# CHAPTER 4
# INSTALLATION TESTS

Pipe installation practices were evaluated through field and laboratory tests. The tests were designed to investigate the effects of different backfill materials and methods on pipe performance.

## 4.1 Laboratory Soil Box Tests

Twenty-five tests were conducted in a specially designed indoor test facility, called the "soil box," which allowed backfilling and compaction of materials around test pipes in a manner simulating certain aspects of field conditions. The soil box was designed for testing pipes with an outside diameter equal to or less than approximately 910 mm (36 in.) and trench widths varying from 1.5 to 2.5 pipe diameters. Tests were conducted with 760 mm (30 in.) inside diameter pipes. Test variables included trench wall stiffness, backfill material, method of compaction, haunching techniques, and bedding condition. The pipe, soil, and trench walls were monitored with a wide variety of instruments. The laboratory tests were conducted in part to evaluate the performance of pipe instrumentation being developed for the field test program described in section 4.2. The laboratory test procedures and data are presented in more detail in Zoladz (1995) and Zoladz et al. (1995).

### 4.1.1 Test Pipe

Three different types of pipes were included in the test program: (1) reinforced concrete (concrete); (2) corrugated, smooth interior wall, high density polyethylene (plastic); and (3) corrugated steel (metal). All test pipes were 760 mm (30 in.) in nominal inside diameter and 0.9 m (3 ft) in length.

The three types of pipes tested in this program span a wide range of pipe hoop stiffness and bending stiffness values and exhibit a wide range of pipe performance. The plastic and metal pipes are considered flexible in bending, whereas the concrete pipe is stiff in bending; however, the concrete and metal pipes are considered to have high hoop stiffness whereas the plastic pipe has a low hoop stiffness. Based on the bending stiffness values, plastic and metal pipes are typically considered flexible and the concrete pipe is considered rigid.

The reinforced concrete pipe was supplied by CSR/New England. Properties of the pipe are summarized in table 4.1. The concrete compressive strength and the concrete modulus of elasticity are estimated values, not test results.

**Table 4.1**
**Section Properties of a Concrete Pipe for Laboratory Tests**

| | |
|---|---|
| Inside diameter, $D_i$, mm (in.) | 760 (30) |
| Wall and thickness, mm, (in.) | Wall B, 89 (3.5) |
| Compressive strength, $f_c'$, MPa (psi) | 28 (4,000) |
| Modulus of elasticity, $E_c$, MPa (psi) | 25,000 ($3.7 \times 10^6$) |
| Cross-sectional area, A, $mm^2/mm$ ($in.^2/in.$) | 89 (3.5) |
| Wall moment of inertia, I, $mm^4/mm$ ($in.^4/in.$) | 58,700 (3.6) |
| Weight per unit length, $W_p$, kN/m (lb/ft) | 5.6 (380) |

The 900 mm (36 in.) diameter plastic pipe was supplied by Hancor, Inc. The pipe wall profile is shown in fig. 4.1a. Section properties were calculated based on measurements and the idealized geometry shown in fig. 4.1b, and are summarized in table 4.2. Two sets of section properties are provided; one assumes that the unbonded portion of the liner (element 1) is effective in carrying stress, and the second assumes that the unbonded portion is not effective. It is likely that the actual effectiveness of the liner is at an intermediate level that will vary with the relative liner thickness. McGrath, et al. (1994) have shown that for some corrugations the structural performance of the pipe is better represented by section properties computed assuming the liner is not effective. The modulus of elasticity is time dependent and can be estimated based on McGrath, et al. (1994). The value for the modulus of elasticity presented in table 4.2 is the AASHTO specified short term modulus.

The galvanized corrugated steel pipe was supplied by CONTECH Construction Products, Inc. Table 4.3 summarizes the pipe wall properties based on AASHTO (1996).

