THE CAPACITY OF CORRUGATED PLASTIC TUBING TO SUPPORT EARTH LOADS

by

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Plastic Tubing Placed Under Deep Earth Loads.

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ABSTRACT

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THE CAPACITY OF CORRUGATED PLASTIC TUBING TO SUPPORT EARTH LOADS

Deflections of corrugated plastic tubes installed at depths varying from 0.9 to 8.2 metres were measured over a period of four years. Tubing varied from 80 mm to 300 mm in diameter.

The 80 mm and 100 mm tubes suffered excessive deflection for fill depths greater than 3.5 metres. The 150 mm, 200 mm and 250 mm diameter tubes carried the maximum depth of fill of 8.2 metres satisfactorily. The 300 mm diameter tubes showed deflection of 20.2 per cent for fill of 5.0 metres and 31.1 per cent for fill up to 8.2 metres.

Deflection was related to bedding condition, stiffness of tubing and load on the conduit.

RESUME

M.Sc. GILLES FERNAND BOLDUC Génie Rurale

LA CAPACITE PORTANTE DES TUYAUX DE PLASTIQUE ONDULE
SONS UN REMBLAI

L'affaissement de tuyaux de plastique ondulés, installés sous un remblai variant de 0,9 à 8.2 mètres, fût mesurés sur une période de quatre ans. Le diamètre intérieur des tuyaux variait entre 80 mm et 300 mm.

Les tuyaux de 80 mm et 100 mm subirent un affaissement très prononcé sous un remblai de plus de 3.5 mètres. Les tuyaux de 150 mm, 200 mm et 250 mm ont supporté la charge sous un remblai de 8.2 mètres de façon satisfaisante. Un tuyau de 300 mm s'est affaissé de 20.0 pourcent sous un remblai de 4.5 mètres et de 31.1 pourcent sous un remblai de 8.2 mètres. La moyenne de l'affaissement pour les tuyaux de 300 mm se situe à 14.1 pourcent à 4.5 mètres et 22.4 pourcent à 8.2 mètres.

La préparation du lit, la rigidité du tuyau ainsi que la charge sur ce dernier furent associés à l'affaissement.

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LIST OF ABBREVIATIONS

ASTM	American Society for Testing of Material
cm	centimetre
CPTA	Corrugated Plastic Tubing Association
ft	foot
g	gram
H	height of fill measured from the top of the tubing
I.D.	inside diameter
in	inch
kN	kiloNewton
kPa	kiloPascal
lb	pound
m	metre
mm	millimetre
MPa	megaPascal
NONP	non perforated
O.D.	outside diameter
PE	polyethylene
PERF	perforated
PPR	polypropylene
WPCF	Water Pollution Control Federation

TABLE OF CONVERSION OF SOME IMPORTANT TERMS

Load

1 lb/in	=	0.1752 kN/m

Stiffness

1 lb/in/in (psi)	=	0.1450 kN/m/m (kPa)

Specific density

1 lb/in^3	=	0.03613 g/cm^3

I. INTRODUCTION

Hundreds of thousands of kilometres of corrugated plastic drain tubing have been installed at depths of 0.7 to two metres for farm drainage with good success. However, when applications which require tubing to be installed at depths greater than two metres are considered, many designers, specification writers and manufacturers are uncertain whether to use standard corrugated plastic tubing, or whether tubing with thicker walls should be specified. There are no well documented cases of the performance of corrugated plastic installed deeper than two metres.

Some applications requiring tubing at greater depth are: drainage around the footings of deep foundations, drains passing under highway embankments, toe drains for dikes and dams, drains for mine tailing piles, city dump and landfills, drains under building floors and storage piles, dewatering drains under sewer and water lines, and some deeply placed communications wire conduits.

Recognizing the need for installation of corrugated plastic tubing with depth of installation increasing up to at least seven metres, the CPTA authorized the installation and study reported herein.

Objectives

The objectives of this work were:

1. To install tubing, in an accessible location, with depths of backfill ranging from zero to at least seven metres.
2. To measure the inside diameter along the length of the tubing during installation, and from time to time after installation.
3. To try to determine whether the reduction in inside diameter (deflection) has a direct relationship to the depth or weight of soil above the tubing.
4. To determine whether there is any long term creep deflection of the tubing.
5. To use tubing in standard sizes available from 80 mm to 304 mm (three to 12 inches) I.D.

Scope limitations

1. Fill depth was limited to 8.2 metres because of the costs involved in going to greater depths at the available experimental site.
2. Commercially available corrugated tubing was used. It was not feasible to obtain tubing with a specific stiffness rating.
3. It was beyond the allowed budget to develop a deflection measuring device.

II. REVIEW OF LITERATURE

A. Theory of load on underground conduits

Since the beginning of the twentieth century, research on the loading of underground conduits has been carried on by engineering organizations and individuals. Serious efforts have been made to analyse external loads and earth pressures to which underground conduits are subjected.

1. Load classification of underground conduits

Underground conduits are classified according to their degree of flexibility. For instance, conduits such as asbestos-cement, concrete, cast iron and clay are considered rigid, whereas corrugated metal, thin walled steel pipe and corrugated plastic pipe are classified as flexible pipe. The principal load-supporting ability of rigid conduits lies in the inherent strength or stiffness of the pipe while flexible conduit relies only partly on the inherent strength of the pipe when resisting external loads; that is, the flexible pipe mobilizes the soil passive resistance to give support to the pipe as the horizontal diameter and deflection of the pipe increase. In addition, underground conduits are classified according to the construction conditions under which they are installed. Three main classes characterize the construction conditions; these are ditch conduits, projecting conduits and negative projecting conduits.

Ditch conduit conditions apply to structures installed and buried in narrow trenches in relatively passive and undisturbed soils. The imperfect ditch method of construction changes projecting conduit conditions to ditch conduit conditions by means of tamping the soil on both sides and above the conduit for some distance.

Projecting conduit conditions apply to structures installed in shallow bedding with the top of the conduit projecting above the original soil surface. Conduits installed in trenches two or three times larger than the horizontal breadth of the conduit are considered to be in projecting conduit conditions.

Negative projecting conduit conditions apply to structures placed in shallow ditches with the top of the conduit below the natural ground surface and covered with an embankment above this ground level.

The terms «ditch conduit, imperfect ditch, projecting and negative projecting conduits» have become established in civil engineering terminology. These terms are identified in the WPCF Manual no 9 and are illustrated in Figure 1.

2. Loads due to fill materials

There was no rational method, prior to 1900, to characterize and determine the magnitude of the loads on underground conduits due to earth overburden and other load soufces. In the early years of the

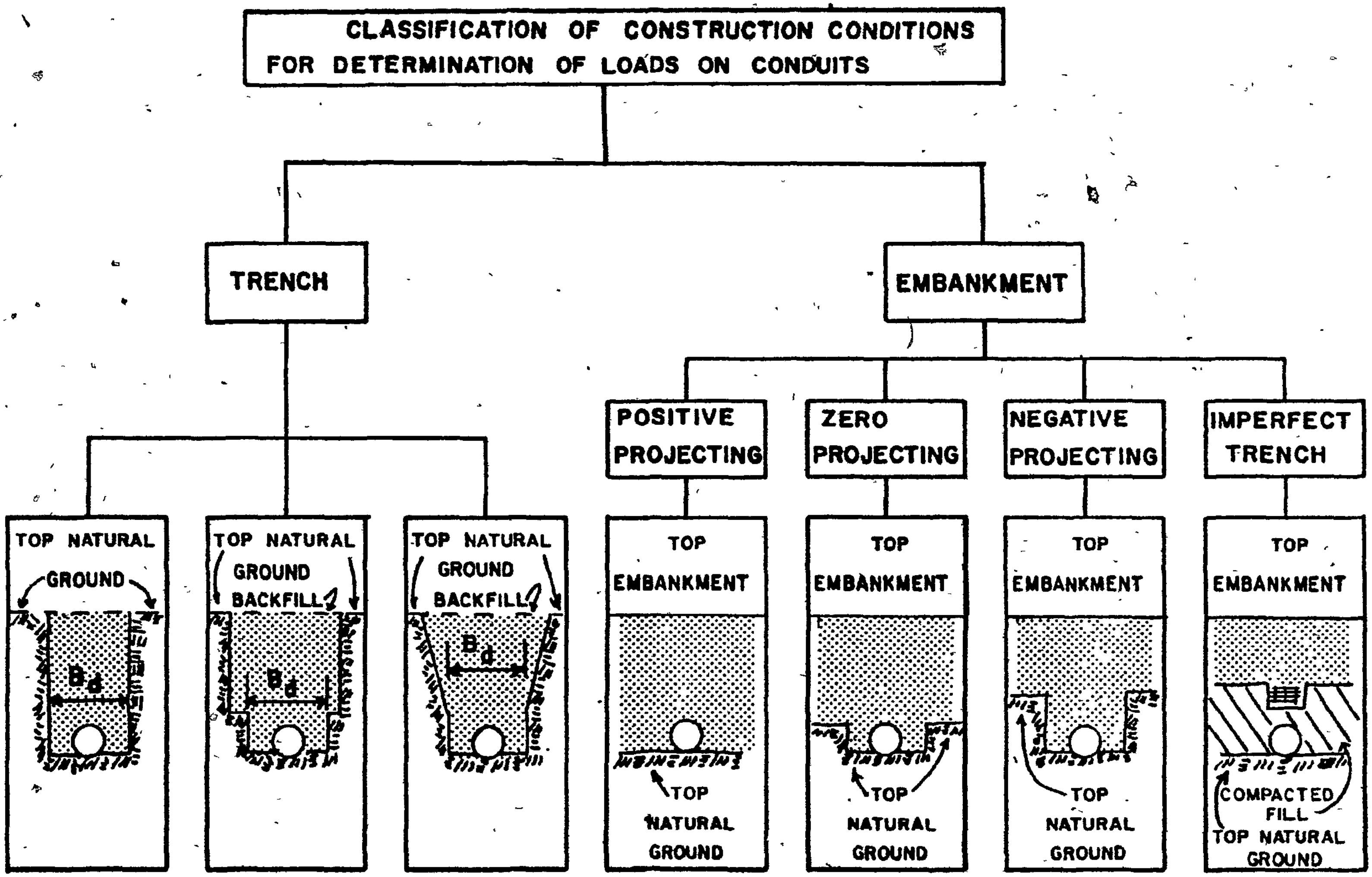


Figure 1. Classification of construction conditions

twentieth century the Marston theory (Spangler and Hardy, 1973) was developed and, since then, much loading research has verified the validity of this approach.

The basic concept of the Marston theory for ditch conduit is that, as the fill material settles into the ditch, friction forces which act upward develop between the side of the ditch and the fill material, creating an arch action that helps to support the soil backfill. Therefore, the load on the pipe is less than the total weight of the soil column above it. This concept is based on the assumption that the fill density is less than that of the original soil. According to the theory the load on the conduit for the case of flexible ditch conduit is given by the following formula.

$$W_c = C_d \gamma B_c B_d \qquad \ldots (1)$$

where

W_c is the dead load FL^{-1} of conduit length to be carried by the flexible conduit

C_d is a loading coefficient dependent on depth of backfill, width of trench and type of soil

γ is the specific weight of backfill soil, FL^{-3}.

B_c is the outside diameter of the conduit, L

B_d is the width of the trench, L

In this case it is assumed that the side fills have been thoroughly tamped and have essentially the same degree of stiffness as the pipe.

Marston's theory includes also the case of projecting conduit conditions and the equation that applies to such conditions is given in the following form:

$$W_c = C_c \gamma B_c^2 \qquad \ldots (2)$$

where

W_c is the load on the conduit in FL^{-1} of conduit length to be carried by the flexible conduit

C_c is a loading coefficient dependent on the depth of backfill, type and outside width of conduit and soil conditions

γ is the specific weight of backfill soil, FL^{-3}

B_c is the outside diameter of the conduit, L

In order to determine the value of the loading coefficient, two factors associated with the conditions under which the conduit is installed have to be considered: the projection ratio, p, and the settlement ratio, r_{sd}. The projection ratio, p, is defined as the ratio of the distance that the top of the conduit projects above the adjacent natural ground surface or the top of the thoroughly compacted fill to the vertical outside height of the conduit.

The settlement ratio is defined in WPCF Manual no. 9, by the following relationship:

$$r_{sd} = \frac{(S_m + S_g) - (S_f + d_c)}{S_m} \qquad \ldots (3)$$

where

r_{sd} is the settlement ratio

S_g is the settlement of the natural ground adjacent to the conduit

S_m is the compression of the columns of soils of height pB_c

$(S_m + S_g)$ is the settlement of the critical plane

d_c is the vertical deflection of the conduit

S_f is the settlement of the bottom of the conduit

$(S_f + d_c)$ is the settlement of the top of the conduit

The WPCF Manual no. 9, recommends the following r_{sd} design values for flexible conduits:

Soil conditions	r_{sd}
Poorly-compacted side fills	-0.4 to 0
Well-compacted side fills	0

3. Surface loads

Underground conduits very often are subjected to loads other than the filling material. Traffic and piles of construction materials at the surface transmit their loads through the fill material to the underground structure. These loads can be concentrated, as in the case of truck wheels, or uniformly distributed, as in the case of piles of construction materials. The effect is greater for shallow covering, but dissipates and spreads rapidly as the depth of cover increases to 1.2 metre or more.

According to Taylor (1948), Boussinesq, in 1885, obtained a general solution of the elastic equations under a point load that was applied to a semi-infinite mass. Sprangler (1947) reported that Griffith was one of the first to suggest the applicability of the Boussinesq solution to problems of transmission of stress through soils and that he had developed a generalized expression for the intensity of vertical pressure in a soil mass due to a concentrated surface load in the form:

$$P_z = \frac{V}{2n} P_\phi \frac{\cos^{v+2}\theta}{Z^2} \quad \ldots (4)$$

where

P_z is the intensity of pressure at a point in the soil mass

P_o is the concentrated load applied at a point on the surface

θ is the angle formed with the vertical by the radius vector from the point of application of the surface load to the point considered

Z is the vertical distance from the surface to the point

V is a concentration or dispersion factor.

Holl's integration of Boussinesq's formula for concentrated load is presented in WPCF Manual no. 9 in the following form:

$$W_{sc} = C_s \frac{PF}{L} \quad \ldots (5)$$

where

W_{sc} is the live load per unit conduit length, FL^{-1}

C_s is the loading coefficient obtained

P is half axle load, F

F is the impact factor of 1.25 for field travel or 1.5 for highways and streets

L is the effective length of conduit over which live load acts, normally taken as 0.9 meter.

For the case of a superimposed load distributed over an area of considerable extent, the formula is:

$$W_{sd} = C_s p F B_c \qquad \ldots (6)$$

where

W_{sd} is the load per unit conduit length, FL^{-1}

p is the intensity of distributed load, FL^{-2}

F is the impact factor

B_c is the width of the conduit, L

C_s is a loading coefficient.

4. Deflection

Some research has been done to estimate the deflection of flexible underground conduits. Flexible conduits fail by excessive deflection as opposed to rigid conduits which fail by rupture of the pipe walls. Flexible pipe resistance to deflection is a function of both the inherent strength of the pipe and its surrounding soil. Spangler (1947) presented the following equation for deflection.

$$\Delta X = \frac{(D_e K W_c + K W_{sc}) r^3}{E\,I + 0.061\,E' r^3} \qquad \ldots (7)$$

where

ΔX is the horizontal deflection

D_e is the deflection lag factor

K is the bedding constant

W_c is the dead load on the conduit, FL^{-1}

W_{sc} is the live load on the conduit, FL^{-1}

r is the mean radius of the conduit, L

E is the modulus of elasticity of the conduit, FL^{-2}

I is the second moment of area of the conduit per unit length, L^3

E' is the modulus of soil reaction, FL^{-2}

Usually, vertical rather than horizontal deflection is measured in the field. Howard (1977) states that the ratio between the vertical and horizontal diameter changes as a circular section deforms elliptically, and gives the following relationship:

$$\Delta X = 0.913\ \Delta Y \qquad \ldots (8)$$

The product EI represents the resistance of the conduit to deflection, and, for a specific conduit wall shape, EI can be determined in the laboratory with parallel plate loading and the Spangler (1973) equation.

$$EI = 0.149\ r^3 \left(\frac{F}{\Delta Y}\right)$$

where

ΔY is the vertical deflection, L

F is the vertical load/unit length on conduit, FL^{-1}.

Figure 6 provides a definition sketch for the terms involved in the measurement of pipe stiffness and deflection.

Putting equations (7), (8) and (9) together yields:

$$\Delta X = 0.913\ \Delta Y = \frac{(D_e KW_c + KW_{sc})r^3}{0.149\left(\frac{F}{\Delta Y}\right)r^3 + 0.061\ E'r^3} \qquad \ldots (10)$$

which simplifies to:

$$\Delta X = \Delta Y = \frac{D_e KW_c + KW_{sc}}{0.136\ \frac{F}{\Delta Y} + 0.056\ E'} \qquad \ldots (11)$$

The expression is applicable to the investigation of plastic drain tube deflections under earth loads. The apparent weakness of predictions based on this equation lies in the selection of proper values for the modulus of soil reaction and uncertainties in the values of the deflection lag factor, D_e.

Howard (1977) stated that for a given degree of compaction and soil type, the soil passive resistance varies. While Spangler (1973) recommended a value for D_e of 1.5 to be used in the equation, Fenemor (1978) found values ranging from 1.5 to 8.6 and an average of 3.4. Shafer (1947) stated that the flexible pipes under external loads continue to function structurally until the deflection results in a concave curvature of the top or bottom of the pipe. He recommended the safe maximum deflection as 20 per cent of the vertical diameter and using a conservative factor of safety of 4, established the design deflection at 5 per cent.

McCandless (1974) studied deflection for tubing installed at depths less than five feet in different types of soils and found deflections in the order of 18 per cent. Tubing in sandy soils generally showed less deflection, which he attributed to the tendency of sand to flow around the pipe during blinding operations, thus providing better quality bedding and more lateral support.

