical load is composed of dead-load and live-load components. The dead-load component was found to be slightly more than the weight of the soil prism directly above the pipe. The live-load was found to essentially follow the Boussinesq stress distribution for maximum vertical stress with depth. However, due to the small size of the vehicle footprints, the effective vertical load for deflection calculations was much less than that computed from theoretical contact pressures.

43. The impact factor was found to be highly variable. The largest impact factors were observed as vehicles crossed obstacles placed on the embankment above the pipe. These impact factors were, however, in no case as large as 5. This upper bound is consistent with tests conducted by Gemperline (1982), Schreiner and Green (1982), and others.

44. The largest total deflection observed in the test section was 0.273 in. Using an impact factor of 5, the maximum anticipated deflection for the test section is 0.475 in. This deflection occurs at zero cover depth. A 0.475-in. deflection is less than 5 percent of the nominal pipe diameter (10 in.) and thus within design limits.

45. The 10-in. pipeline in the PCMS can be expected to experience smaller deflections than the test section pipe and is thus within deflection design limits for cover depths of 0 to 30 in. The observed cover depth versus deflection relationships indicate that while live-loads become negligible

at greater depths, the dead-load will continue to increase linearly. Thus, a maximum cover depth must also be established to maintain the 5 percent design limit on deflection. Extrapolation of the test data suggests that this maximum cover depth is greater than 40 ft. The maximum cover depth at the PCMS is currently less than 13 ft.

46. Analysis using the Spangler deflection formula indicates that both absolute and percent deflections for both the 19A-8-in. and 19B-8-in. pipelines crossing the PCMS will be smaller than those of the 10-in. line.

47. Internal pressurization of the pipe has the effect of reducing the vertical load on the deflected pipe. Expected deflections in the operating (pressurized) pipeline crossing the PCMS are thus even less than those predicted by trafficking over the unpressurized test section pipe. The maximum total deflection at zero cover depth is estimated to be less than 2.2 percent.

48. The above findings indicate that the pipeline at the PCMS (pressurized or unpressurized) is not susceptible to damage from random crossings by anticipated military traffic with the maximum expected impacts as long as there is any cover over the pipeline. Further, any deflections which do occur, even at points of concentrated vehicle crossings, will be less than the recommended design limit of 5 percent of the nominal pipe diameter.

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Table 1

Soil Analysis, Observation Pit Samples, 3/7/83

Sample No.	Material	Weight, g			γ_w	γ_d	w
		Wet	Dry	Water	pcf	pcf	Percent
3	Backfill	380.27	343.49	36.28	79	72	10.5
10	Virgin	527.15	485.44	41.71	110	101	8.6
31	Backfill	482.52	453.52	28.88	101	95	6.4
34	Backfill	399.99	366.27	33.72	83	76	9.2

Note: Sample size: 2-7/8 in. diam by 2-3/16 in. high → 18.26 $in.^3$ = 0.010566 ft^3

Average for backfill (samples 3, 31, 34):

γ_w = 88 pcf

γ_d = 81 pcf

w = 8.7 percent

Table 2

Test Section Soil Conditions

Test section density + moisture measurements, 6/9/83

a. 24-in. cover depth, after traffic

Nuclear moisture-density gage, 6-in. depth, "normal" mode; Density Standard (DS) = 3,149, Moisture Standard (MS) = 534

(1) Roadway crown, 5 ft south of pipe; Density Count (DC) = 2,000, Moisture Count (MC) = 72

γ_w = 129.0 pcf

γ_D = 122.7 pcf

M = 6.4 pcf

W = 5.2%

(2) Roadway crown, directly over pipe; DC = 2,299, MC = 76

γ_w = 127.0 pcf

γ_D = 120.2 pcf

M = 6.8 pcf

W = 5.7%

(3) Roadway crown, 5 ft north of pipe; DC = 2,004, MC = 75

γ_w = 133.3 pcf

γ_D = 126.6 pcf

M = 6.7 pcf

W = 5.3%

b. 18-in. cover depth, before traffic

Nuclear moisture-density gage, 6-in. depth, "normal" mode; DS = 3,147, MS = 534

(1) Roadway crown, 5 ft south of pipe; DC = 2,104, MC = 74

γ_w = 131.0 pcf

γ_D = 122.3 pcf

M = 8.7 pcf

W = 7.1%

(Continued)

Table 2 (Concluded)