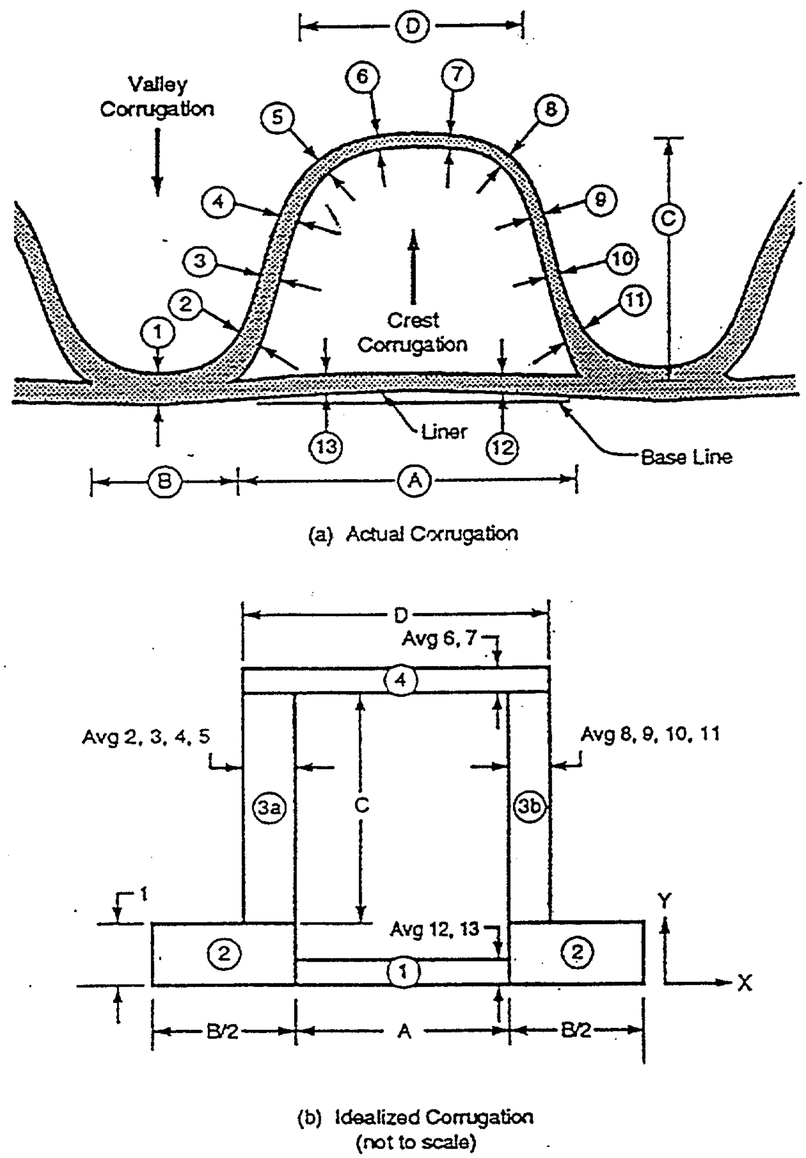



**Figure 4.1 Plastic Pipe Corrugation Profile**

**Table 4.2**
**Section Properties of a Plastic Pipe for Laboratory Tests**

| Property | Liner effective | Liner ineffective |
|---|---|---|
| Inside diameter, $D_i$, mm (in.) | 760 (30) | |
| Distance from inside surface to centroid, Y , mm (in.) | 28 (1.1) | 32 (1.3) |
| Short term modulus of elasticity, E, MPa (psi) | 780 ($1.1 \times 10^5$) | |
| Wall height, H, mm (in.) | 76 (3.0) | |
| Width of corrugation $L_c$, mm (in.) | 100 (3.9) | |
| Cross-sectional area A, $mm^2/mm$ ($in.^2/in.$) | 9.4 (0.4) | 8.1 (0.3) |
| Wall moment of inertia I, $mm^4/mm$ ($in.^4/in.$) | 6,100 (0.37) | 5,100 (0.31) |
| Section modulus to inner surface, $S_i$, $mm^3/mm$ ($in.^3/in.$) | 220 (0.34) | 160 (0.24) |
| Section modulus to outer surface, $S_o$, $mm^3/mm$ ($in.^3/in.$) | 130 (0.20) | 120 (0.18) |
| Weight per unit length, $W_p$, kN/m (lb/ft) | 0.27 (18.4) | |

**Table 4.3**
**Section Properties of a Metal Pipe for Laboratory Tests (AASHTO 1996)**

| | |
|---|---|
| Inside diameter, $D_i$, mm (in.) | 760 (30) |
| Corrugation size (in. x in., gage) | 2-2/3 x 1/2, 16 gage |
| Modulus of elasticity, E, MPa (psi) | 205,000 ($3.0 \times 10^7$) |
| Specified thickness, mm (in.) | 1.63 (0.064) |
| Cross-sectional area, A, $mm^2/mm$ ($in.^2/ft.$) | 1.64 (0.064) |
| Wall moment of inertia, I, $mm^4/mm$ ($in.^4/in.$) | 31 (0.0019) |
| Weight per unit length, $W_p$, kN/m (lb/in.) | 0.35 (24.3) |

The section properties of the test pipe and the bending stiffness and hoop stiffness are compared in table 4.4.