Schwab and Drablos (1975) reported an average overall maximum deflection of 17.2 per cent for farm sites they investigated. The sizes of pipe inspected were 250, 300 and 380 mm diameter. They also found from field tests that 80 per cent of the deflection occurred during the first two years after installation.

Drablos and Schwab (1972) and Drablos and Walker (1976) found, from extensive sampling of farm drainage tubing installed at depths less than 1.4 metre, that many 100, 125, 150, 200 and 250 mm diameter tubes had deflections in excess of 25 per cent and means were from 20 to 26 per cent for tubes with stiffness greater than 125 kPa. The tubes did not collapse and continued to carry water.

Fenemor _et al._ (1978) found average deflections of 6 and 12 per cent respectively for pipes buried in narrow and wide trenches at a depth of 1.4 metre for sizes of 150, 200, 250, 300 and 380 mm in diameter.

III. MATERIALS AND METHODS

1. Field experiment

Four replicates of each of the seven types of tubing indicated in Table 1 were installed in the side of a hill on the Macdonald College Farm. This arrangement left the tubing accessible for many years, used a minimum area of land and reduced earth-moving costs compared with some other arrangements. One end of the tubing remained open for measurement access, while the other end was buried at the maximum depth. Figures 2 and 3 show the profile and the front view of the hill with the tubing installation.

Excavation of the hillside started on 15 November 1976, using a power shovel and a Caterpillar 955 crawler tractor for pushing the soil aside. The excavation was done in two cuts: the first, 4.5 metres deep, and the second, 3 metres deep. The soil on the hill consisted of about 1.2 metres of sand underlain by a soft sticky blue clay which made excavation and soil removal difficult. The excavation finished on 1 December 1976. The cut measured 28 metres wide and 24 metres back into the hill. The maximum depth was 8.2 metres and the side slopes were about one to one.

Five lines were installed in the bottom of the excavated area to a depth of approximately 0.75 metres using a Buckeye trencher (see Figures 2 and 3). These included three 100 mm I.D. perforated filter

TABLE 1. Manufacturer, material and stiffness of tubing used

Inside diameter nominal (inches)	Actual diameter		Manufacturer	Material	Stiffness	
	Inches	mm			5% deflection (kPa)	10% deflection (kPa)
3	3.15	80.1	Daymond Ltd.	PE	255	192
4	3.98	101.1	Drainbec Ltd.	PE	298	228
4	4.00	101.6	Hercules Canada Ltd.	PPR	288	233
6	6.05	153.7	Big «O» Plastics Ltd.	PE	210	173
8	8.15	207.1	Big «O» Plastics Ltd.	PE	265	201
10	10.07	255.7	Advanced Drainage Systems	PE	283	220
12	11.80	299.7	Universal Plastics Inc.	PE	283	231

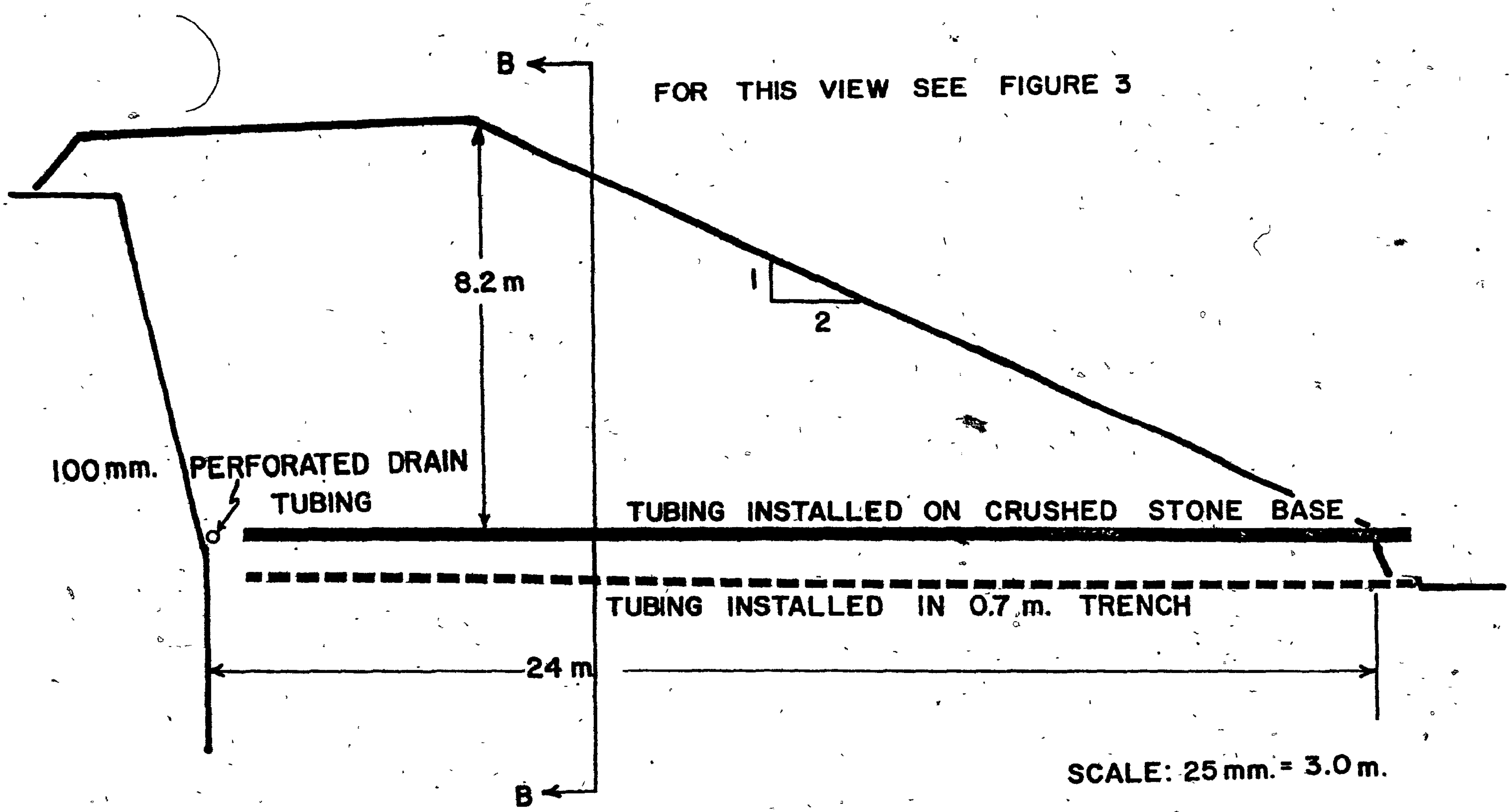


Figure 2. Profile of hill with tubing installation.

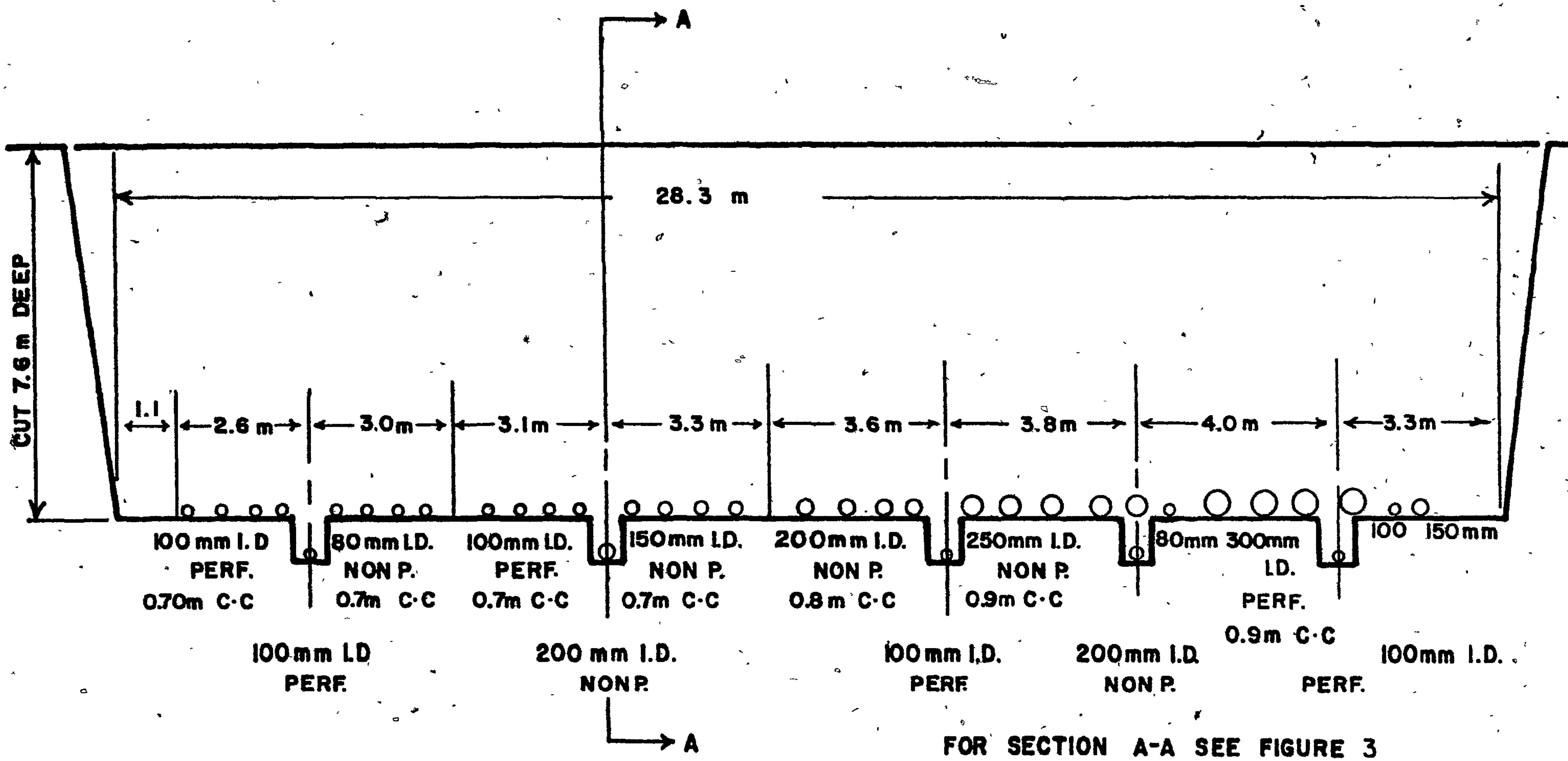


Figure 3. Front view of tubing installation.

wrapped P.E. and two 200 mm I.D. non-perforated P.E. lines. Original plans called for four lines of each type, but the surface was too soft for the trencher to continue. These five lines were placed on the clay trench bottom and were backfilled with crushed stone ranging in size from zero to 13 mm diameter. After completion of this backfilling, an attempt was made to smooth the surface using a D7 bulldozer, but the soil proved too soft to achieve an adequate result. The surface was then left for two days during which temperatures down to -20°C prevailed, and enough freezing took place to allow machinery to move and work effectively for the installation of the upper layer of tubing.

A 100 mm perforated filter-covered drain tube was placed along the back wall of the excavated area to aid in drainage. This line was connected to the first line installed at this level which was also a 100 mm polyethylene perforated tube enveloped with a filter sock. The tubing was installed as paired lines as shown in Figure 4, using the following procedure steps:

1. A 150 mm depth of crushed stone was laid down, compacted and smoothed with the Caterpillar 955 crawler loader and a MF 165 wheel tractor.
2. Ninety degree grooves were made in the crushed stone with the tractor attachment and hand work.
3. The tubing was laid in the grooves.
4. The tubing was covered by hand with 150 mm of crushed stone, smoothed and tamped with a motorized tamper.
5. A 150 mm layer of sand was placed on top of the crushed stone, then smoothed and tamped.

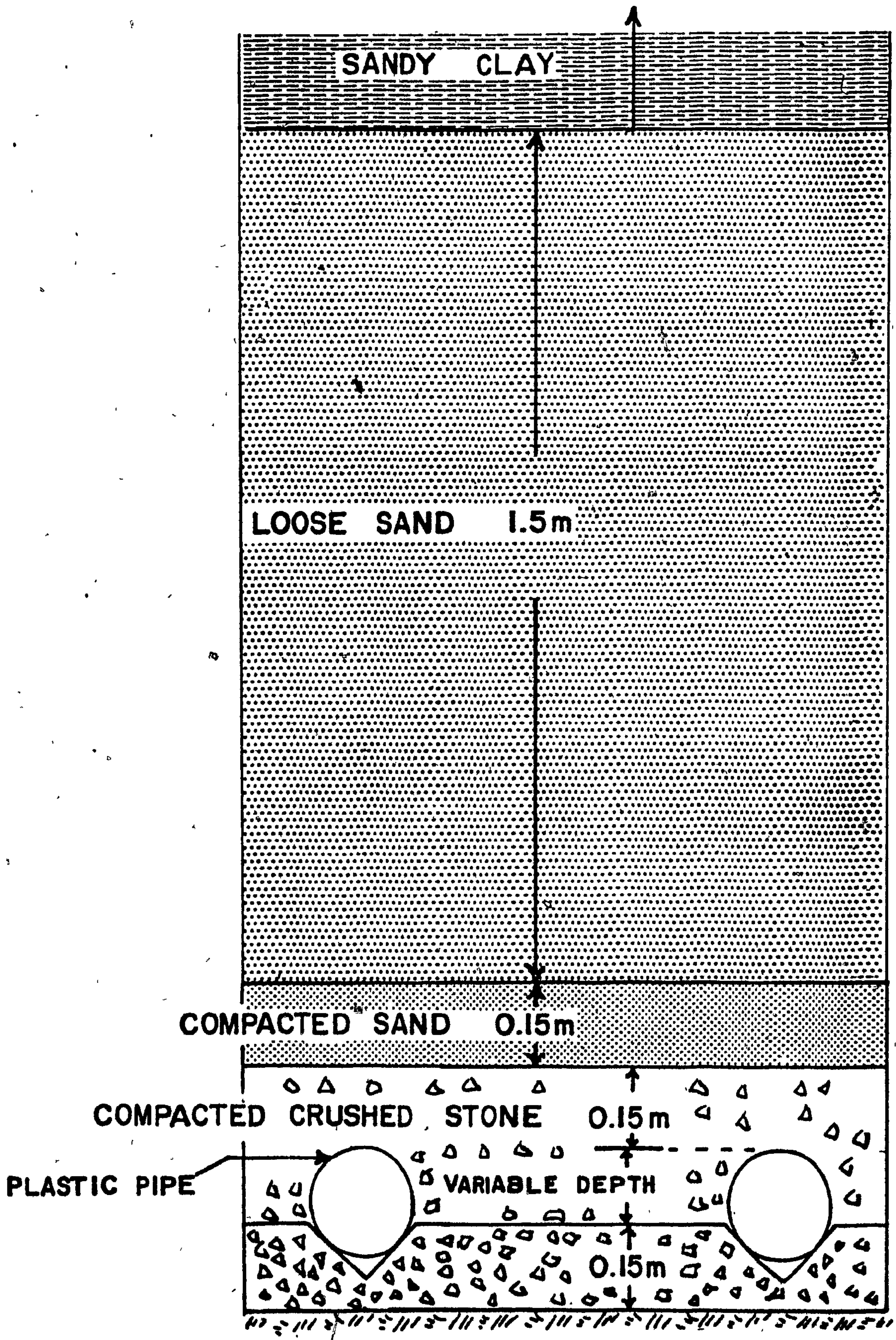


Figure 4. Bedding and backfill used

6. One and one half metre of sand was placed as loosefill with the 955 crawler loader.
7. Steps 1 to 6 were repeated with two rows of tubing at a time.
8. After all the tubing was installed in this manner, backfilling was done with a D7 bulldozer by pushing sand ahead so that at least three metres of sand covered the tubing before it was subjected to the weight of a bulldozer.
9. The rest of the hillside loading area was filled and graded with a clay and sand mixture.

When installation of the 250 mm tubing was finished, the plug test was performed for the 80 mm tubing and it was found that two lines had been damaged during covering. To provide three replicates, one more 80 mm line was installed. For similar reasons, there were also installed extra lines of 100 mm PPR and 150 mm and 250 mm PE tubing (see Figure 3).

About 0.6 metre of soil was added to the top of the loading area in the summer of 1977 to smooth out irregularities in the fill and to bring the hillside to its final grade. The bank was then seeded to grass to reduce erosion. A low retaining wall was built in the summer of 1977 at the outlet end of the corrugated tubes so that the starting point for length measurements could be unified and the base of the slope could be kept stable with about 0.6 metre of soil over the top of the tubes.

2. Field measurements

The vertical dimension inside the tubes was determined by inserting spherical wooden plug gauges attached to a nominal half-inch (13 mm) diameter galvanized steel pipe, on which length measurements were marked. The plug gauges were made in diameter increments, as shown in Table 2. Measurements began with the largest diameter plug which would enter each pipe. The distance at which that plug stopped was noted. Then the next smaller plug was inserted until it encountered a stopping resistance, then the next smaller plug was inserted, etc. Results of each set of measurements are shown in Figures 8 to 16. It must be realized, when looking at these figures, that the numbers given as the maximum deflection are in step increments because the plug gauges have step increases in diameter. The actual deflections along any tube will be between the amounts indicated by the last two percentage numbers; that is, between the deflection indicated by the plug which goes through and the plug which does not go through. It was noted that the plug gauging gives rather coarse measurements. Also, it would be possible for the plug to be stopped by a dent in the tubing at one location and for the tubing to have a larger vertical dimension beyond the dent. But it was not possible to rent a suitable electrical diameter measuring device which might give better precision, and it was beyond our financial resources to build such an instrument.