(2) Roadway crown, directly over pipe; DC = 2,250, MC = 86

γ_w = 127.9 pcf

γ_D = 120 pcf

M = 7.9 pcf

W = 6.6%

(3) Roadway crown, 5 ft north of pipe; DC = 2,304, MC = 106

γ_w = 126.7 pcf

γ_d = 116.7 pcf

M = 10.0 pcf

W = 8.6 percent

c. 18 in. cover depth, before traffic

Oven-dry moisture content, surface material, roadway crown

Sample No.	Location	Weight (g) Wet	Dry	Water	w Percent
103	5 ft south of pipe	268.6	256.9	11.7	4.6
133	Directly over pipe	279.3	264.6	14.7	5.6
167	5 ft north of pipe	290.1	272.5	17.6	6.5

d. 18-in. cover depth *after traffic*

2-7/8-in. diam by 2-13/16-in. high drive tubes, oven dry, roadway crown

Sample No.	Location	Weight (g) Wet	Dry	Water	γ_w pcf	γ_d pcf	w Percent
183	5 ft south of pipe	550.8	533.9	16.9	114.7	111.2	3.2
32	Directly over pipe	551.9	538.6	13.3	114.9	112.1	2.5
173	5 ft north of pipe	551.9	538.7	13.2	114.9	112.2	2.5

Nuclear moisture density gage registers densities approximately 7 pcf higher than drive-tube densities, and moisture contents approximately 1 percent higher than oven-dry moisture contents.

Table 3

Dead-Load Deflections

Cover Depth in.	Dead-Load (Soil) Deflection, in. Channel 1	2	3	4	Average	Remarks
30	0.024	0.023	0.029	0.026	0.025	Before traffic
30	0.030	0.030	0.036	0.033	0.032	During traffic
29	0.031	0.032	0.038	0.034	0.034	During traffic
29	0.031	0.033	0.037	0.033	0.033	During traffic
29	0.031	0.032	0.037	0.034	0.033	During traffic
29	0.031	0.032	0.037	0.034	0.033	After traffic
24	0.026	0.028	0.028	0.026	0.027	Before traffic
23	0.025	0.026	0.026	0.027	0.026	During traffic
23	0.025	0.026	0.026	0.027	0.026	During traffic
23	0.026	0.026	0.027	0.027	0.026	After traffic
23	0.025	0.029	0.026	0.026	0.026	After traffic
18	0.019	0.018	0.019	0.020	0.019	Before traffic
18	0.019	0.018	0.020	0.021	0.019	Before traffic
17	0.019	0.018	0.020	0.021	0.019	During traffic
17	0.019	0.018	0.019	0.020	0.019	During traffic
17	0.019	0.019	0.020	0.020	0.019	During traffic
17	0.022	0.022	0.023	0.023	0.022	During traffic
17	0.023	0.023	0.023	0.023	0.023	During traffic
17/14	0.019	0.018	0.029	0.019	0.019	Embankment/ditch above pipe
17/12	0.016	0.014	0.014	0.015	0.015	Embankment/ditch above pipe
17	0.016	0.015	0.015	0.016	0.015	After traffic
17	0.018	0.020	0.016	0.016	0.017	After traffic
12	0.012	0.012	0.011	0.012	0.012	Before traffic
12	0.012	0.012	0.011	0.012	0.012	Before traffic
12	0.013	0.015	0.011	0.012	0.013	Before traffic
12	0.012	0.012	0.011	0.012	0.012	During traffic
12	0.017	0.016	0.016	0.016	0.016	During traffic
11	0.016	0.015	0.014	0.015	0.015	After traffic
11	0.015	0.015	0.014	0.015	0.015	After traffic
8	0.008	0.007	0.006	0.007	0.007	Before traffic
8	0.007	0.005	0.005	0.007	0.006	During traffic
8	0.011	0.009	0.009	0.011	0.010	During traffic
7-1/2	0.008	0.007	0.007	0.010	0.008	During traffic
7	0.009	0.006	0.006	0.009	0.007	During traffic
7	0.009	0.007	0.007	0.009	0.008	During traffic
7	0.009	0.010	0.006	0.009	0.008	During traffic
7	0.006	0.010	0.006	0.009	0.008	During traffic
7	0.006	0.005	0.003	0.007	0.005	During traffic
7	0.006	0.004	0.004	0.007	0.005	During traffic
7	0.001	-0.002	-0.001	0.002	0.000	During traffic
7	-0.002	-0.001	-0.005	-0.001	-0.002	During traffic
7	-0.002	-0.005	-0.005	-0.001	-0.003	During traffic

(Continued)

Table 3 (Concluded)