TABLE 2. Per cent of actual inside diameter deflection indicated by the stopping of the plug

Plug gauge diameter		Tubing	Deflection % of tubing inside diameter					
		Nominal I.D. (in)	3	4	6	8	10	12
Inches	mm	Actual I.D. (in)	3.15	4.00	6.05	8.15	10.07	11.80
		Actual I.D. (mm)	80.1	101.6	153.7	207.1	255.7	299.7
1.38	35.1		56.2	65.5				
2.05	52.1		35.0	48.7				
2.26	57.4		28.3	43.5				
2.49	63.2		20.1	37.7				
2.80	71.1		11.2	30.0				
2.88	73.2		8.6	28.0				
3.00	76.2			25.0				
3.26	82.9			18.5				
3.50	88.0			12.5				
3.61	91.8			9.6	40.3			
3.98	101.1				34.2			
4.54	115.4				24.9	44.3		
5.00	127.0				17.4	38.7		
5.19	131.8				14.2	36.3		
5.49	139.4				9.3	32.7	45.5	
6.06	154.0					25.6	39.8	
6.48	164.7					20.5	35.6	45.0
7.02	178.3					13.9	30.3	40.5
7.25	184.2					11.0	28.0	38.6
7.52	191.1					7.7	25.3	36.2
7.75	196.8						23.0	34.3
8.13	206.6						19.2	31.1
8.61	218.6						14.5	27.1
8.75	222.3						13.1	25.8
9.09	230.8						9.7	23.0
9.42	239.2						6.5	20.2
9.75	247.6						3.2	17.4
10.04	254.9							14.9
10.48	266.3							11.1
11.01	279.7							6.7
11.44	290.7							3.0

3. Laboratory measurements

The particle size distributions for the crushed stone used as bedding and the sand used as backfill are given in Figure 5.

Pipe stiffness was measured in the laboratory with a parallel flat plate loading device as indicated in Figure 6. A summary of the results is given in Table 1. The procedure for the stiffness test for corrugated plastic drain tubes is given in the Canadian Government Specification Board Standard 41-GP-29a and by ASTM standard F-405.

A preload is used to straighten the pipe and obtain contact between the plates and the pipe along the full test length. The load is then applied such that the deflection proceeds at a constant rate, usually 12 mm per minute, and the load F, and deflection ΔY, are measured simultaneously, or obtained directly from a graph provided by the testing machine.

A load-deflection graph for a 200 mm inside diameter corrugated polyethylene drain tube is shown in Figure 7. The shape of this graph is typical of the corrugated polyethylene tubes of diameters ranging from 100 mm to 300 mm which were measured.

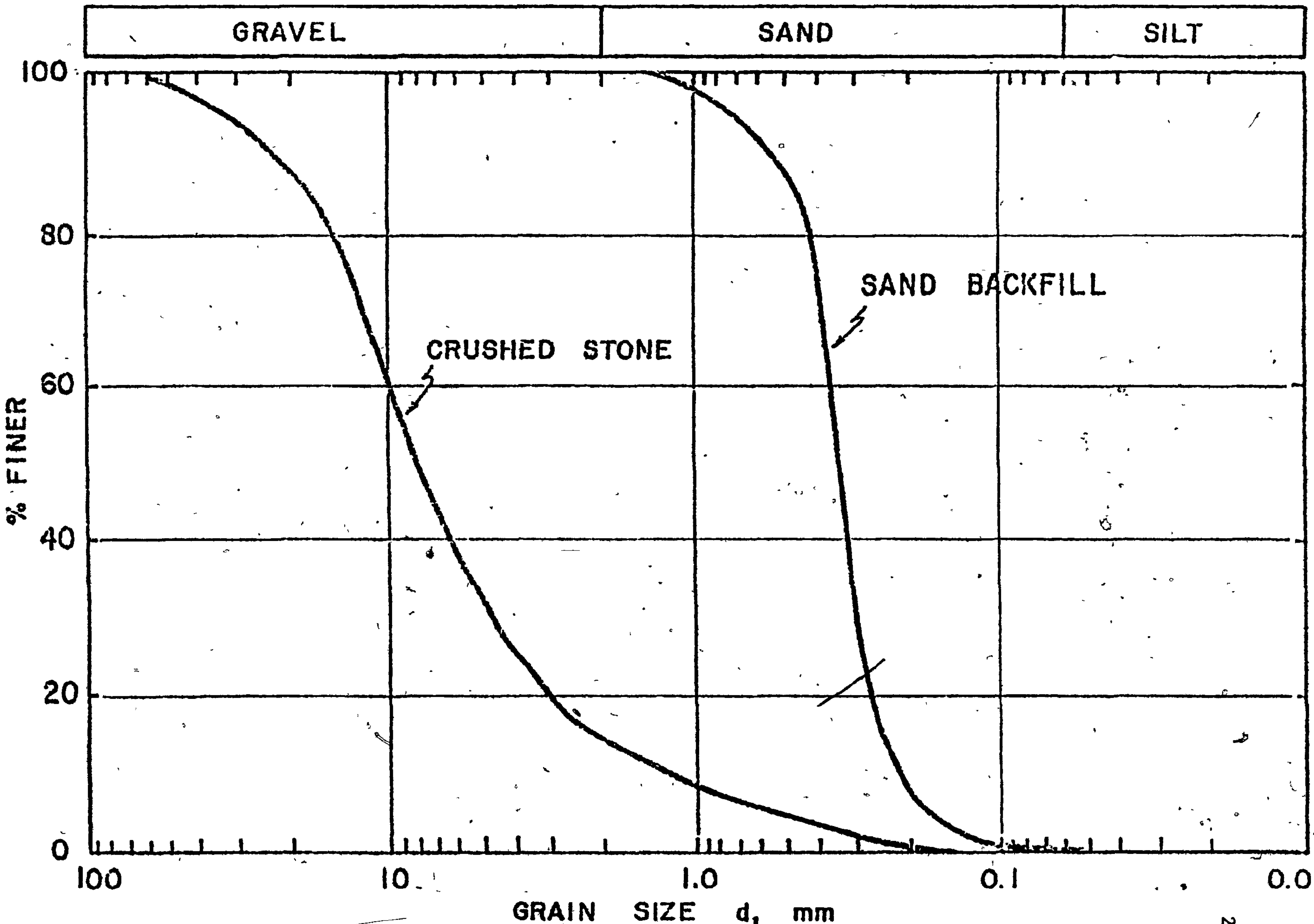
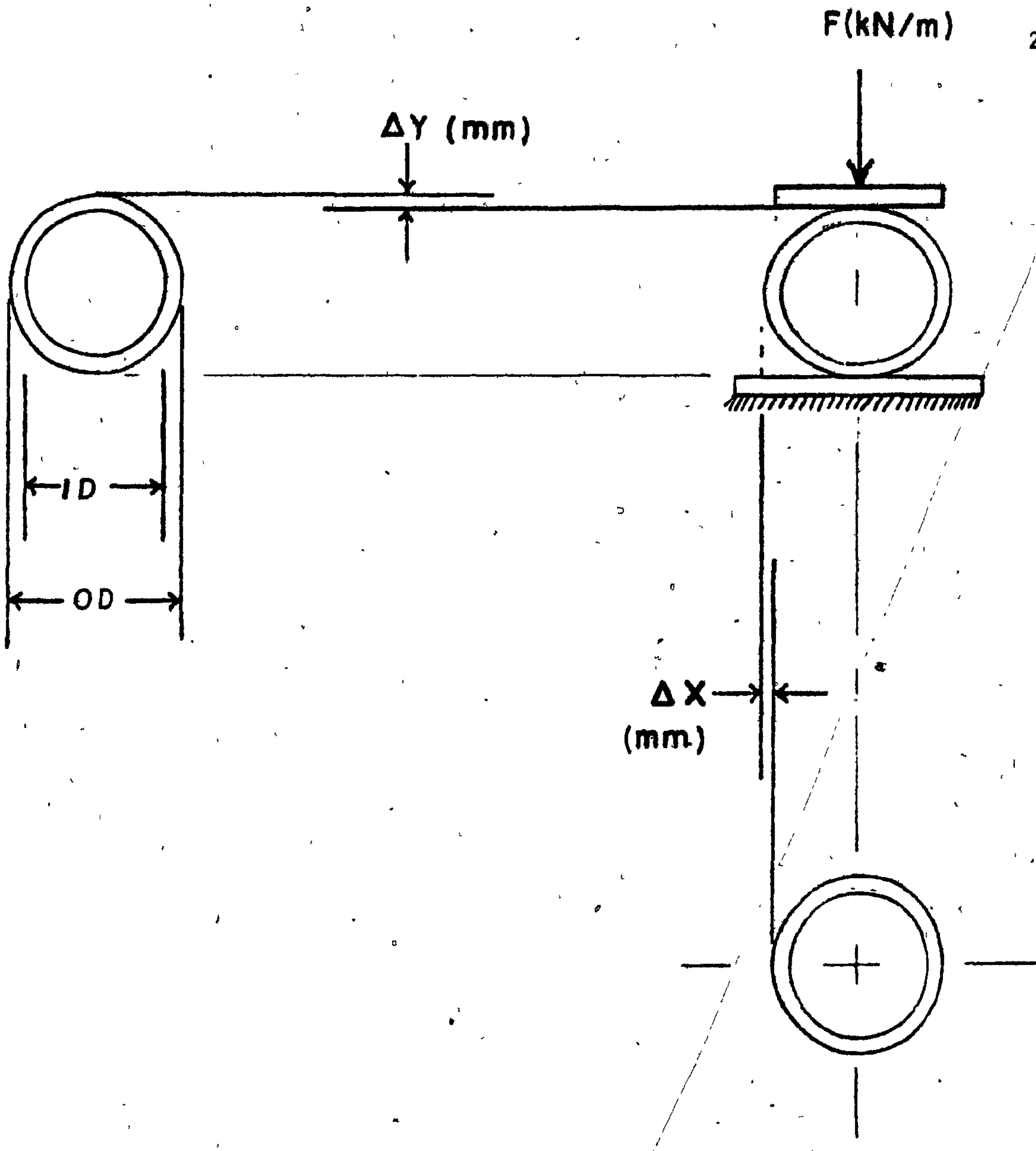


Figure 5. Grain Size Curves of Crushed Stone Bedding and Sand Backfill.



$$\text{PIPE STIFFNESS} = \frac{F}{\Delta Y} \quad \frac{kN/m}{m} \quad \text{OR kPa}$$

Figure 6: Definition sketch for laboratory measurement of pipe stiffness

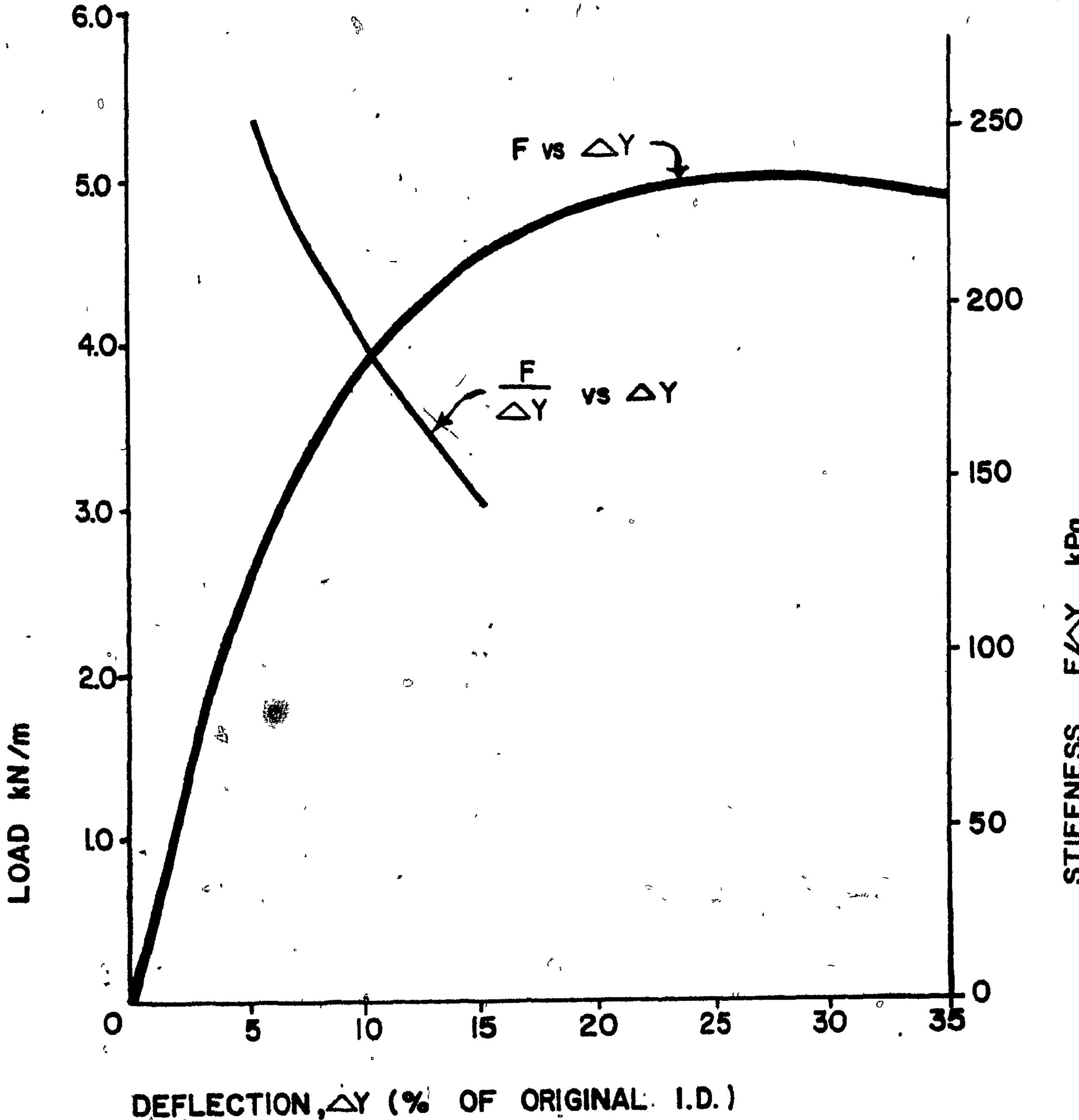


Figure 7. Load F, and stiffness F/ΔY versus deflection ΔY for a typical 200 mm I.D. corrugated polyethylene drain tube.

IV. RESULTS AND DISCUSSION

No deflection large enough to stop the largest suitable plug gauge was noted on 13 and 15 December 1976 during the filling process. However, changes were noted in the larger tubes on 22 December 1976, four days after earth-work was completed (see Figures 8 and 9). The actual diameter measurements are given in Figure 8 and are presented as a deflection percentage of the nominal inside diameter in Figure 9.

The data for the deflections measured in 1977, 1978 and 1980 are given as Figures 10 through 16. From these figures it can be seen that most of the tubes have undergone approximately five per cent additional deflection between 22 December 1976 and 29 April 1977. Some tubes had very small additional deflection between 29 April 1977 and 16 September 1977. There has been practically no change from 16 September 1977 to 3 November 1980. This shows that more than 90 per cent of the total deflection measured after four years occurred during the first year after installation.

Fenemor et al. (1978) defines the lag factor D_e as:

$$D_e = \frac{\text{maximum deflection after 5 years}}{\text{initial maximum deflection}} \qquad \ldots (12)$$

Data from Figures 9 and 15 were used in equation 12 to calculate lag factor for the 150, 200, 250, 300 mm I.D. plastic tubes. The mean deflection at specific depths for each tubing group of the same size was used in the calculations. Results are presented in Table 3. The D_e calculated ranged from 1.0 to 1.8, with an average of 1.45.

Since there has been practically no change in deflection from 16 September 1977 to 3 November 1980, it is believed that the D_e calculated is of the same magnitude as if observations after 5 years had been used in the calculation.

Deflections as large as 28.2 per cent were observed at a fill depth of 3.6 m for the 80 mm tubes and up to 56.2 per cent at a depth of 5 metres. The 100 mm I.D. tubes showed deflections of 18.4 per cent for a fill of 1.5 m and more than 28 per cent for greater depths of fill. The 100 mm tubes placed in trenches suffered less deformation and showed a deflection of 18.4 per cent for a fill up to 7.5 m, while the 200 mm tubes placed in trenches have deflected more than 20 per cent under a fill of 7.5 m.

The mean deflections for sizes 150, 200, 250 and 300 mm are given in Tables 4, 5, 6, and 7. They were plotted against depth of fill and are shown in Figures 17 through 20. Theoretical deflection curves were plotted on the different graphs using values obtained from Spangler's equation, in order to see how predicted deflections compared with measured mean deflections.

A lag factor of 1.5 was used in the equation since this value seemed to apply to the experiment, and a K value of 0.096 was taken as the bedding constant for 90° grooves. Based on judgement and on

Howard's (1977) soil description and degree of compaction of bedding for recommended values of E', a value of 1379 kPA (200 psi) was adopted to calculate deflection for all sizes of tubes and depths of fill. A specific density of 1.76 g/cm³ (110 lb/ft³) was assumed to be representative of the fill material.

Since the actual construction conditions did not fit exactly one of the construction condition classifications shown in Figure 1, theoretical deflections were calculated assuming different installation conditions. The projecting conduit condition was retained as a comparison with measured deflections. The imperfect trench condition produced similar results, although slightly below those of the projecting conduit condition. Equations 2 and 6 were used to determine the load on the different tubes.

Two situations were studied: one considered a surcharge, and the other consisted in extrapolating a value of the loading coefficient C_c. In the first situation, for values of H/B_c greater than 10, a surcharge was considered. For instance, taking the case of a 200 mm I.D. tube having 250 mm O.D., under 7.5 m of fill:

$$\frac{H}{B_c} = \frac{7.5}{0.25} = 30$$

Since the W.P.C.F. Manual no.9 does not provide values for such a high ratio, a height of surcharge was determined in the following way; using the 200 mm drain tube as an example under 7.5 m of fill.

Setting $\frac{H}{B_c} = 10$

Then H = 10 x 0.25 = 2.5 metres

and the surcharge is: 7.5 - 2.5 = 5.0 metres

The loading coefficient C_s for the surcharge was taken from Table XXVI, page 206 of W.P.C.F. Manual no.9 (see Appendix A).

This method of evaluating the load on the tubes yielded theoretical deflections that follow the general tendency of the actual measured deflections versus depth, curves. For depth of fill of 1.5 m the standard procedure was used to find C_c and no surcharge was considered.

Looking at Figures 17, 18 and 19, which show field deflection curves for the 150, 200 and 250 mm tubes, it can be seen that deflection increases rapidly until a depth of fill of 1.5 m is reached and then all three curves show a change in slope. Looking at the theoretical curves, one sees also, a break in increase at a depth of 1.5 m, although the break is not as drastic as in the case of the actual deflection curves.

If a value of C_c is extrapolated from the curves in Figure 49 of the WPCF Manual no.9, which is possible since an r_{sd} value equal to zero has been used and there is a direct relationship between H/B_c and C_c; then a curve pattern as shown in Figures 17 through 20 is obtained, which is different from the measured deflection curves.

Loads calculated by the projecting conduit condition with a surcharge, yielded a difference between measured and predicted deflections within 2.0 per cent of the tubing I.D. at all depths for the 250 mm tubes. At 4.5 and 7.5 metres deep for the 200 mm tubes, predicted deflections are within 3.0 per cent of I.D. from the measured deflections at all depths. For the 150 mm tubes, although the predicted deflections fall below the measured ones, the two curves have the same tendency. In the case of the 300 mm tubes, predicted deflections are within 2.0 per cent of the I.D. from what was measured at 1.5 and 4.5 metres deep, but as the depth of fill increases to 7.5 metres, the predicted deflection curve stays almost constant, whereas the measured deflection curve keeps increasing steadily. The deflection curve of the 300 mm tubes fits the predicted deflections calculated without considering a surcharge, within 1.5 percent of the tubing I.D.

. Spangler's equation for deflection was applied to calculate the modulus of soil reaction E', using the measured deflection values. For calculation purposes the formula can be written:

$$E' = 16.393 \left[\frac{D_e K W_c}{0.913\ \Delta Y} - 0.149\ F/\Delta Y \right] \text{ no surcharge} \quad \ldots (13)$$

$$E' = 16.393 \left[\frac{(D_e K W_c + K W_{sd})}{0.913\ \Delta Y} - 0.149\ F/\Delta Y \right] \quad \ldots (14)$$

surcharge included

The modulus of soil reaction was calculated using loads determined by the positive projecting conduit conditions plus a surcharge load. Table 8 summarizes the values for the fill at 1.5 m.

Howard (1977), in his experiments, observed variations of E' within each category of bedding he tried. But he reports that predicted deflection using a mean value of E', was between 0.5 and 2.0

per cent of the measured deflection. The various types of pipe he surveyed varied in size from 12 to 180 inches (300 to 4570 mm) in diameter.

From Table 8 it can be seen that E' values decrease as tubing size decreases. Although the modulus of soil reaction does not have any relationship to the diameter of a buried pipe, there seems to be some connection. It may be that the fill material adjacent to the tube, i.e., 20 mm or so, does not get as well compacted as the material farther away from it, and so the tube does not receive much support from the first two centimetres on each side of the pipe. Although this 20 mm does not seem very much, it should be realized that in the case of a 150 mm tube, a 20 mm deflection represents 13 per cent deflection, whereas in the case of a 1220 mm pipe, for instance, it would only be a 1.6 per cent deflection. Therefore, one can see that compaction is more critical for small size pipes than for larger size pipes. Referring to the results of deflection at a depth of 1.5 m, the 300 mm tubes show a 3 per cent deflection, whereas the others have larger and increasing percentage deflection as the diameter decreases.

An interesting observation is that the 150 mm tubes, when tested between flat plates, sustained a load of 2.9 kN/m. The calculated load for 1.5 m of fill is 4.6 kN/m. Therefore, if the pipe can support 2.9 kN/m of its own, then the soil's share is 1.7 kN/m. Since the predicted deflection was 4.5 per cent for 1.5 m of fill and the measured deflection was 15 per cent, then there is a 10.5 per cent deflection which is not taken into account in the equations used to calculate loads and deflections. At 10 per cent deflection, the tubes took a load of 2.3 kN/m when tested between flat plates. This value is reasonably close to the 2.9 kN/m load which the pipe was expected to carry in the field, considering all the uncertainties involved in the calculations. Therefore, it seems probable that the tubes had to deflect 10.5 per cent before getting some support from the adjoining soil. At 4.5 and 7.5 metre depths, deflection did not increase, indicative that any additional load was absorbed by the soil.

Similarly, the flat plate tests for the 200 mm tubes showed that they can take 3.4 kN/m at 8 per cent deflection. The calculated load was 6.6 kN/m at 1.5 m deep; therefore, a load of 3.2 kN/m was carried by the soil. Since the predicted deflection is 4.4 per cent, the pipe would have deflected 3.6 per cent or approximately 7 mm in horizontal deflection before the soil reaction took place.

Another observation is the load increase at each depth for each size of tube. With ~~depth of~~ fill increasing, deflections of the 150, 200 and 250 mm tubes increased but at a much slower rate than that of the 300 mm tubes. The latter show a nearly constant increase in deflection with increasing depth. Table 9 gives the calculated load for each size of tube at three different depths. The values shown are the sum of a dead load and a surcharge. At 1.5 m, the load on each size of the tube is incremented by approximately 2.0 kN/m, ranging from the 150 to the 300 mm tubes. For depths of 4.5 and 7.5 m, the load increment between the 150, 200 and 250 mm tubes is of the order of 5 kN/M. Between the 250 and 300 mm tubes, the increment is nearly double that of the others.

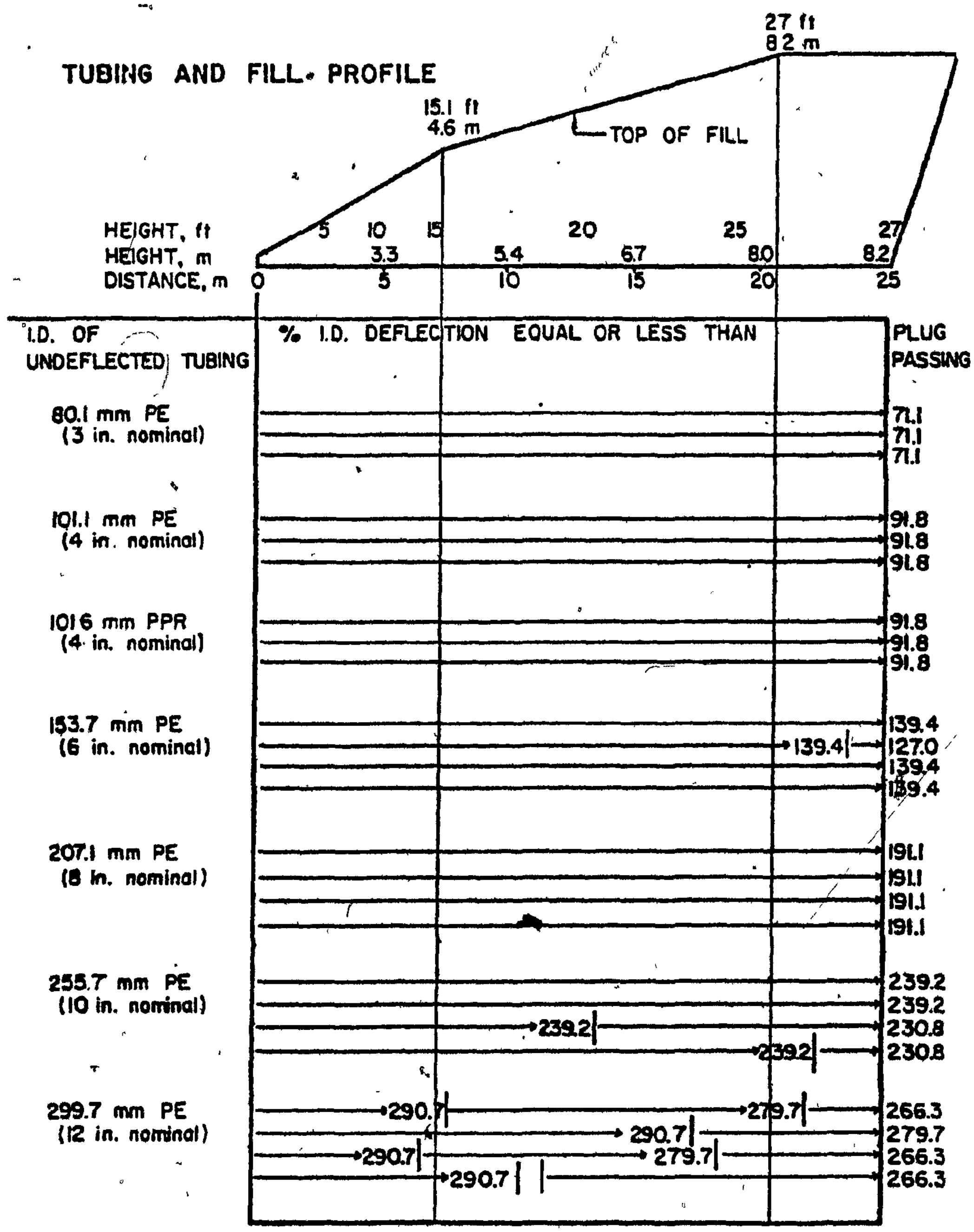


Figure 8. Diameter in millimeters of the largest plug gauge passing through tubes on 22 December 1976. Vertical bar indicates the place where plug stopped. The final column indicates the diameter of the plug which went all the way.

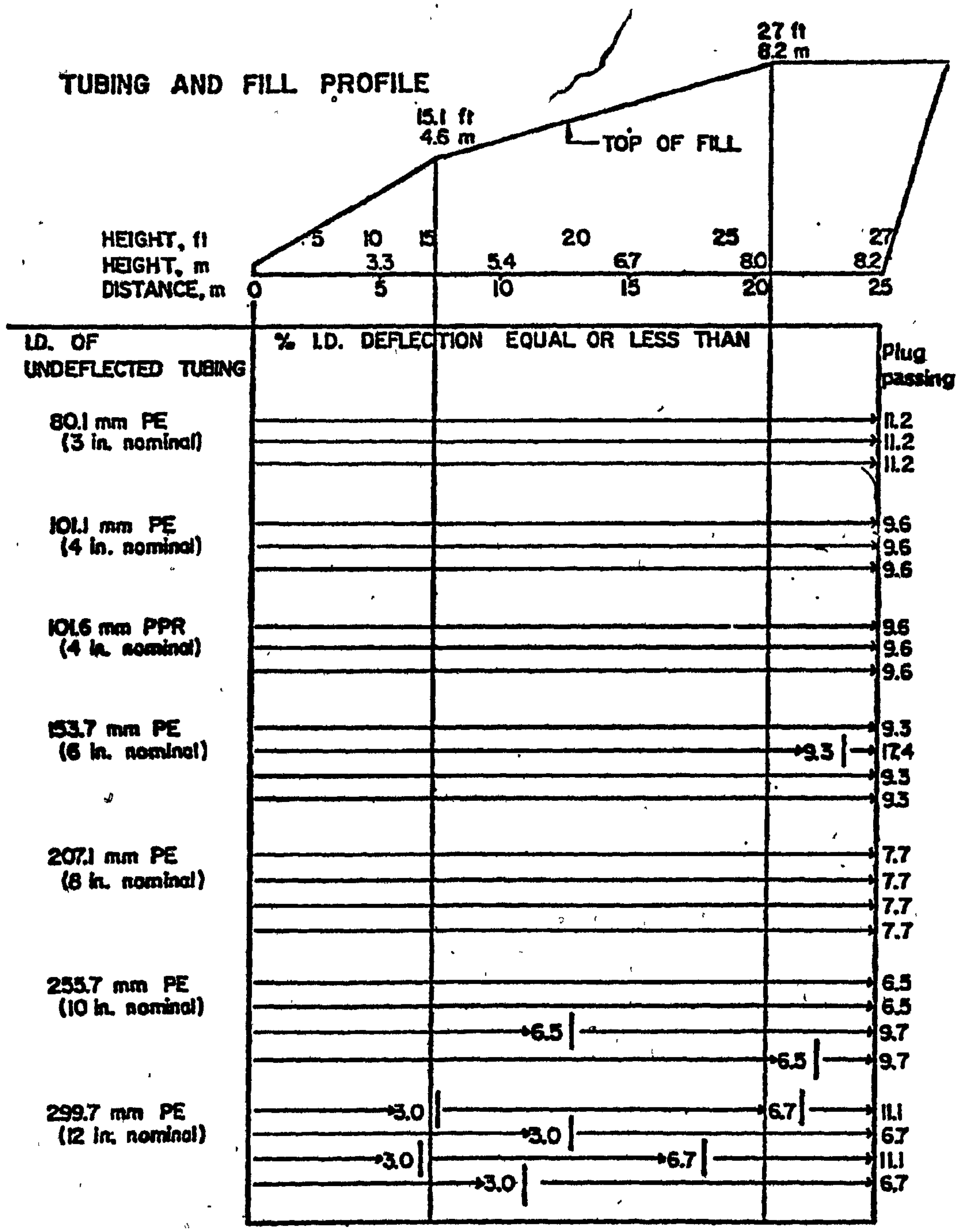


Figure 9. On 22 December 1976, one week after installation, tubing deflections were equal or less than the percent of the initial I.D. shown. The vertical bar indicates the place where the plug gauge stopped. The final column indicates the plug which went all the way.

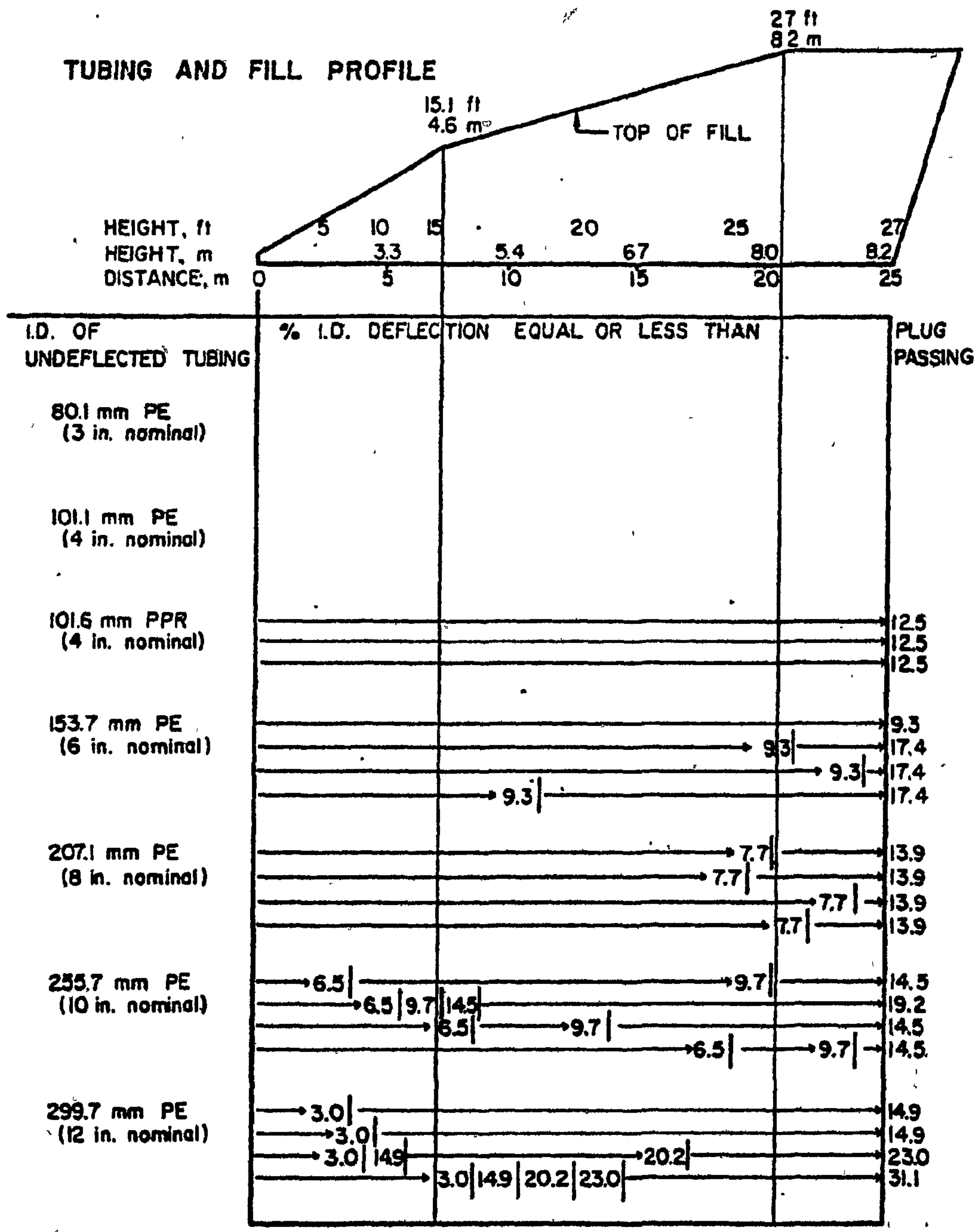


Figure 10. On 29 April, 1977, 4.5 months after installation, tubing deflections were equal or less than the percent of the initial I.D. shown. The vertical bar indicates the place where the plug gauge stopped. The final column indicates the plug which went all the way.