Cover Depth in.	Dead-Load (Soil) Deflection, in.					Remarks
	Channel					
	1	2	3	4	Average	
8	0.003	0.004	-0.001	0.002	0.002	During traffic
8	0.002	0.001	-0.001	0.003	0.001	During traffic
8	0.005	0.002	0.001	0.005	0.003	After traffic
4	0.002	0.001	-0.002	0.001	0.001	Before traffic
4	0.009	0.012	0.010	0.009	0.010	During traffic
4	0.011	0.013	0.011	0.010	0.011	During traffic
4	0.009	0.009	0.010	0.009	0.009	After traffic
0	-0.000	-0.000	-0.001	-0.001	0.000	Before traffic
0	0.001	0.001	0.001	0.002	0.001	During traffic
0	0.001	0.002	0.002	0.002	0.002	During traffic
0	-0.007	-0.005	-0.005	-0.003	-0.005	After traffic

Table 4

Static Deflections, Live-Load, 5-Ton Truck

Cover Depth in.	Static Deflection, in. Channel 1	2	3	4	Interpolated Maximum, in.	Remarks
29	0.008	0.008	0.008	0.008	0.008	Voltmeter
29	0.011	0.010	0.010	0.010	0.010	Forward 1.5 mph
23	0.013	0.013	0.015	0.015	0.015	Voltmeter
23	0.014	0.015	0.015	0.014	0.015	Static
17	0.016	0.019	0.020	0.017	0.020	Static
17	0.017	0.020	0.020	0.019	0.020	Static
17	0.015	0.018	0.018	0.017	0.018	Static
14	0.013	0.013	0.013	0.012	0.013	Static - 3 in. ditch
14	0.012	0.013	0.014	0.012	0.014	Static - 3 in. ditch
12	0.017	0.019	0.019	0.018	0.019	Static - 5 in. ditch
11	0.023	0.025	0.025	0.025	0.025	Voltmeter
11	0.023	0.024	0.025	0.023	0.025	Static
7	0.040	0.045	0.044	0.039	0.045	Voltmeter
7	0.038	0.044	0.045	0.038	0.046	Static
7	0.035	0.041	0.045	0.040	0.044	Static
7	0.020	0.025	0.027	0.027	0.027	Static
4	0.052	0.056	0.057	0.052	0.057	Voltmeter
4	0.053	0.067	0.065	0.050	0.069	Static
0	0.049	0.062	0.062	0.051	0.064	Voltmeter
0	0.052	0.067	0.066	0.051	0.070	Static

Table 5

Dynamic Deflections, Live-Load, 5-Ton Truck

Cover Depth in.	Dynamic Deflection, in. Channel 1	2	3	4	Interpolated Maximum, in.	Remarks
29	0.012	0.012	0.013	0.011	0.013	Forward 5.5 mph
29	0.010	0.010	0.010	0.008	0.010	Forward 7.4 mph
29	0.012	0.012	0.011	0.011	0.012	Forward 8.2 mph
29	0.012	0.012	0.011	0.011	0.012	Forward stop
29	0.010	0.009	0.010	0.009	0.010	Forward stop
29	0.008	0.008	0.006	0.007	0.008	Forward stop
29	0.011	0.011	0.011	0.010	0.011	Forward stop
29	0.012	0.012	0.011	0.010	0.012	Forward stop
29	0.008	0.009	0.010	0.010	0.010	Reverse stop
29	0.011	0.012	0.013	0.011	0.013	Reverse stop
29	0.011	0.012	0.010	0.010	0.011	Reverse stop
23	0.010	0.011	0.013	0.012	0.012	Forward stop
23	0.011	0.011	0.013	0.012	0.012	Forward stop
23	0.010	0.010	0.011	0.010	0.011	Forward 7.7 mph
23	0.010	0.010	0.010	0.009	0.010	Forward 9.8 mph
23	0.012	0.012	0.012	0.012	0.012	Reverse stop
23	0.010	0.011	0.012	0.012	0.012	Reverse stop
23	0.012	0.013	0.013	0.013	0.013	On 2 × 4 in. board, 10 mph
23	0.009	0.009	0.009	0.009	0.009	Off 3 in. ramp @8 in. 3 mph
23	0.010	0.010	0.009	0.010	0.010	Off 3 in. ramp @8 in. 8.4 mph
23	0.010	0.011	0.012	0.011	0.012	Off 3 in. ramp @16 in. 3.9 mph
23	0.013	0.013	0.012	0.013	0.013	Off 3 in. ramp @16 in. 8.9 mph
23	0.014	0.015	0.015	0.014	0.015	On 3 × 4 in. board, 7.1 mph
23	0.013	0.015	0.015	0.016	0.016	On 3 × 4 in. board, 8.2 mph
23	0.013	0.014	0.015	0.014	0.015	On 3 × 4 in. board, 11.4 mph
23	0.014	0.014	0.015	0.014	0.015	On 3 × 4 in. board, 9.3 mph
17	0.015	0.018	0.019	0.016	0.019	Forward 3.2 mph
17	0.012	0.015	0.010	0.016	0.010	Reverse 2.6 mph
17	0.014	0.017	0.018	0.018	0.018	Forward 9 mph
17	0.013	0.016	0.017	0.016	0.017	Forward 15.2 mph
17	0.015	0.018	0.019	0.017	0.019	Forward 8.4 mph
17	0.015	0.018	0.019	0.016	0.019	Forward 15.8 mph
17	0.013	0.017	0.017	0.015	0.018	Forward 15.6 mph
17	0.015	0.017	0.017	0.017	0.017	Forward 15.8 mph
17	0.015	0.017	0.017	0.016	0.017	Forward 15.8 mph
17	0.015	0.019	0.020	0.018	0.020	Forward stop
17	0.011	0.012	0.013	0.012	0.013	Forward stop
17	0.010	0.011	0.013	0.012	0.012	Forward stop
17	0.011	0.012	0.014	0.011	0.013	Forward stop
17	0.013	0.015	0.015	0.014	0.015	Forward stop
17	0.011	0.013	0.014	0.013	0.014	Forward stop
17	0.012	0.014	0.015	0.013	0.015	Forward stop