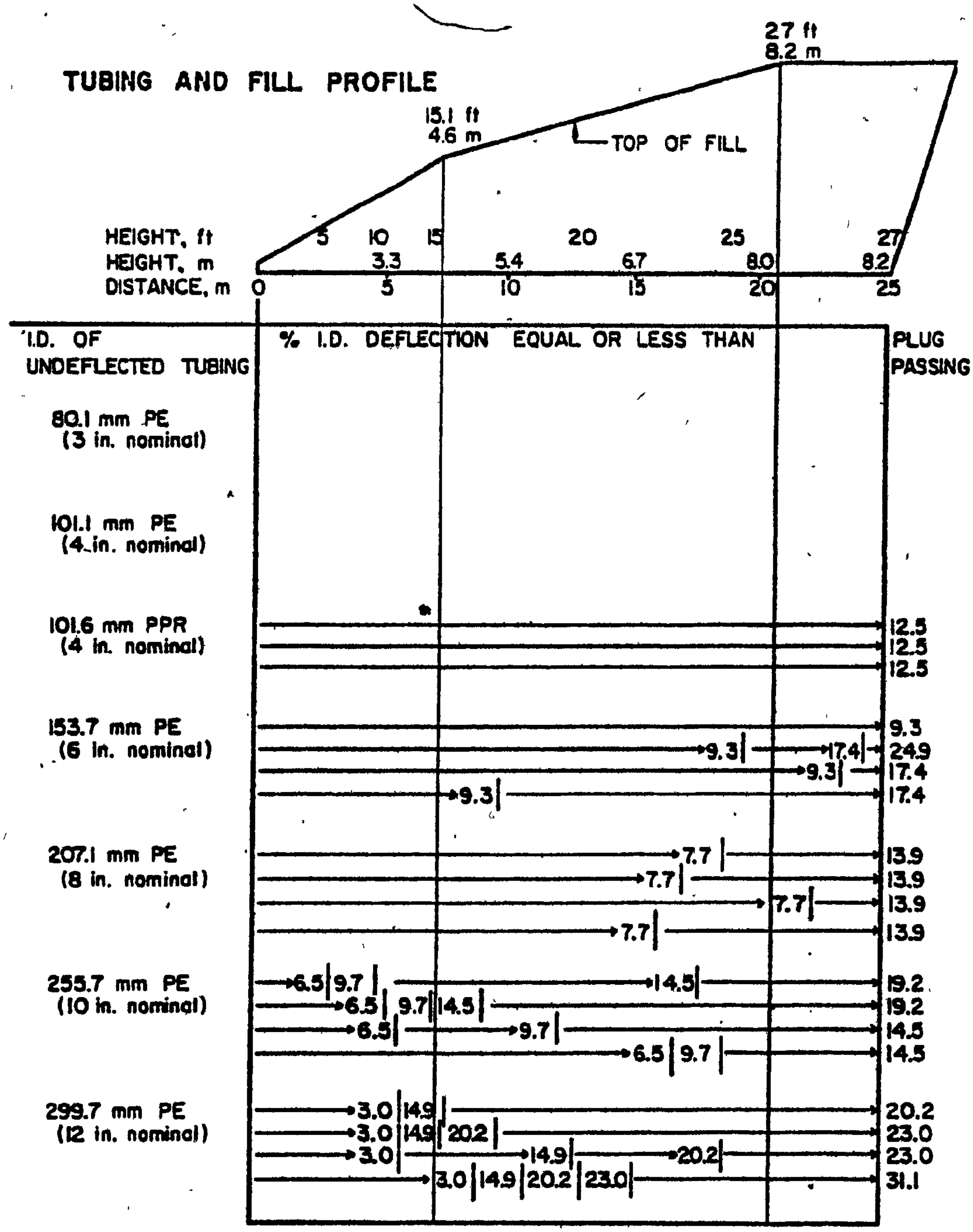


Figure 11. On 16 September 1977, 9 months after installation, tubing deflections were equal or less than the percent of the initial I.D. shown. The vertical bar indicates the place where the plug gauge stopped. The final column indicates the plug which went all the way.

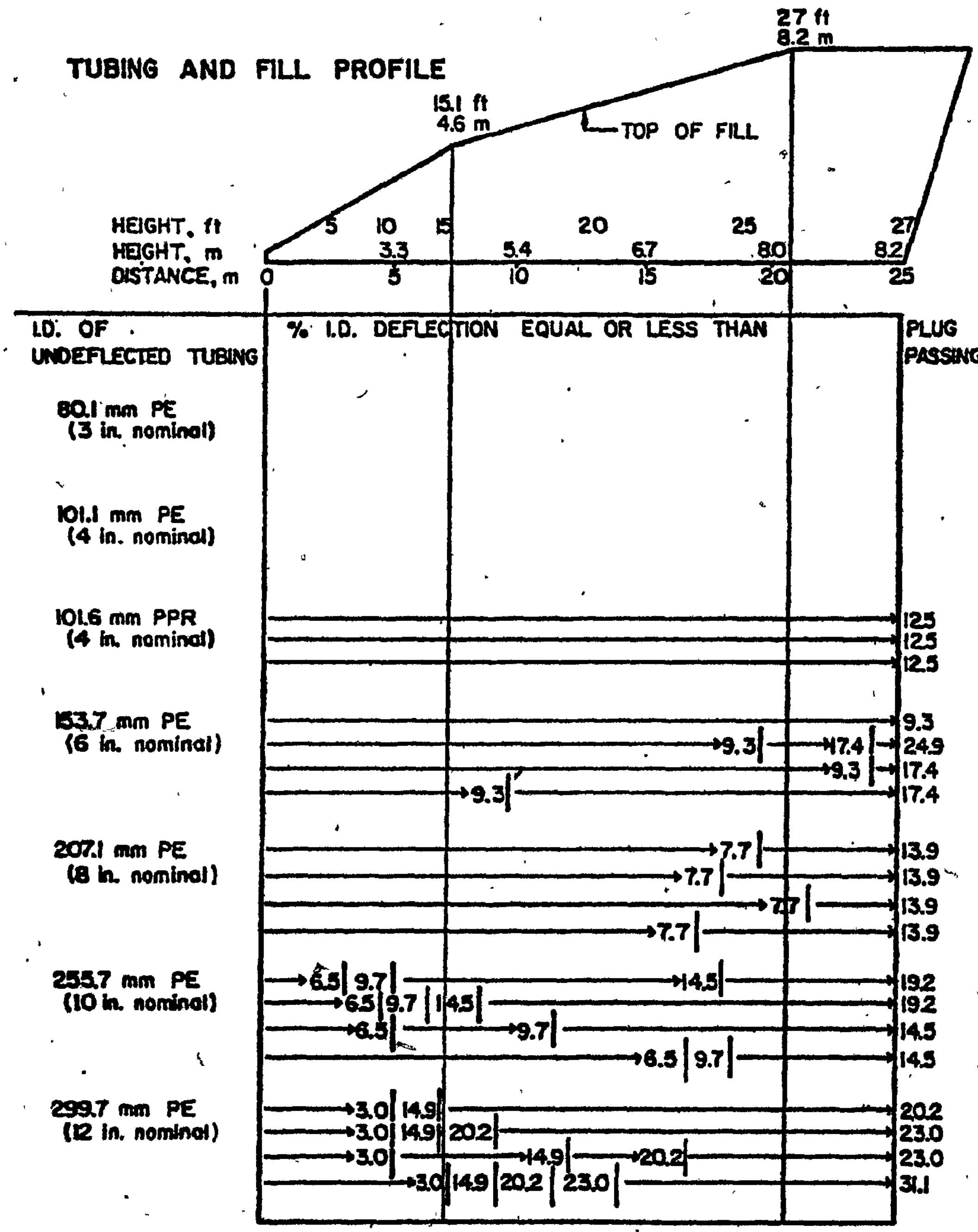


Figure 12. On 30 November 1977, 11.5 months after installation, tubing deflections were equal or less than the percent of the initial I.D. shown. The vertical bar indicates the place where the plug gauge stopped. The final column indicates the plug which went all the way.

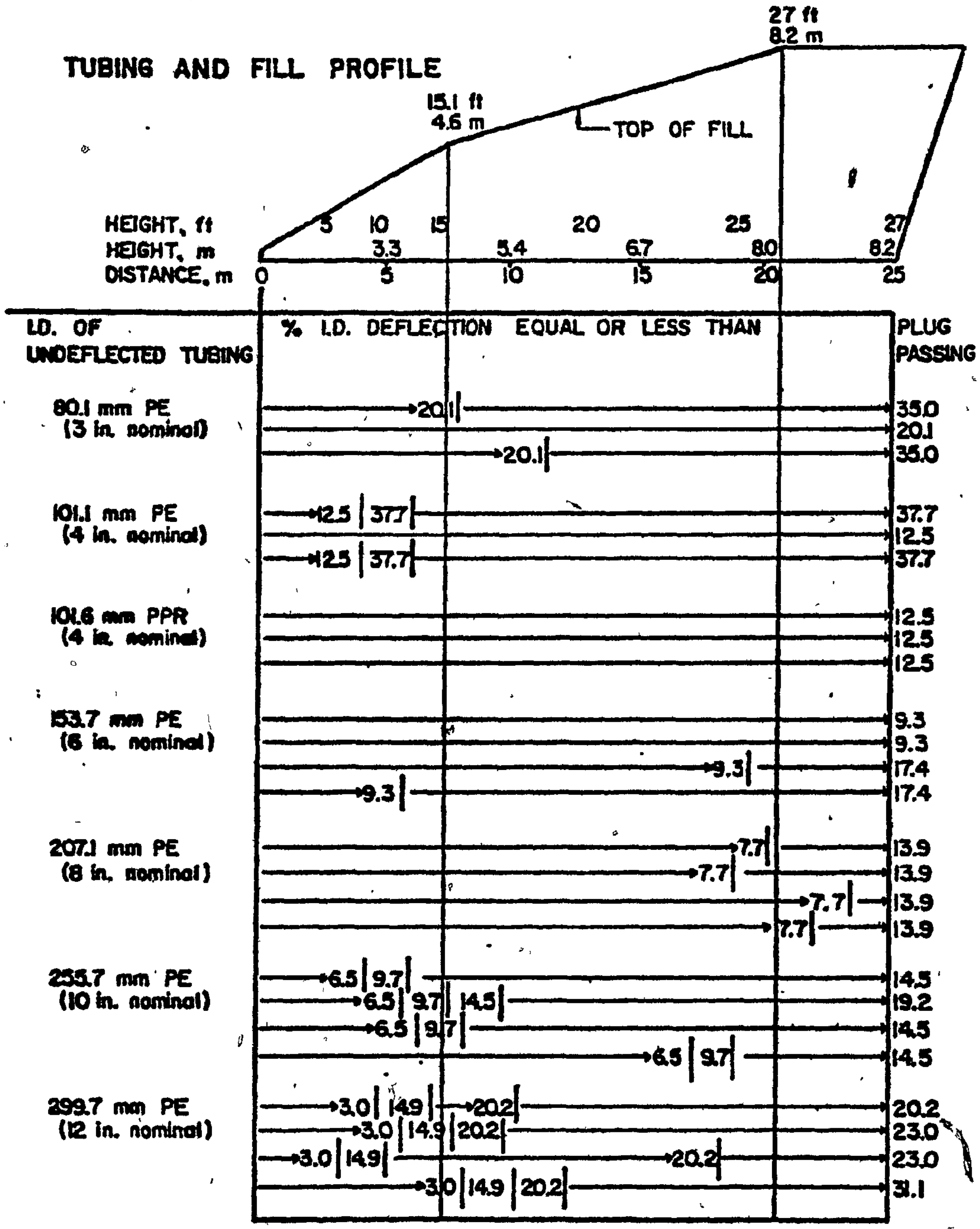


Figure 13. On 10 October 1978, 22 months after installation, tubing deflections were equal or less than the percent of the initial I.D. shown. The vertical bar indicates the place where the plug gauge stopped. The final column indicates the plug which went all the way.

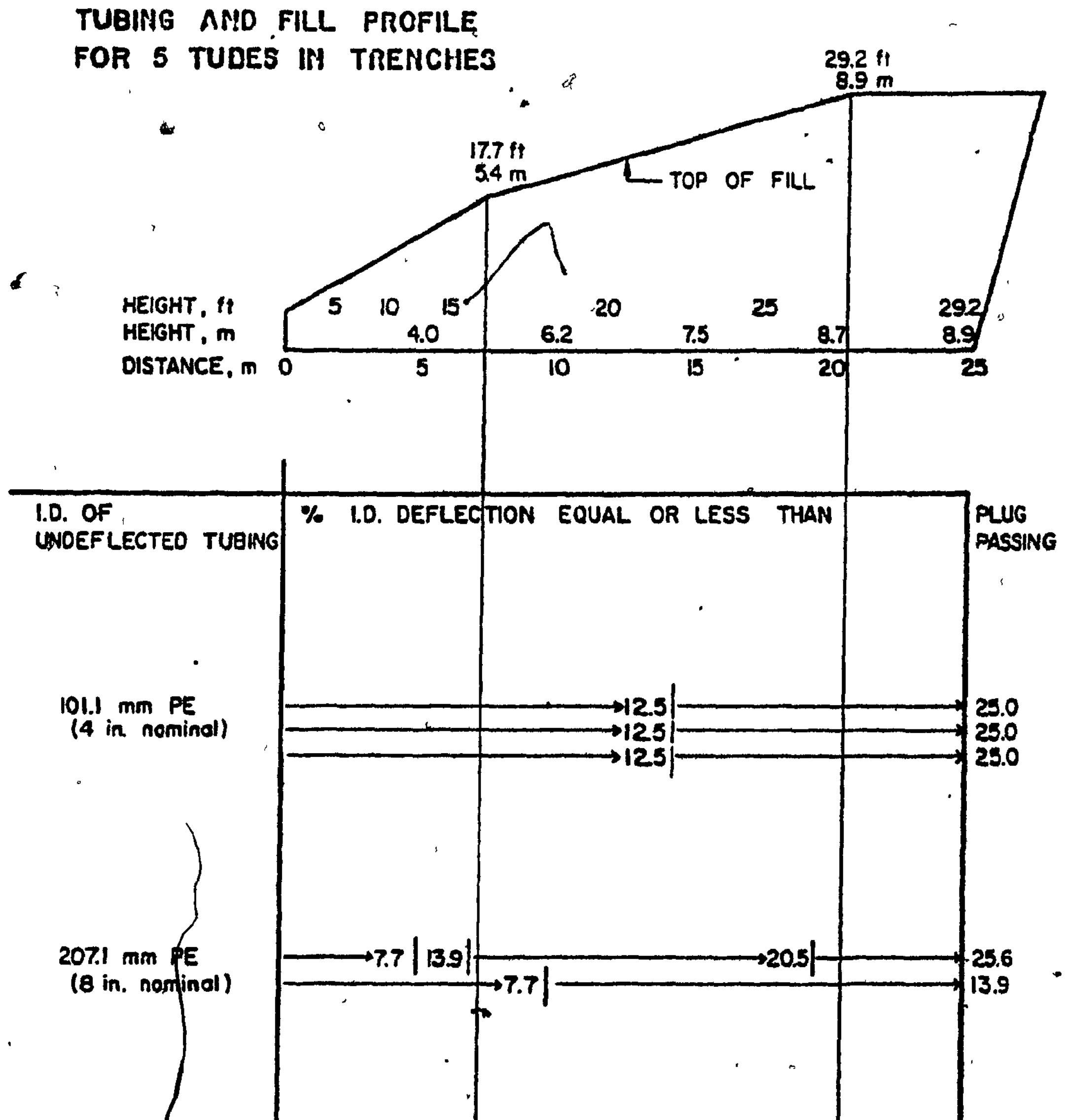


Figure 14. On 10 October 1978, 22 months after installation, the deflections of the five tubes placed in trenches were equal or less than the percent of the initial I.D. shown. The vertical bar indicates the place where the plug gauge stopped. The final column indicates the plug which went all the way.

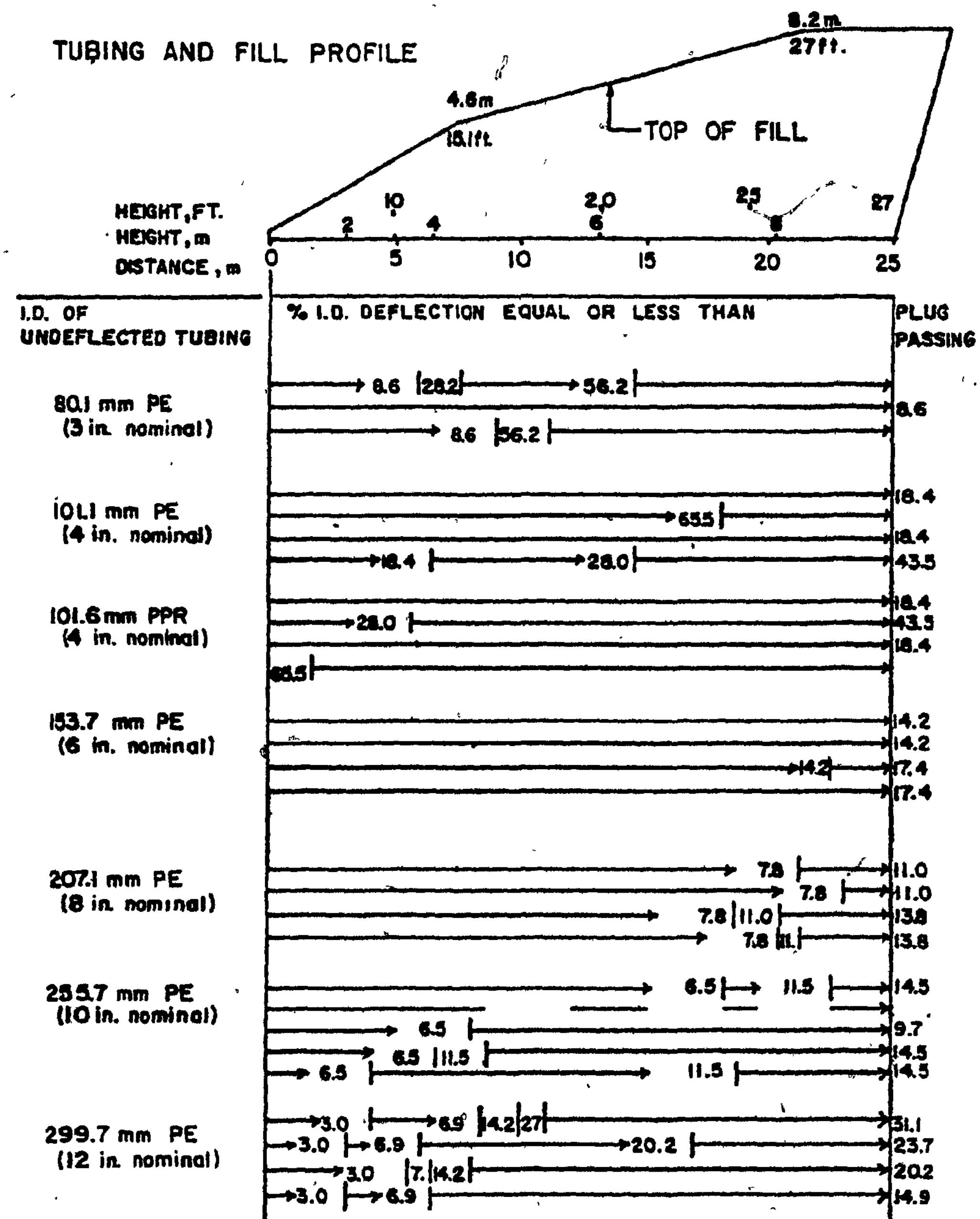


Figure 15. On November 3, 1980, 47 months after installation, tubing deflections were equal or less than the percent of initial I.D. shown. The vertical bar indicates the place where the plug gauge stopped. The final column indicates the plug which went all the way.

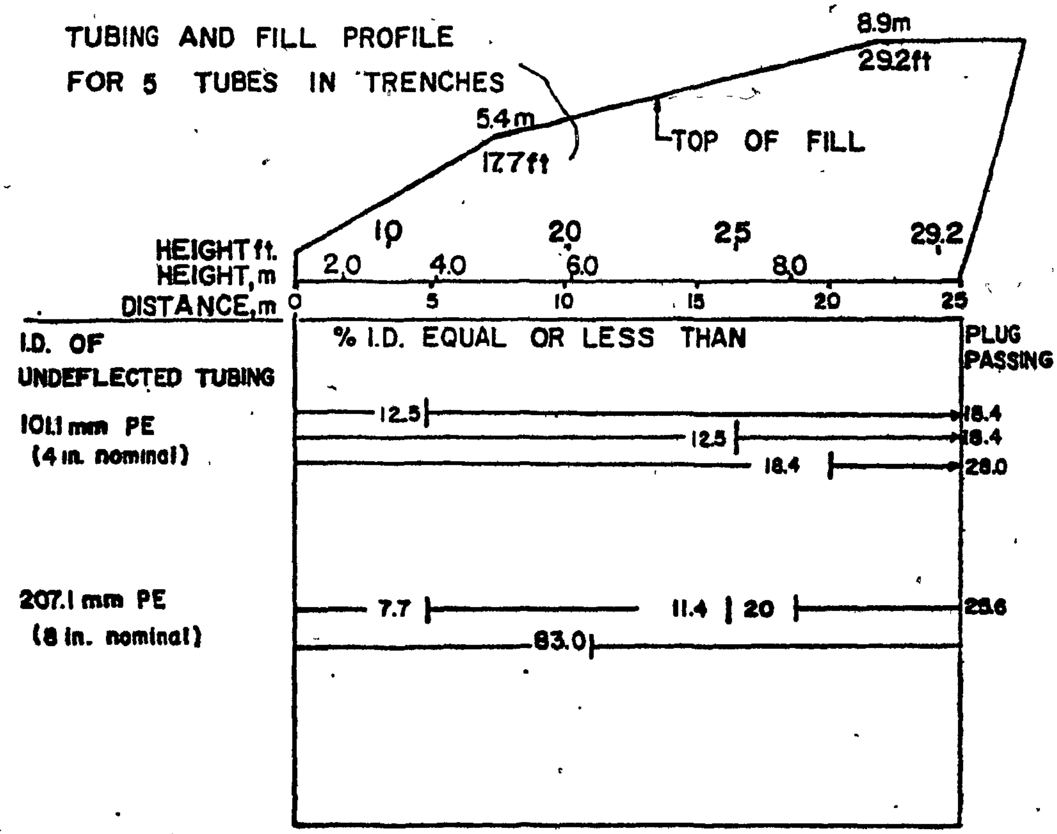


Figure 16. On November 3, 1980, 47 months after installation, the deflections of five tubes placed in trenches were equal or less than the percent of initial I.D. shown. The vertical bar indicates the place where the plug gauge stopped. The final column indicates the plug which went all the way.

TABLE 3. Calculated lag factor D_e

Size (mm)	% Deflection at t* = 0	% Deflection at t = 4 years	Depth of fill (m)	D_e
150	9.3	15.0	1.5	1.6
150	9.3	15.0	4.5	1.0
150	11.3	15.0	7.5	1.4
200	7.7	7.8	1.5	1.0
200	7.7	8.5	4.5	1.6
200	7.7	10.9	7.5	1.1
250	6.5	6.5	1.5	1.0
250	6.5	7.7	4.5	1.2
250	8.1	11.8	7.5	1.5
300	3.0	3.0	1.5	1.4
300	3.9	14.0	4.5	1.8
300	8.9	22.4	7.5	2.5

* t is time at which deflection was measured

TABLE 4. Observed per cent I.D. deflection of 150 mm diameter plastic tubing

Depth (metres)	1.5	4.5	7.5
150 mm I.D.	14.2	14.2	14.2
	14.2	14.2	14.2
	14.2	14.2	14.2
	17.4	17.4	17.4
Mean deflection	15.0	15.0	15.0
Maximum deflection	17.4	17.4	17.4

TABLE 5. Observed per cent I.D. deflection of 200 mm diameter plastic tubing

Depth (metres)	1.5	4.5	7.5
200 mm I.D.	7.8	7.8	7.8
	7.8	7.8	7.8
	7.8	7.8	7.8
	7.8	7.8	11.0
Mean deflection	7.8	7.8	8.6
Maximum deflection	7.8	7.8	11.0

TABLE 6. Observed per cent I.D. deflection of 250 mm diameter plastic tubing

Depth (metres)	1.5	4.5	7.5
250 mm I.D.	6.5	6.5	11.5
	6.5	11.5	14.5
	6.5	6.5	9.7
	6.5	6.5	11.5
Mean deflection	6.5	7.7	11.8
Maximum deflection	6.5	11.5	14.5

TABLE 7. Observed per cent I.D. deflection of 300 mm diameter plastic tubing

Depth (metres	1.5	4.5	7.5
300 mm I.D.	3.0	6.9	31.1
	3.0	20.2	23.7
	3.0	14.2	20.2
	3.0	14.9	14.9
Mean deflection	3.0	14.1	22.4
Maximum deflection	3.0	20.2	31.1

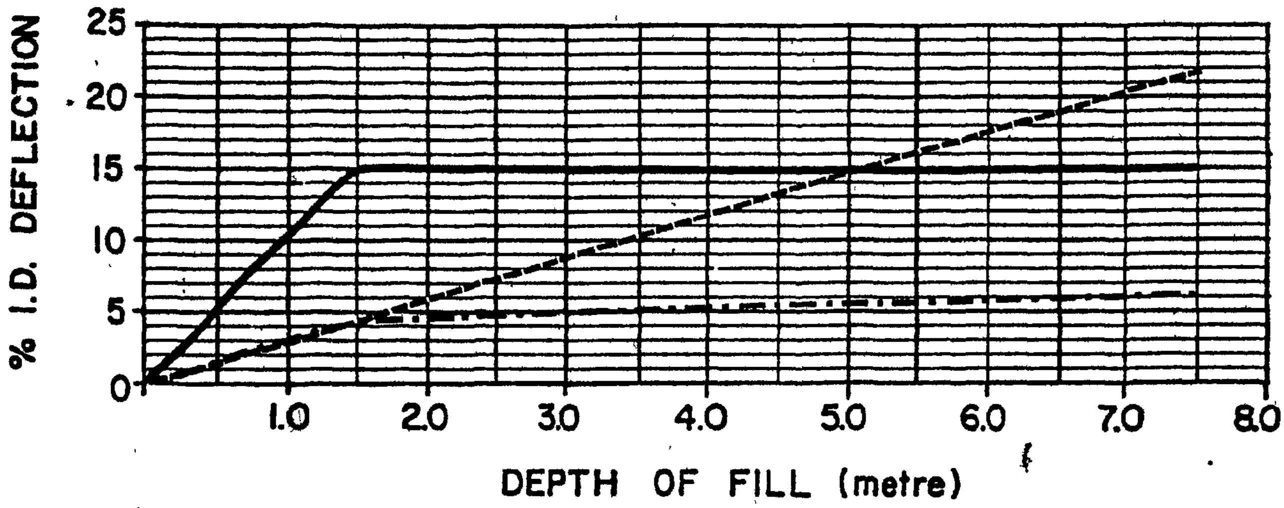


Figure 17. Measured and predicted deflections of 150 mm corrugated plastic tubing.

———— Measured deflections.

—·—·—· Predicted deflections considering a variable height of fill as a surcharge.

- - - - - Predicted deflections considering the total depth of fill as a unique load.

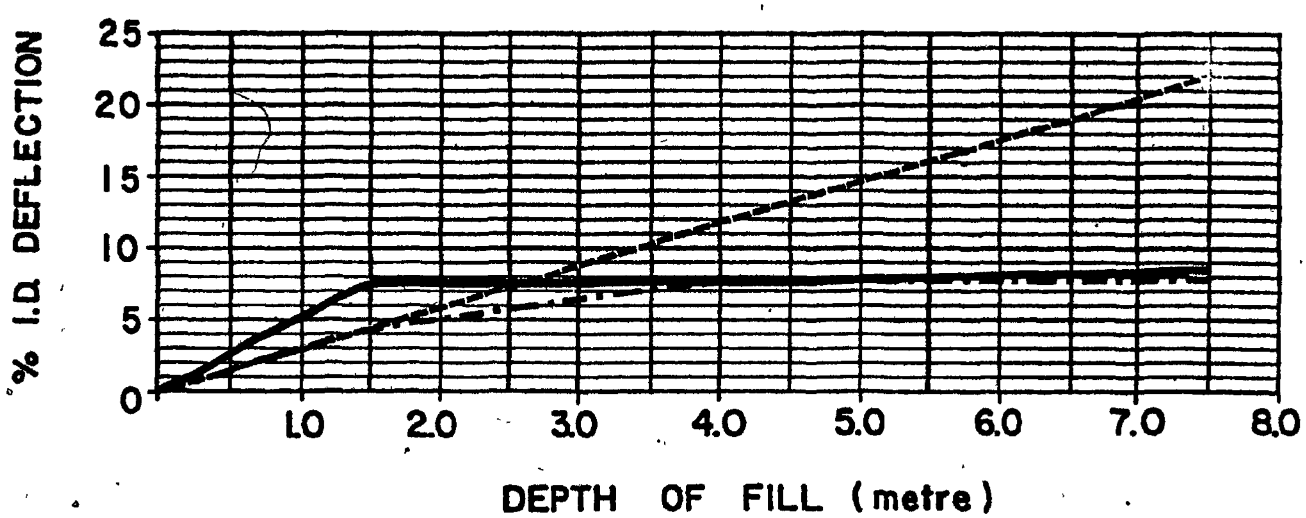


Figure 18. Measured and predicted deflections of 200 mm corrugated plastic tubing.

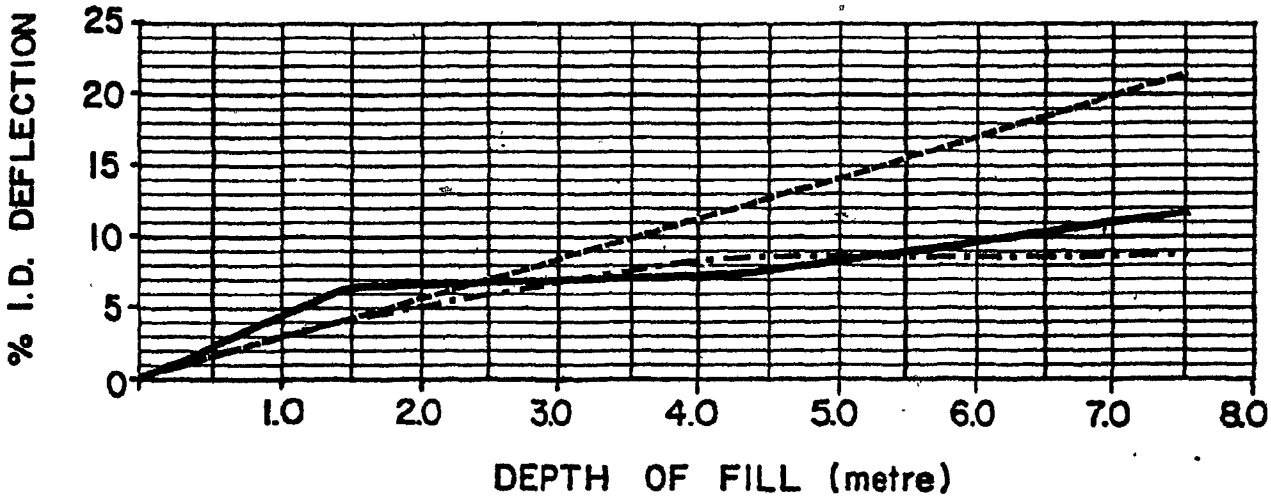


Figure 19. Measured and predcted deflections of 250 mm corrugated plastic tubing.

——— Measured deflections

-·-·-·- Predicted deflections considering a variable height of fill as a surcharge.

- - - - - Predicted deflections considering the total depth of fill as a unique load.

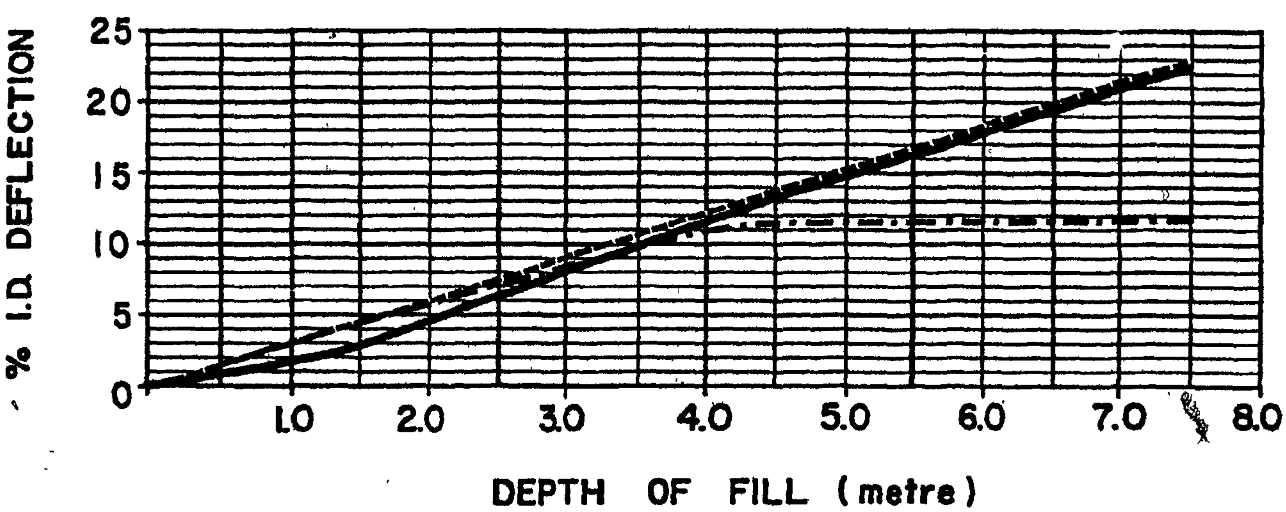


Figure 20. Measured and predicted deflections of 300 mm corrugated plastic tubing.

TABLE 8. Back-calculated modulus of soil reaction, E', for 1.5 metre of fill

Size (mm)	150	200	250	300
E' kPa	124	565	744	2400

TABLE 9. Total load, $W_t = W_c + W_{sd}$

Size (mm)	Total load W_t kN/m		
	Earth fill depth (m)		
	1.5	4.5	7.5
150	4.6	6.4	7.4
200	6.6	11.5	12.3
250	8.0	16.3	17.0
300	10.0	25.2	25.9

V. SUMMARY AND CONCLUSIONS

Professional engineers and organizations have recognized the need for installation standards for corrugated plastic tubing based on depth of installation up to at least 7 metres. The utilisation of corrugated plastic drain tubes should make drainage work less laborious and more economical because of its light weight as compared to alternative products. Concerns have been expressed regarding the probability of corrugated plastic tubing collapsing under deep earth fills. In this study the behaviour of plastic drain tubes under field conditions was investigated.

Four replicates of 80, 100, 150, 200, 250 and 300 mm I.D. corrugated plastic tubes were installed in the side of a hill on the Macdonald College Farm. Deformation of the vertical dimension inside the tubes was measured using wooden plug gauges.

During the investigation, the following points were noted:

1. If one considers deflections greater than 20% of the I.D. to be excessive, the 80 mm and 100 mm tubes placed on the crushed stone bedding suffered excessive deflection with fills greater than 3.5 metres.

2. The 100 mm tubes placed on the clay bed in 0.75 metre deep trenches and backfilled with crushed stone were satisfactory for earth fills up to the maximum depth of 8.2 metres.

3. The 150, 200, 250 mm diameter tubes placed on a crushed stone bedding have carried the maximum depth of 8.2 m satisfactorily.

4. One of the 300 mm diameter tubes deflected 27 per cent of the I.D. under a 5.0 m high fill. The other three 300 mm diameter tubes carried fill up to 7.0 m high with deflections less than 20.2 per cent and fills up to 8.2 m high with deflections less than 23.7 per cent.

5. A lag factor of 1.5, as suggested by Spangler and Hardy (1973), appears to be consistent with the values found in this experiment.

6. The calculated load using the projecting conduit conditions plus a surcharge yielded predicted deflections that follow the path of the measured deflections for the 150, 200, 250 mm tubes.

7. The predicted deflections considering the projecting conditions for the entire depth of fill, fit the measured deflections of the 300 mm tubes, within 1.5 per cent of the tubing I.D.

8. The degree of compaction and quality of bedding material near the tube wall affect small size tubes much more than larger tubes.

VI RECOMMENDED PRACTICES

1. If tubing is to be installed deeper than 2.0 metres, care should be taken to provide good bedding and backfill with sand or crumbly, stone-free soil for at least 150 mm away from the pipes.