(Continued)

(Sheet 1 of 3)

Table 5 (Continued)

Cover Depth in.	Dynamic Deflection, in. Channel 1	2	3	4	Interpolated Maximum, in.	Remarks
17	0.016	0.019	0.020	0.016	0.020	Forward stop
17	0.015	0.017	0.016	0.014	0.017	Forward stop
17	0.018	0.020	0.020	0.018	0.020	Forward stop
17	0.019	0.020	0.020	0.018	0.020	Forward stop
17	0.018	0.020	0.019	0.018	0.020	Forward stop
17	0.018	0.021	0.021	0.020	0.021	On 2 × 4 in. board, 10 mph
17	0.020	0.021	0.023	0.021	0.022	On 2 × 4 in. board, 10.4 mph
17	0.018	0.021	0.023	0.021	0.023	On 3 × 4 in. board, 10.1 mph
17	0.021	0.024	0.025	0.022	0.025	On 3 × 4 in. board, 10.6 mph
17	0.022	0.025	0.025	0.022	0.026	On 3 × 4 in. board, 11 mph
17	0.015	0.018	0.018	0.017	0.018	On 3 × 4 in. board, 2.3 mph
17	0.020	0.023	0.022	0.020	0.023	On 3 × 4 in. board, 6.8 mph
17	0.016	0.019	0.018	0.017	0.019	On 3 × 4 in. board, 2.6 mph
17	0.015	0.017	0.018	0.016	0.018	On 3 × 4 in. board, 2.6 mph
14	0.011	0.012	0.013	0.010	0.013	In 3 in. ditch, 2 mph
14	0.028	0.028	0.027	0.024	0.028	In 3 in. ditch, 8.4 mph
14	0.023	0.024	0.022	0.021	0.023	In 3 in. ditch, 10.6 mph
14	0.024	0.024	0.023	0.021	0.024	In 3 in. ditch, 9.4 mph
14	0.024	0.024	0.024	0.022	0.024	In 3 in. ditch, 14.6 mph
12	0.023	0.024	0.025	0.024	0.025	In 5 in. ditch, 3.4 mph
12	0.033	0.034	0.032	0.029	0.034	In 5 in. ditch, 8.3 mph
12	0.033	0.034	0.032	0.032	0.033	In 5 in. ditch, 8.7 mph
11	0.026	0.029	0.028	0.025	0.029	On 3 × 4 in. board, 0.5 mph
11	0.024	0.027	0.026	0.025	0.027	On 3 × 4 in. board, 7.9 mph
11	0.027	0.031	0.030	0.029	0.031	On 3 × 4 in. board, 10.2 mph
11	0.028	0.028	0.029	0.028	0.029	On 3 × 4 in. board, 10.2 mph
11	0.021	0.022	0.022	0.021	0.022	Forward 2.9 mph
11	0.018	0.021	0.022	0.021	0.022	Forward 7.6 mph
11	0.018	0.021	0.021	0.020	0.021	Forward 11.1 mph
11	0.017	0.020	0.020	0.019	0.020	Forward 12.0 mph
7	0.035	0.038	0.040	0.032	0.040	Forward 1.2 mph
7	0.033	0.038	0.037	0.034	0.039	Forward 2.9 mph
7	0.036	0.039	0.040	0.031	0.041	Forward 7.1 mph
7	0.040	0.046	0.050	0.040	0.050	On 3 × 4 in. board, 2.3 mph
7	0.047	0.051	0.050	0.038	0.052	On 3 × 4 in. baord, 9.6 mph
7	0.049	0.060	0.060	0.046	0.062	On 3 × 4 in. board, 10.3 mph
4	0.045	0.050	0.052	0.042	0.052	Forward 2.6 mph
4	0.047	0.049	0.054	0.049	0.052	Reverse 2.4 mph
4	0.045	0.049	0.051	0.047	0.051	Forward 8.5 mph
4	0.047	0.050	0.053	0.047	0.052	Forward 11.4 mph
4	0.043	0.048	0.050	0.047	0.050	Forward 11.7 mph