2. Sharp-edged crushed stone should not be placed adjacent to the pipes.

3. Special care should be taken to compact the fill on the sides of the tubing, particularly for smaller diameter tubes (80, 100, 150 and 200 mm).

4. Use heavy duty tubing.

5. Do not install 80 and 100 mm diameter polyethylene tubing deeper than 2.0 m unless it is placed in a narrow trench before the fill overburden is added.

6. Where pipes must be placed in locations where they will later be covered over by great depths of soil, or mine tailings or other loads, the "imperfect trench" type of construction shown in Figure 1 should be used wherever possible, or alternately, a strip of deformable material such as soft styrofoam should be attached along the top of the pipe so that it could deform before the pipe deformed and thus allow a soil arching action develop in the earth fill. This would allow the soil passive pressure to build up on the sides of the pipe before the vertical load on the pipe built up. Credit should be given to Mr. D. Selby for this alternative suggestion for reducing the deflection of flexible pipes placed under deep fills.

VII. LOAD CARRYING CAPACITY OF CORRUGATED POLYETHYLENE TUBING PLACED IN TRENCHES

Pipe stiffness measurements were made in the laboratory by the flat plate method. From Figure 7 it can be seen that the tubing does not fail at a deflection of 10 per cent of the inside diameter, but obtains additional strength as deflections increase beyond 10 per cent.

Some tubes may have their ultimate strength at 25 per cent, and others at 30 or 35 per cent deflection. Also, the increase in load carrying capacities for deflections beyond 10 per cent I.D. may be relatively small for some tubes and large for others. It is therefore considered safe to use a deflection of 10 per cent I.D. for the purpose of calculating the safe supporting strength and the factor of safety for different loading situations with corrugated plastic drain tubing.

For smooth wall plastic pipe used in sanitary sewage applications, W.P.C.F. Manual no.9 recommends calculating the safe supporting strength for a deflection of five per cent of the inside diameter. The pipe does not fail at a deflection of five per cent of the I.D., but there is concern over connections of branch lines, and stress concentration effects.

Corrugated plastic tubing used for land drainage performs its water conveyance function quite satisfactorily at deflections of 20 per cent of I.D. or more. Since it is obvious from the geometry that the tubing will gain much more support from the soil as it deflects from 5 to 10 per cent I.D., it seems realistic and safe to base the factor of safety for load carrying capacity on the supporting strength at 10 per cent I.D. deflection. Standards for plastic drain tubes do not need to contain a complete range of load deflections, but should contain the values of stiffness obtained at 5 and 10 per cent I.D. deflection. The minimum pipe stiffness, $F/\Delta Y$, values allowed by the A.S.T.M. Standard F405 and the C.G.S.B. Standard 41-GP-29A are: 175 kPa at 5 per cent I.D. deflection and 131 kPa at 10 per cent I.D. deflection for standard quality tubing and 207 kPa at 5 per cent I.D. deflection and 172 kPa at 10 per cent I.D. deflection for heavy duty tubing.

The U.S. Soil Conservation Service standards also require that plastic tubing shows no cracks or other signs of failure when loaded to a deflection of 20 per cent I.D.

Tables C-6 to C-8 and C-11 to C-13 show factors of safety for 200, 250 and 300 mm diameter corrugated plastic tubes placed in trenches with soil cover depths ranging from 0.76 to 9.1 metres and subjected to live loading. H-10 live loading corresponds to the load due to a truck, tractor or wagon with a total load of 9100 kg (10 tons), 7300 kg of which is on the rear axle. Live loads of this magnitude are about the heaviest which could normally be expected to pass over drain pipes placed in agricultural soils. H-20 live loading corresponds to the load due to a vehicle of 18200 kg (20 tons), 14500 kg of which is on the rear axle. Live loads of this magnitude are expected on public roads and on construction sites.

The dead and live loads to be carried by the tubing are given in Tables C-3, C-4, C-5, C-9, C-10, and C-11. Values of coefficient C_d for use in equation 1 to obtain the dead load are given in Table C-1. Values of C_s for use in equation 5 to obtain the live loads are given in Table C-2. These values of C_d and C_s are obtained by the use of a graph and table given in Appendix A.

Trench widths ranging from 0.45 to 1.2 m have been used in these tables to give the full range of cases which might exist for various field installations. While it would be hoped that most drain tubes would be installed with trenchless plows or with trenching machines which give a grooved trench bottom on grade and a trench width from 0.45 to 0.75 m, it is recognized that some main drains will be installed in trenches made with a backhoe or power shovel which makes trenches wider than 0.75 metre.

The tables have been prepared for tubing which has a stiffness of 172 kPa when deflected 10 per cent of the inside diameter. Most, if not all, of the corrugated plastic tubes in the 200, 250 and 300 mm diameter classes are currently manufactured with stiffnesses greater than this.

For the factor of safety tables, C-6, C-7 and C-8, a slightly compacted backfill soil condition having a modulus of soil reaction of 1.4 MPa was considered. A wet specific soil weight of 1.6 g/cm³ (100 lb/ft³ or 0.058 lb/in³) has been considered. This is consistent with a loose backfill.

For Tables C-11, C-12 and C-13, compacted fill to 85 per cent of the proctor density was assumed, which would yield a modulus of soil reaction of about 2.8 MPa. The fill relative density for this case was considered to be 2.0 g/cm³ (0.072 lb/in³) and the pipe stiffness values were taken from a laboratory measurement.

The factors of safety have been based on the load, W_t', calculated to cause the tubing to deflect 10 per cent of its inside diameter. In the WPCF Manual no.9, a factor of safety of 1.25 is considered adequate. The stiffness value of 172 kPa at 10 per cent I.D. deflection was used because this is the value which is required by ASTM Designation F405 for heavy duty corrugated polyethylene tubing.

Based on these reasonable and conservative restricting conditions, it can be seen from the attached tables that, if properly installed, the tubing is safe for all depths of fill up to 9.1 m for trench widths of 0.9 metres or less. For trenches as wide as 1.2 metre, the tubing should not be buried deeper than 4.5 m, unless sand or gravel is placed beside the tubing and compacted, or a tubing with a greater initial stiffness is used.

VIII. RECOMMENDATIONS FOR FURTHER RESEARCH

The following topics related to the work of this dissertation are considered important for further investigations:

1. The development of a deflection measuring device for corrugated tubes, which would provide more precise measures for installations than the one used in this study.

2. Measures of loads on buried tubes using pressure cells.

3. Experimentation on larger diameter pipes.

4. Experimentation on other bedding materials and bedding conditions.

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APPENDIX A

Figure A-1

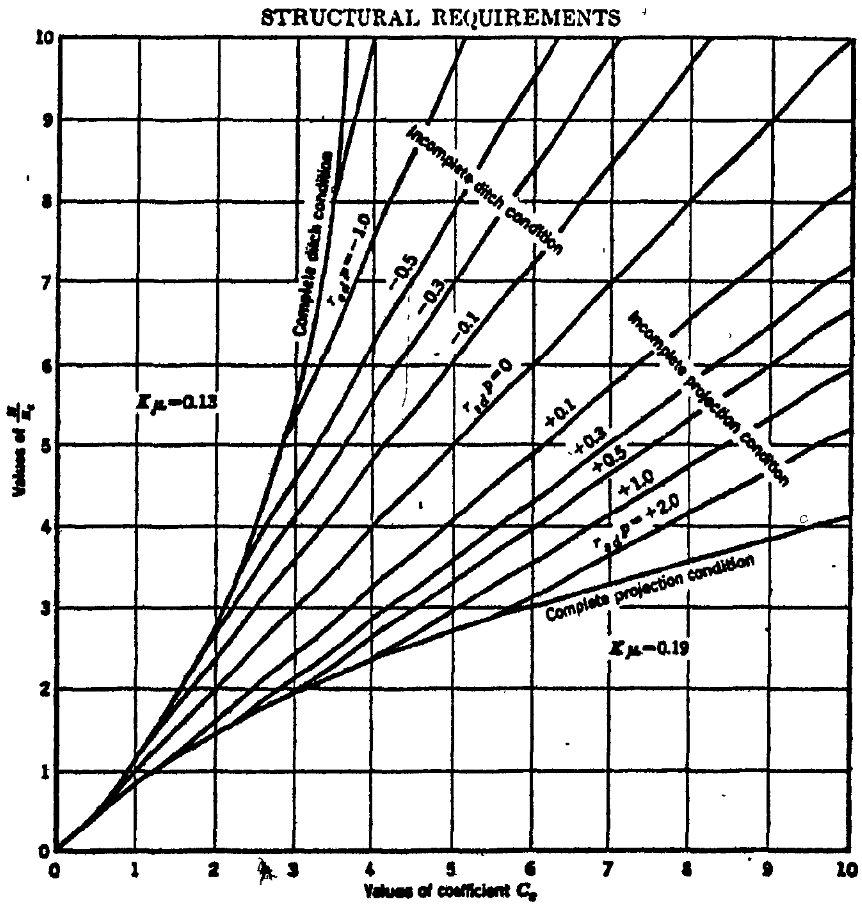


Diagram for coefficient C_c for positive projecting conduits.

(After WPCF Manual)

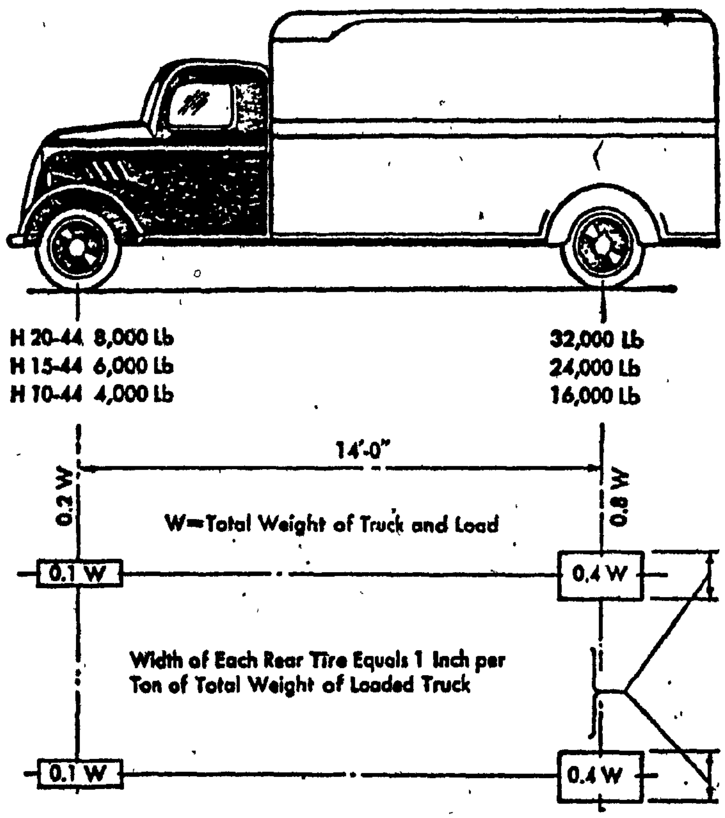
TABLE A-1. Values of load coefficients, C_s, for concentrated and distributed superimposed loads vertically centered over conduit*

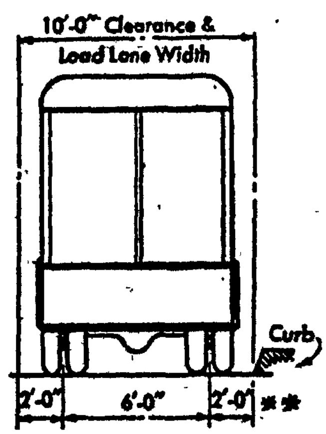
$\frac{D}{2H}$ or $\frac{B_a}{2H}$	$\frac{M}{2H}$ or $\frac{L}{2H}$													
	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9	1.0	1.2	1.5	2.0	5.0
0.1	0.019	0.037	0.053	0.067	0.079	0.089	0.097	0.103	0.108	0.112	0.117	0.121	0.124	0.128
0.2	0.037	0.072	0.103	0.131	0.155	0.174	0.189	0.202	0.211	0.219	0.229	0.238	0.244	0.248
0.3	0.053	0.103	0.149	0.190	0.224	0.252	0.274	0.292	0.306	0.318	0.333	0.345	0.355	0.360
0.4	0.067	0.131	0.190	0.241	0.284	0.320	0.349	0.373	0.391	0.405	0.425	0.440	0.454	0.460
0.5	0.079	0.155	0.224	0.284	0.336	0.379	0.414	0.441	0.463	0.481	0.505	0.525	0.540	0.548
0.6	0.089	0.174	0.252	0.320	0.379	0.428	0.467	0.499	0.524	0.544	0.572	0.596	0.613	0.624
0.7	0.097	0.189	0.274	0.349	0.414	0.467	0.511	0.546	0.584	0.597	0.628	0.650	0.674	0.688
0.8	0.103	0.202	0.292	0.373	0.441	0.499	0.546	0.584	0.615	0.639	0.674	0.703	0.725	0.740
0.9	0.108	0.211	0.306	0.391	0.463	0.524	0.574	0.615	0.647	0.673	0.711	0.742	0.766	0.784
1.0	0.112	0.219	0.318	0.405	0.481	0.544	0.597	0.639	0.673	0.701	0.740	0.774	0.800	0.816
1.2	0.117	0.229	0.333	0.425	0.505	0.572	0.628	0.674	0.711	0.740	0.783	0.820	0.849	0.868
1.5	0.121	0.238	0.345	0.440	0.525	0.596	0.650	0.703	0.742	0.774	0.820	0.861	0.894	0.916
2.0	0.124	0.244	0.355	0.454	0.540	0.613	0.674	0.725	0.766	0.800	0.849	0.894	0.930	0.956

* Influence coefficients for solution of Holl's and Newmark's integration of the Boussinesq equation for vertical stress.

APPENDIX B

Figure B-1





* In the design of floors (concrete slabs, steel grid floors, and timber floors) for H 20 or H 20—S 16 loading, one axle load of 24,000 pounds or two axle loads of 16,000 pounds each, spaced 4 feet apart may be used, whichever produces the greater stress, instead of the 32,000 pound axle shown.

** For slab design the center line of wheel shall be assumed to be 1 foot from face of curb (See Art. 3. 3. 2. (b).)

Standard highway loadings. *In the design of bridge floors for H 20 (or H 20—S 16) loading, one axle of 24,000 lb or two axle loads of 16,000 lb each, spaced 4 ft apart may be used, whichever produces the greater stress, instead of the 32,000 lb axle shown.—Reproduced from Standard Specifications for Highway Bridges, AASHO, 1953.*

(After Armco Steel and Drainage Products)

APPENDIX C

TABLE C-1. Values of C_d for use in calculating the dead soil load above flexible conduits by Marston's formula $W_c = C_d W B_c B_d$*

Trench width B_d (mm)	Height of cover, H (metres)											
	0.15	0.30	0.46	0.61	0.76	1.1	1.5	3.0	4.6	6.1	7.6	9.1
304	0.47	0.87	1.25	1.50	1.80	2.20	2.70	3.20	3.30	3.30	3.30	3.30
457	0.32	0.60	0.87	1.20	1.30	1.70	2.30	2.90	3.20	3.20	3.30	3.30
610	0.23	0.47	0.67	0.87	1.00	1.40	1.80	2.60	3.00	3.20	3.20	3.30
762	0.19	0.38	0.55	0.71	0.86	1.20	1.50	2.30	2.80	2.90	3.20	3.20
914	0.16	0.31	0.47	0.60	0.74	0.95	1.30	2.20	2.70	2.80	3.10	3.20
1066	0.14	0.28	0.40	0.52	0.63	0.87	1.10	1.90	2.40	2.80	3.00	3.10
1219	0.13	0.24	0.36	0.47	0.57	0.78	1.00	1.80	2.30	2.70	2.90	3.00
1371	0.11	0.22	0.31	0.41	0.52	0.70	0.92	1.60	2.20	2.50	2.70	2.90
1524	0.10	0.19	0.28	0.38	0.46	0.63	0.86	1.50	2.00	2.40	2.60	2.80
1676		0.17	0.26	0.33	0.42	0.57	0.80	1.40	1.80	2.30	2.50	2.70
1829		0.16	0.23	0.31	0.38	0.53	0.74	1.40	1.70	2.10	2.40	2.60
1981		0.15	0.22	0.29	0.36	0.49	0.70	1.20	1.70	2.00	2.30	2.50
2133		0.14	0.20	0.28	0.34	0.46	0.65	1.20	1.50	1.80	2.20	2.40

* Based on data taken from line C on graph, Figure 45, page 189, WPFC Manual no.9, ASCE Manual of Practice No.37, 1969, or the line for K_u and K'_u = 0.150 in Figure 4 of «Underground conduits - an appraisal of modern research» by M. G. Spangler, Paper 2337, ASCE Proceedings, June 1947. Line C from which the values of C_d have been calculated is for a saturated loam. Values of C_d will be smaller for sand and gravel and larger for saturated clay.

TABLE C-2. C_s values to be used in live load calculations*

Nominal conduit diameter (mm)	Approximate outside diameter B_c (mm)	Height of cover over conduit H (metres)								
		0.15	0.30	0.46	0.61	0.76	1.1	1.5	3.0	4.6
200	250	0.742	0.454	0.296	0.203	0.146	0.085	0.050	0.020	0.005
250	300	0.855	0.525	0.347	0.240	0.174	0.100	0.053	0.024	0.006
300	380	1.505	0.610	0.418	0.293	0.213	0.125	0.066	0.030	0.008

* Based on loading theory given in WPCF Manual no.9 and interpolations between data from Table XXVI, p. 206, in WPCF Manual no.9.

TABLE C-3. Dead load W_c kN/m of conduit length to be carried by flexible conduits for various heights of soil cover and wet specific weight of soil of 1.6 g/cm³ (0.058 lb/in³)

Nominal conduit I.D. (mm)	Approx. conduit O.D. B_c (mm)	Trench width at conduit top, B_d (mm)	Height of cover over conduit H (metres)							
			0.76	1.1	1.5	3.0	4.6	6.1	7.6	9.1
200	250	457	2.5	3.2	4.2	5.3	5.8	5.8	6.0	6.0
200	250	610	2.5	3.3	4.4	6.3	7.4	7.9	7.9	8.1
200	250	762	2.6	3.7	4.6	7.0	8.6	8.8	9.8	9.8
200	250	914	2.6	3.5	4.7	8.1	9.8	10.2	11.4	11.7
200	250	1066	2.6	3.7	4.7	8.1	10.2	11.9	12.8	13.3
200	250	1219	2.8	3.9	4.9	8.8	11.2	13.1	14.2	14.7
250	300	457	2.8	3.7	5.1	6.3	7.0	7.0	7.2	7.2
250	300	610	3.0	4.0	5.3	7.5	8.8	9.3	9.3	9.6
250	300	762	3.2	4.4	5.4	8.4	10.2	10.7	11.7	11.7
250	300	914	3.3	4.2	5.8	9.6	11.9	12.3	13.7	14.0
250	300	1066	3.2	4.4	5.6	9.8	12.3	14.4	15.4	15.9
250	300	1219	3.3	4.6	5.8	10.5	13.5	15.8	17.0	17.5
300	380	457	3.5	4.7	6.3	7.9	8.8	8.8	9.1	9.1
300	380	610	3.7	5.1	6.7	9.5	11.0	11.7	11.7	12.1
300	380	762	3.9	5.4	6.8	10.5	12.8	13.3	14.7	14.7
300	380	914	4.0	5.3	7.2	12.1	14.9	15.4	17.0	17.5
300	380	1066	4.0	5.6	7.0	12.1	15.4	17.9	19.3	19.8
300	380	1219	4.2	5.8	7.4	13.1	16.8	19.8	21.2	21.9

* **Based on Marston's formula $W_c = C_d w B_c B_d$**

TABLE C-4. Live-loads W_{sc} in kN/m of conduit length* for H-10 loading or single axle load of 7272 kg (16,000 lb)

Nominal conduit diameter (mm)	Approx. outside diameter B_c (mm)	Height of cover over conduit H (metres)				
		0.76	1.1	1.5	3.0	4.6
200	250	7.2	4.2	2.5	1.1	0.2
250	300	8.4	4.9	2.6	1.2	0.4
300	380	10.3	6.1	3.2	1.4	0.4

* Based on Holl's integration of Boussinesq's formula given in WPCF Manual no.9, p.205, as $W_{sc} = C_s \frac{PL}{L}$, using an impact factor F of 1.25 for field travel, using L = 0.9 metre, using P = 3,636 kg (8,000 lb), and using C_s values from Table 11.

TABLE C-5. Total load W_t kN/m on top of a flexible corrugated polyethylene conduit* for soil density of 1.6 g/cm³ (0.058 lb/in³) - H-10 live load

Nominal conduit I.D. (mm)	Approx. conduit O.D. B_c (mm)	Trench width at conduit top, B_d (mm)	Height of cover over conduit H (metres)							
			0.76	1.1	1.5	3.0	4.6	6.1	7.6	9.1
200	250	457	9.6	7.4	6.7	6.3	6.0	5.8	6.0	6.0
200	250	610	9.6	7.5	6.8	7.4	7.5	7.9	7.9	8.1
200	250	762	9.8	7.9	7.0	8.1	8.8	8.8	9.8	9.8
200	250	914	9.8	7.7	7.2	9.1	10.0	10.2	11.4	11.7
200	250	1066	9.8	7.9	7.2	9.1	10.3	11.9	12.8	13.3
200	250	1219	10.0	8.1	7.4	9.8	11.4	13.1	14.2	14.7
250	300	457	11.2	8.6	7.7	7.5	7.4	9.3	9.3	9.6
250	300	610	11.4	8.9	7.9	8.8	9.1	9.3	9.3	9.6
250	300	762	11.6	9.3	8.1	9.6	10.5	10.7	11.7	11.7
250	300	914	11.7	9.1	8.4	10.9	12.3	12.3	13.7	14.0
250	300	1066	11.6	9.3	8.2	11.0	12.6	14.4	15.4	15.9
250	300	1219	11.7	9.5	8.4	11.7	13.8	15.8	17.0	17.5
300	380	457	13.8	10.9	9.5	9.3	9.1	8.8	9.1	9.1
300	380	610	14.0	11.2	9.8	10.9	11.4	11.7	11.7	12.1
300	380	762	14.2	11.6	10.0	11.9	13.1	13.3	14.7	14.7
300	380	914	14.4	11.4	10.3	13.5	15.2	15.4	17.0	17.5
300	380	1066	14.4	11.7	10.2	13.5	15.8	17.9	19.3	19.8
300	380	1219	14.5	11.9	10.5	14.5	17.2	19.8	21.2	21.9

$W_t = W_c + W_{sc}$ kN/m of conduit length

TABLE C-6. Factors of safety for 200 mm I.D. corrugated P.E. conduit based on the load to cause 10% deflection divided by the total load likely to occur with H-10 live loading and heights of cover shown

For conduit $F/\Delta Y$ = 172 kPa (25 lb/in²) at 10% I.D. deflection

soil E' = 1.4 MPa (200/in²)

W = 1.6 g/cm³ (0.058 lb/in³)

Trench width (mm)	Height of cover over conduit H (metres)							
	0.76	1.1	1.5	3.0	4.6	6.1	7.6	9.1
457	1.54	2.02	2.23	2.36	2.50	2.57	2.50	2.50
610	1.54	1.98	2.18	2.02	1.98	1.89	1.89	1.85
762	1.52	1.89	2.12	1.85	1.70	1.70	1.52	1.52
914	1.52	1.93	2.07	1.63	1.49	1.46	1.31	1.27
1066	1.52	1.89	2.07	1.63	1.44	1.25	1.16	1.12
1219	1.49	1.85	2.02	1.52	1.31	1.13	1.05	1.01

Loading to produce 10% deflection with 200 mm I.D. conduit having $F/\Delta Y$ 172 kPa at 10% I.D. deflection and soil E' = 1.4 MPa; is estimated as

$$W_t' = \frac{0.10(8.0)[0.149(25) + 0.061(200)]}{1.5 \times (0.10)} = 84.9 \text{ lb/in}^2, 585 \text{ kPa}$$

$$F.S. = W_t'/(W_c + W_{sc}) = W_t'/W_t = 84.9/W_t$$

TABLE C-7. Factors of safety for 250 mm I.D. corrugated P.E. conduit based on the load to cause 10% deflection divided by the total load likely to occur with H-10 live loading and heights of cover shown

For conduit $F/\Delta Y$ = 172 kPa (25 lb/in²) at 10% I.D. deflection

soil E' = 1.4 MPa (200/in²)

W = 1.6 g/cm³ (0.058 lb/in³)

Trench width (mm)	Height of cover over conduit H (metres)							
	0.76	1.1	1.5	3.0	4.6	6.1	7.6	9.1
457	1.66	2.17	2.41	2.47	2.53	2.00	2.00	1.93
610	1.63	2.08	2.30	2.12	2.04	2.00	2.00	1.93
762	1.61	2.00	2.31	1.93	1.77	1.74	1.58	1.58
914	1.58	2.04	2.21	1.71	1.52	1.52	1.36	1.33
1066	1.61	2.00	2.26	1.69	1.47	1.29	1.21	1.17
1219	1.58	1.97	2.21	1.58	1.34	1.18	1.09	1.06

Loading to produce 10% deflection with 250 mm I.D. conduit having $F/\Delta Y$ 172 kPa at 10% I.D. deflection and soil E' = 1.4 MPa; is estimated as

$$W_t' = \frac{0.10(10.0)[0.149(25) + 0.061(200)]}{1.5 \times (0.10)} = 106.2 \text{ lb/in}^2,\ 732 \text{ kPa}$$

$$F.S. = W_t'/(W_c + W_{sc}) = W_t'/W_t = 106.2/W_t$$

TABLE C-8. Factors of safety for 300 mm I.D. corrugated P.E. conduit based on the load to cause 10% deflection divided by the total load likely to occur with H-10 live loading and heights of cover shown

For conduit $F/\Delta Y$ = 172 kPa (25 lb/in²) at 10% I.D. deflection

soil E' = 1.4 MPa (200/in²)

W = 1.6 g/cm³ (0.058 lb/in³)

Trench width (mm)	Height of cover over conduit H (metres)							
	0.76	1.1	1.5	3.0	4.6	6.1	7.6	9.1
457	1.61	2.05	2.36	2.40	2.45	2.55	2.45	2.45
610	1.59	1.99	2.28	2.05	1.96	1.90	1.90	1.85
762	1.57	1.93	2.24	1.87	1.70	1.68	1.52	1.52
914	1.55	1.96	2.16	1.65	1.46	1.45	1.31	1.27
1066	1.55	1.90	2.20	1.65	1.42	1.25	1.16	1.13
1219	1.53	1.87	2.12	1.53	1.30	1.13	1.05	1.02

Loading to produce 10% deflection with 300 mm I.D. conduit having $F/\Delta Y$ 172 kPa at 10% I.D. deflection and soil E' = 1.4 MPa; is estimated as

$$W_t' = \frac{0.10(12.0)[0.149(25) + 0.061(200)]}{1.5 \times (0.10)} = 127.4 \text{ lb/in}^2,\ 878 \text{ kPa}$$

$$F.S. = W_t'/(W_c + W_{sc}) = W_t'/W_t = 127.4/W_t$$

TABLE C-9: Dead load W_c kN/m of conduit length to be carried by flexible conduits for various heights of soil cover*

Nominal conduit I.D. (mm)	Approx. conduit O.D. B_c (mm)	Trench width at conduit top, B_d (mm)	Height of cover over conduit H (metres)							
			0.76	1.1	1.5	3.0	4.6	6.1	7.6	9.1
200	250	457	3.0	3.9	5.3	6.7	7.2	7.2	7.5	7.5
200	250	610	3.0	4.2	5.4	7.9	9.1	9.6	9.6	9.6
200	250	762	3.3	4.6	5.4	8.8	10.5	10.5	12.1	12.1
200	250	914	3.3	5.1	6.0	10.0	12.3	12.8	14.0	14.5
200	250	1066	3.3	4.6	6.3	10.0	12.8	14.9	15.9	16.5
200	250	1219	3.5	4.7	6.1	10.9	13.8	16.3	17.5	18.2
250	300	610	3.7	5.1	6.5	9.5	10.9	11.6	11.6	11.6
250	300	762	3.9	5.4	6.8	10.5	12.8	13.1	14.5	14.5
250	300	914	4.0	5.4	7.0	11.9	14.0	15.2	16.8	17.5
250	300	1066	4.0	5.6	7.7	12.1	15.2	17.9	19.1	19.6
250	300	1219	4.2	5.6	7.4	13.1	16.6	19.6	21.0	21.7
300	380	610	5.4	6.3	8.2	11.7	13.7	14.5	14.5	14.5
300	380	762	4.9	6.7	8.6	13.1	15.9	16.5	18.2	18.2
300	380	914	5.1	6.7	8.9	15.1	18.4	19.1	21.2	21.7
300	380	1066	5.1	6.8	8.8	15.1	19.1	22.2	23.8	24.7
300	380	1219	5.3	7.0	9.1	16.3	20.8	24.5	27.3	27.3

* Based on Marston's formula $W_c = C_d w B_c B_d$

C_d is dimensionless, B_c and B_d (L)

w for this table is 2.0 g/cm³

TABLE C-10. Live-loads W_{sc} in kN/m of conduit length* for H-20 loading or single axle load of 14,545 kg (32,000 lb)

Nominal conduit diameter (mm)	Approx. outside diameter B_c (mm)	Height of cover over conduit (metres)				
		0.76	1.1	1.5	3.0	4.6
200	250	17.0	10.0	5.8	2.3	0.5
250	300	20.3	11.7	6.1	2.8	0.7
300	380	24.9	14.5	7.7	3.5	0.9

* Based on Holl's integration of Boussinesq's formula given in WPCF Manual no.9, p. 205, as

$$W_{sc} = \frac{PF}{L} C_s$$

using an impact factor F of 1.5 for highways and streets, using L = 0.9 m, using P = 7272 kg (16,000 lb) and C_s values from Table 11.

TABLE C-11. Total load W_t kN/m on top of flexible corrugated polyethylene tubing* for soil density of 2.0 g/cm³ (0.072 lb/in³) - H-20 live load

Nominal conduit I.D. (mm)	Approx. conduit O.D. B_c (mm)	Trench width at conduit top, B_d (mm)	Height of cover over conduit H (metres)							
			0.76	1.1	1.5	3.0	4.6	6.1	7.6	9.1
200	250	610	20.0	14.2	10.7	10.2	9.6	9.6	9.6	9.6
200	250	762	20.3	14.5	11.2	11.0	11.0	10.5	12.1	12.1
200	250	914	20.3	15.1	11.7	12.3	12.8	12.8	14.0	14.5
200	250	1066	20.3	14.5	12.1	12.3	13.3	14.9	15.9	16.5
200	250	1219	20.5	14.7	11.9	13.1	14.4	16.3	17.5	18.2
250	300	610	24.0	16.8	12.6	12.3	11.6	11.6	11.6	11.6
250	300	762	24.2	17.2	13.0	13.3	13.5	13.1	14.5	14.5
250	300	914	24.3	17.2	13.1	14.7	14.7	15.2	16.8	17.5
250	300	1066	24.3	17.3	13.8	14.9	15.9	17.9	19.1	19.6
250	300	1219	24.5	17.3	13.5	15.9	17.3	19.6	21.0	21.7
300	380	610	30.3	20.8	15.9	15.2	14.5	14.5	14.5	14.5
300	380	762	29.8	21.2	16.3	16.6	16.8	16.5	18.2	18.2
300	380	914	29.9	21.2	16.6	18.6	19.3	19.1	21.2	21.7
300	380	1066	29.9	21.4	17.2	18.6	20.1	22.2	23.8	24.7
300	380	1219	30.1	21.5	16.8	19.8	21.7	24.5	27.3	27.3

*$W_t = W_c + W_{sc}$ kN/m of conduit length

TABLE C-12. Factors of safety for 200 mm I.D. corrugated P.E. conduit based on the load to cause 10% deflection divided by the total load likely to occur with H-20 live loading and heights of cover shown

For conduit $F/\Delta Y$ = 201 kPa at 10% I.D. deflection

soil E' = 2.8 MPa (400/in²) compaction 85% Proctor max.

Trench width (mm)	Height of cover over conduit H (metres)							
	0.76	1.1	1.5	3.0	4.6	6.1	7.6	9.1
610	1.34	1.89	2.51	2.64	2.78	2.78	2.78	2.78
762	1.32	1.85	2.39	2.43	2.43	2.55	2.22	2.22
914	1.32	1.78	2.28	2.19	2.10	2.10	1.91	1.84
1066	1.32	2.39	2.22	2.19	2.02	1.80	1.68	1.63
1219	1.31	1.83	2.25	2.04	1.87	1.65	1.53	1.47

Loading to produce 10% deflection with 200 mm I.D. conduit having $F/\Delta Y$ = 201 kPa at 10% I.D. deflection and soil E' = 2.8 MPa; is estimated as

$$W'_t = \frac{0.10(8.00)[0.149(29.2 + 0.061(400)]}{1.5 \times (0.10)}$$

$$F.S. = W'_t/(W_c + W_{sc}) = W'_t/W_t = 153.3/W_t$$

TABLE C-13. Factors of safety for 250 mm I.D. corrugated P.E. conduit based on the load to cause 10% deflection divided by the total load likely to occur with H-20 live loading and heights of cover shown

For conduit F/ΔY = 172 kPa at 10% I.D. deflection

soil E' = 2.8 MPa (400/in²) compaction 85% Proctor max.

Trench width (mm)	Height of cover over conduit H (metres)							
	0.76	1.1	1.5	3.0	4.6	6.1	7.6	9.1
610	1.37	1.95	2.60	2.68	2.84	2.84	2.84	2.84
762	1.36	1.91	2.53	2.46	2.43	2.49	2.25	2.25
914	1.35	1.91	2.50	2.23	2.23	2.10	1.95	1.88
1066	1.34	1.89	2.44	2.06	1.89	1.67	1.56	1.51

Loading to produce 10% deflection with 250 mm I.D. conduit having F/ΔY = 172 kPa at 10% I.D. deflection and soil E' = 2.8 MPa is estimated as

$$W_t' = \frac{0.10(10.0)[0.149(25) + 0.061(400)]}{1.5 \times (0.10)}$$

$$F.S. = W_t'/(W_c + W_{sc}) = W_t'/W_t = 187.5/W_t$$

TABLE C-14. Factors of safety for 300 mm I.D. corrugated P.E. conduit based on the load to cause 10% deflection divided by the total load likely to occur with H-20 live loading and heights of cover shown

For conduit $F/\Delta Y$ = 243 kPa at 10% I.D. deflection

soil E' = 2.8 MPa (400/in²) compaction 85% Proctor max.

Trench width (mm)	Height of cover over conduit H (metres)							
	0.76	1.1	1.5	3.0	4.6	6.1	7.6	9.1
610	1.37	1.99	2.61	2.73	2.86	2.86	2.86	2.86
762	1.40	1.96	2.55	2.50	2.47	2.52	2.28	2.28
914	1.39	1.96	2.50	2.24	2.16	2.18	1.96	1.91
1066	1.39	1.95	2.42	2.24	2.06	1.87	1.74	1.68
1219	1.38	1.93	2.47	2.10	1.91	1.70	1.52	1.52

Loading to produce 10% deflection with 300 mm I.D. conduit having $F/\Delta Y$ = 243 kPa at 10% I.D. deflection and soil E' = 2.8 MPa; is estimated as

$$W_t' = \frac{0.10(12.0)[0.149(35.3 + 0.061(400)]}{1.5 \times (0.10)} = 237.3 \text{ lb/in}^2,\ 1.6 \text{ MPa}$$

$$\text{F.S.} = W_t'/(W_c + W_{sc}) = W_t'/W_t = 237.3/W_t$$