(Continued)

Table 5 (Concluded)

Cover Depth in.	Dynamic Deflection, in. Channel 1	2	3	4	Interpolated Maximum, in.	Remarks
4	0.053	0.072	0.068	0.056	0.073	On 3 × 4 in. board 2.2 mph forward
4	0.059	0.076	0.074	0.062	0.078	On 3 × 4 in. board 2.0 mph reverse
4	0.056	0.070	0.076	0.068	0.076	On 3 × 4 in. board 9.4 mph forward
4	0.080	0.092	0.089	0.076	0.093	On 3 × 4 in. board 11.5 mph forward
0	0.048	0.058	0.054	0.040	0.059	Forward 1.4 mph
0	0.048	0.061	0.050	0.047	0.061	Forward 2.6 mph
0	0.048	0.061	0.056	0.054	0.060	Forward 9.7 mph
0	0.048	0.061	0.061	0.050	0.063	Forward 12.3 mph
0	0.041	0.047	0.051	0.044	0.050	Forward 12.3 mph
0	0.041	0.054	0.054	0.044	0.056	Forward 12.3 mph

(Sheet 3 of 3)

Table 6

Static Deflections, Live-Load, Modified M48 Tank

Cover Depth in.	Static Deflection, in. Channel 1	2	3	5	Interpolated Maximum, in.	Remarks
30	0.024	0.025	0.025	0.024	0.025	Voltmeter
29	0.023	0.024	0.024	0.024	0.024	Forward 3 mph
29	0.029	0.025	0.026	0.025	0.026	Reverse 2 mph
23	0.024	0.026	0.025	0.023	0.026	Voltmeter
23	0.024	0.025	0.026	0.025	0.026	Forward 2 mph
23	0.026	0.025	0.025	0.024	0.026	Static
23	0.024	0.027	0.028	0.026	0.028	Static
23	0.026	0.029	0.029	0.027	0.029	Static
17	0.025	0.028	0.028	0.027	0.028	Static
17	0.031	0.033	0.034	0.030	0.034	Static
12	0.033	0.034	0.037	0.033	0.036	Voltmeter
12	0.037	0.040	0.042	0.036	0.042	Static
11	0.034	0.039	0.041	0.035	0.041	Forward 2.3 mph
8	0.054	0.063	0.069	0.061	0.068	Static
7	0.051	0.058	0.060	0.050	0.061	Voltmeter
7	0.049	0.058	0.059	0.050	0.060	Static
4	0.041	0.049	0.049	0.041	0.051	Voltmeter
4	0.085	0.087	0.089	0.087	0.089	Voltmeter-crossways
4	0.039	0.049	0.047	0.038	0.050	Static
0	0.068	0.091	0.090	0.071	0.095	Voltmeter
0	0.065	0.088	0.091	0.076	0.093	Voltmeter-crossways
0	0.069	0.087	0.083	0.074	0.088	Static
0	0.069	0.084	0.083	0.078	0.086	Static-crossways

Table 7

Dynamic Deflections Live-Load, Modified M48 Tank

Cover Depth in.	Dynamic Deflection, in. Channel 1	2	3	4	Interpolated Maximum, in.	Remarks
16.3	0.069	0.074	0.076	0.068	0.076	12 in. hump @12 ft, 7.6 mph
15.8	0.086	0.101	0.088	0.085	0.096	12 in. hump @12 ft, 7.9 mph
15.3	0.048	0.050	0.043	0.034	0.050	12 in. hump @12 ft, 8.9 mph
14.8	0.112	0.126	0.128	0.107	0.130	12 in. hump @12 ft, 8.9 mph
14.5	0.147	0.157	0.151	0.130	0.158	12 in. hump @12 ft, 9.8 mph
14.1	0.112	0.117	0.125	0.112	0.123	12 in. hump @12 ft, 10.2 mph
13.9	0.118	0.129	0.128	0.117	0.131	12 in. hump @12 ft, 10.3 mph
13.7	0.117	0.126	0.135	0.132	0.134	12 in. hump @12 ft, 10.6 mph
13.5	0.096	0.105	0.106	0.075	0.107	12 in. hump @12 ft, 10.3 mph
29.0	0.023	0.024	0.025	0.024	0.025	Forward 5.6 mph
29.0	0.023	0.025	0.024	0.024	0.025	Forward stop
29.0	0.028	0.030	0.031	0.029	0.031	Forward stop
29.0	0.033	0.032	0.032	0.030	0.033	Forward stop
23.0	0.027	0.028	0.028	0.026	0.028	Forward accelerate
23.0	0.026	0.028	0.028	0.030	0.030	Forward stop
23.0	0.025	0.026	0.028	0.027	0.028	Reverse accelerate
23.0	0.031	0.032	0.032	0.026	0.033	Forward stop
23.0	0.026	0.027	0.028	0.025	0.028	Forward stop
23.0	0.026	0.026	0.028	0.026	0.027	Reverse accelerate
23.0	0.028	0.028	0.028	0.026	0.028	Forward stop
23.0	0.028	0.029	0.029	0.028	0.029	Forward stop
23.0	0.023	0.025	0.027	0.027	0.027	Forward stop
23.0	0.021	0.025	0.027	0.027	0.027	Forward stop
23.0	0.022	0.025	0.026	0.026	0.026	Reverse accelerate
23.0	0.023	0.026	0.027	0.024	0.027	Reverse accelerate
23.0	0.017	0.018	0.020	0.018	0.019	Forward stop
23.0	0.026	0.026	0.026	0.026	0.026	Forward stop
23.0	0.022	0.023	0.025	0.022	0.024	Forward stop
23.0	0.026	0.026	0.027	0.026	0.027	Reverse accelerate
23.0	0.026	0.028	0.028	0.026	0.028	Reverse accelerate
23.0	0.026	0.027	0.029	0.027	0.028	Forward stop
23.0	0.027	0.027	0.028	0.027	0.028	Reverse stop
23.0	0.021	0.024	0.024	0.025	0.025	Forward 9.5 mph
17.0	0.030	0.031	0.031	0.026	0.032	Forward 3.7 mph
17.0	0.029	0.033	0.032	0.028	0.033	Forward 9.5 mph
17.0	0.030	0.032	0.033	0.030	0.033	Forward 12.7 mph
17.0	0.027	0.030	0.032	0.028	0.032	Forward 11.5 mph
17.0	0.028	0.029	0.031	0.027	0.030	Forward stop
17.0	0.029	0.030	0.030	0.028	0.030	Forward stop
17.0	0.029	0.031	0.032	0.029	0.032	Forward 7.2 mph

(Continued)

Table 7 (Continued)

r h	Dynamic Deflection, in. Channel 1	2	3	4	Interpolated Maximum, in.	Remarks
0	0.046	0.051	0.049	0.042	0.051	On 3 × 4 in. board, 0.9 mph
0	0.041	0.046	0.046	0.039	0.047	On 3 × 4 in. board, 2.9 mph
0	0.042	0.046	0.046	0.042	0.047	On 3 × 4 in. board, 13 mph
0	0.014	0.013	0.014	0.014	0.014	In 3 in. ditch, 1.6 mph
0	0.021	0.019	0.020	0.019	0.020	In 3 in. ditch, 11.2 mph
0	0.019	0.020	0.019	0.017	0.020	In 3 in. ditch, 11.5 mph
0	0.034	0.037	0.041	0.035	0.041	Forward 2.3 mph
0	0.032	0.037	0.037	0.034	0.038	Forward 4.4 mph
0	0.030	0.037	0.036	0.033	0.037	Forward 10.6 mph
0	0.033	0.037	0.037	0.036	0.038	Forward 13 mph
0	0.032	0.036	0.036	0.031	0.037	Forward 11.5 mph
0	0.062	0.071	0.070	0.063	0.072	On 3 × 4 in. board, 1.2 mph
0	0.055	0.062	0.059	0.053	0.062	On 3 × 4 in. board, 3.7 mph
0	0.046	0.052	0.051	0.046	0.053	On 3 × 4 in. board, 11 mph
0	0.053	0.063	0.064	0.053	0.066	Forward 2 mph
0	0.047	0.053	0.052	0.039	0.055	Forward 6.9 mph
0	0.050	0.061	0.063	0.053	0.064	Forward 10.5 mph
0	0.047	0.053	0.054	0.047	0.055	Forward 9.9 mph
0	0.103	0.120	0.120	0.105	0.123	On 3 × 4 in. board, 1.2 mph
0	0.080	0.092	0.091	0.081	0.094	On 3 × 4 in. board, 6 mph
0	0.068	0.090	0.083	0.078	0.084	On 3 × 4 in. board, 10.6 mph
0	0.088	0.099	0.094	0.086	0.098	On 3 × 4 in. board, 10.4 mph
0	0.028	0.033	0.032	0.030	0.033	Forward 2 mph
0	0.043	0.054	0.055	0.052	0.056	Reverse accelerate
0	0.048	0.055	0.054	0.046	0.056	Reverse accelerate
0	0.045	0.056	0.054	0.045	0.057	Forward stop
0	0.045	0.058	0.060	0.053	0.061	Forward stop
0	0.043	0.057	0.060	0.052	0.061	Forward stop
0	0.041	0.054	0.053	0.049	0.055	Reverse accelerate
.0	0.047	0.054	0.053	0.044	0.055	Reverse accelerate
.0	0.039	0.049	0.048	0.045	0.050	Reverse accelerate
.0	0.042	0.054	0.053	0.046	0.055	Reverse accelerate
.0	0.044	0.055	0.053	0.042	0.056	Forward stop
.0	0.045	0.058	0.057	0.048	0.060	Forward stop
.0	0.036	0.046	0.046	0.036	0.048	Forward 12 in. hump @8 ft, 4.6 mph
.0	0.030	0.039	0.040	0.032	0.041	Forward 12 in. hump @2 ft, 3 mph
.0	0.060	0.068	0.064	0.045	0.069	Forward 12 in. hump @8 ft, 11 mph
.0	0.062	0.075	0.072	0.056	0.076	Forward 12 in. hump @8 ft, 11.5 mph

(Continued)

(Sheet 2 of 3)

Table 7 (Concluded)

Cover Depth in.	Dynamic Deflection, in. Channel 1	2	3	4	Interpolated Maximum, in.	Remarks
4.0	0.041	0.055	0.052	0.041	0.056	Forward 2.2 mph
4.0	0.042	0.055	0.052	0.043	0.056	Reverse 1.9 mph
4.0	0.040	0.043	0.043	0.041	0.043	Forward 5.1 mph
4.0	0.043	0.055	0.055	0.045	0.057	Reverse 2.7 mph
4.0	0.056	0.067	0.059	0.047	0.066	Forward 10.5 mph
4.0	0.043	0.049	0.049	0.043	0.050	Forward 12.2 mph
4.0	0.133	0.168	0.172	0.139	0.177	On 3 × 4 in. board, 2.3 mph forward
4.0	0.120	0.150	0.153	0.124	0.157	On 3 × 4 in. board, 2.3 mph reverse
4.0	0.232	0.265	0.260	0.220	0.269	On 3 × 4 in. board, 11.9 mph forward
4.0	0.134	0.164	0.170	0.146	0.173	On 3 × 4 in. board, 2.4 mph reverse
4.0	0.218	0.260	0.259	0.203	0.269	On 3 × 4 in. board, 11.9 mph forward
0.0	0.067	0.101	0.100	0.068	0.106	Forward 1.75 mph
0.0	0.076	0.118	0.113	0.083	0.172	Reverse 1.3 mph
0.0	0.076	0.101	0.100	0.085	0.104	Forward 5.25 mph
0.0	0.079	0.124	0.123	0.081	0.132	Reverse 2.6 mph
0.0	0.089	0.134	0.133	0.085	0.142	Forward 9.2 mph
0.0	0.069	0.091	0.083	0.081	0.089	Forward 10.5 mph
0.0	0.082	0.098	0.098	0.084	0.101	Reverse 2.7 mph
0.0	0.079	0.098	0.097	0.086	0.100	Forward 11.2 mph
0.0	0.084	0.108	0.105	0.087	0.110	Reverse 2.8 mph
0.0	0.086	0.115	0.112	0.092	0.118	Forward 12.6 mph
0.0	0.072	0.112	0.109	0.091	0.116	Reverse 2.7 mph
0.0	0.089	0.127	0.133	0.085	0.138	Forward 11.2 mph
0.0	0.082	0.115	0.112	0.091	0.119	Reverse 2.6 mph

(Sheet 3 of 3)

Table 8

Wheeled Test Vehicle Impact Factors

Depth in.	Deflection, in. Static	Dynamic	Impact Factor	Remarks
29	0.010	0.013	1.3	Fwd, 5.5 mph
29	0.010	0.012	1.2	Fwd/Stop
29	0.010	0.013	1.3	Rev/Stop
23	0.014	0.012	0.9	Fwd/Stop
23	0.014	0.011	0.8	Fwd, 7.7 mph
23	0.014	0.012	0.9	Rev/Stop
23	0.014	0.013	0.9	Over 2 × 4 in. board, 10 mph
23	0.014	0.013	0.9	Off 3 in. ramp, 8.9 mph
23	0.014	0.016	1.1	Over 3 × 4 in. board, 8.3 mph
17	0.020	0.019	1.0	Fwd, 3.2 - 15.8 mph
17	0.020	0.016	0.8	Rev, 2.6 mph
17	0.020	0.020	1.0	Fwd/Stop
17	0.020	0.022	1.1	Over 2 × 4 in. board, 10.4 mph
17	0.020	0.026	1.3	Over 3 × 4 in. board, 11 mph
14	0.025	0.028	1.1	In 3 in. ditch over pipe, 8.4 mph
12	0.029	0.034	1.2	In 5 in. ditch over pipe, 8.3 mph
11	0.031	0.022	0.7	Fwd, 2.9 - 7.6 mph
11	0.031	0.031	1.0	Over 3 × 4 in. board, 10.2 mph
7	0.046	0.041	0.9	Fwd, 7.1 mph
7	0.046	0.062	1.3	Over 3 × 4 in. board, 10.3 mph
4	0.062	0.052	0.8	Fwd, 2.4 - 11.4 mph
4	0.062	0.093	1.5	Over 3 × 4 in. board, 11.5 mph
0	0.070	0.063	0.9	Fwd, 12.3 mph

Table 9

Tracked Test Vehicle Impact Factors

Depth in.	Deflection, in. Static	Deflection, in. Dynamic	Impact Factor	Remarks
29	0.026	0.025	1.0	Fwd, 5.6 mph
29	0.026	0.033	1.3	Fwd/Stop
23	0.029	0.028	1.0	Fwd/Accel
23	0.029	0.028	1.0	Rev/Accel
23	0.029	0.033	1.1	Fwd/Stop
23	0.029	0.028	1.0	Rev/Stop
17	0.035	0.033	0.9	Fwd, 9.5 - 12.7
17	0.035	0.030	0.9	Fwd/Stop
17	0.035	0.051	1.5	Over 3 × 4 in. board, 0.9 mph
16.3	0.036	0.076	2.1	Over 12 in. hump 12 ft from pipe, 7.6 mph
15.8	0.037	0.096	2.6	Over 12 in. hump 12 ft from pipe, 7.9 mph
15.3	0.038	0.050	1.3	Over 12 in. hump 12 ft from pipe, 8.0 mph
14.8	0.039	0.130	3.3	Over 12 in. hump 12 ft from pipe, 8.7 mph
14.5	0.039	0.158	4.1	Over 12 in. hump 12 ft from pipe, 9.8 mph
14.1	0.040	0.123	3.1	Over 12 in. hump 12 ft from pipe, 10.2 mph
13.9	0.041	0.131	3.2	Over 12 in. hump 12 ft from pipe, 10.3 mph
13.7	0.041	0.134	3.3	Over 12 in. hump 12 ft from pipe, 10.6 mph
13.5	0.042	0.107	2.5	Over 12 in. hump 12 ft from pipe, 10.3 mph
14	0.041	0.020	0.5	In 3 in. ditch over pipe, 11.5 mph
11	0.050	0.041	0.8	Fwd, 2.3 mph
11	0.050	0.072	1.4	Over 3 × 4 in. board, 1.2 mph
7	0.074	0.066	0.9	Fwd, 2 mph
7	0.074	0.123	1.7	Over 3 × 4 in. board, 1.2 mph
7	0.074	0.056	0.8	Rev/Accel
7	0.074	0.061	0.8	Fwd/Stop
7	0.074	0.076	1.0	Over 12 in. hump 8 ft from pipe, 11.5 mph
4	0.089	0.066	0.7	Fwd, 10.5 mph
4	0.089	0.057	0.6	Rev, 2.7 mph
4	0.089	0.269	3.0	Over 3 × 4 in. board, 11.9 mph
0	0.095	0.106	1.1	Fwd, 1.75 mph
0	0.095	0.122	1.3	Rev, 1.3 mph

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