**Table 4.4**
**Summary of Properties of Laboratory Test Pipe**

**a. SI units**

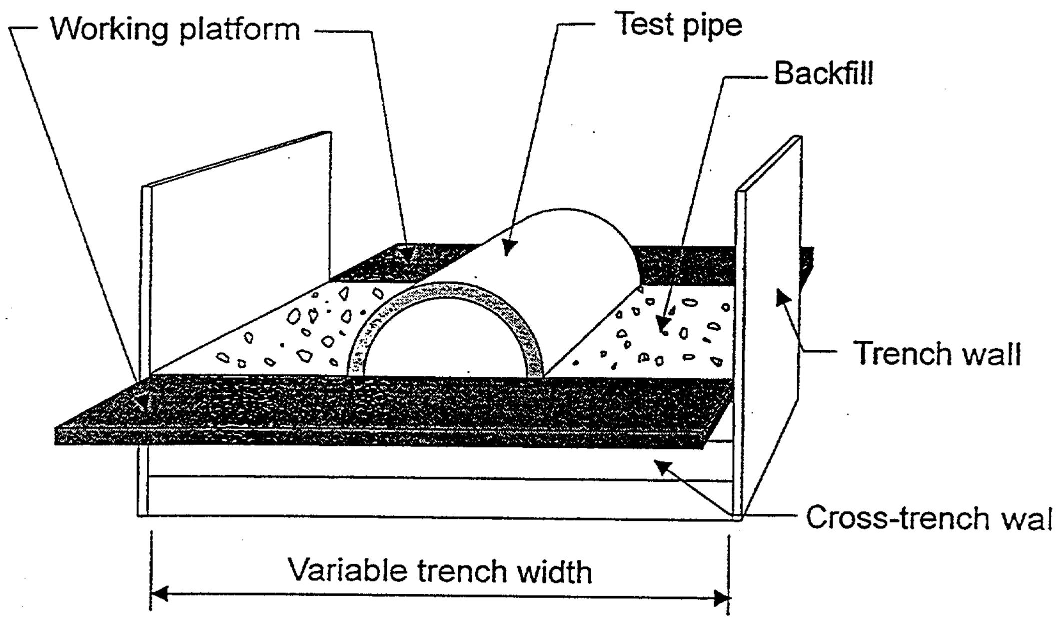
| Pipe Type | E (MPa) | Wall height (mm) | A ($mm^2/mm$) | I ($mm^4/mm$) | $PS_H$ (kN/m/m) | $PS_B$ (kN/m/m) |
|---|---|---|---|---|---|---|
| Concrete | 25,000 | 89 | 89 | 58,700 | $5.2 \times 10^6$ | $1.3 \times 10^5$ |
| Plastic (w/ liner) | 780 | 76 | 9.4 | 6,100 | $1.8 \times 10^4$ | $4.3 \times 10^2$ |
| Metal | 205,000 | 12.7 | 1.64 | 31.0 | $8.7 \times 10^5$ | $7.3 \times 10^2$ |

**b. English units**

| Pipe Type | E (psi) | Wall height (in.) | A ($in.^2/in.$) | I ($in.^4/in.$) | $PS_H$ (lb/in./in.) | $PS_B$ (lb/in./in.) |
|---|---|---|---|---|---|---|
| Concrete | $3.7 \times 10^6$ | 3.5 | 3.5 | 3.6 | 750,000 | 19,000 |
| Plastic | $1.1 \times 10^5$ | 3.0 | 0.4 | 0.37 | 2,600 | 62 |
| Metal | $3.0 \times 10^7$ | 0.5 | 0.06 | 0.0019 | 130,000 | 110 |

### 4.1.2 Soil Box

The soil box facility was designed to allow backfilling and compaction of the test pipe in a manner representative of actual practice. The box was designed for the pipe with an outside diameter of approximately 910 mm (36 in.) and trench widths varying from 1.5 to 2.5 pipe diameters. Fig. 4.2 is a schematic drawing of the primary elements of the soil box. For any given test, the trench walls were fixed, but the cross-trench walls could be raised, along with a platform surrounding the soil box, in 150 mm (6 in.) increments. This allowed compaction equipment to move from the platform at one end of the test pipe across the backfill to the platform on the other side of the test pipe, producing a reasonably realistic representation of a compactor moving along an actual pipe.




The working platform and the cross-trench wall are raised incrementally with the backfill elevation

**Figure 4.2 Primary Elements of the Soil Box**

**Trench Conditions** - The soil box was designed to have two trench widths, a wide trench, nominally 2.3 m (7.5 ft) wide, and a narrow trench nominally, 1.5 m (5 ft) wide. In situ soils were modeled with three different trench wall stiffnesses by incorporating foam material into the trench walls. Bare plywood walls were used as a "hard" trench wall test. A very soft 100 mm (4 in.) thick foam rubber with a modulus of elasticity determined in unconfined compression of 10 kPa (1.5 psi) was used for the "soft" trench wall tests and a 19 mm (0.75 in.) thick foam rubber with a modulus of elasticity determined to be 340 kPa (49 psi) was used in tests with "intermediate" trench wall stiffness.

The narrow trench was constructed by placing two wooden inserts at each end of the trench. The inserts have a height of 1.6 m (5.3 ft), length of 0.9 m (3 ft), and width of 130 mm (15 in.) when the three 90 mm by 90 mm (U.S. 4x4 nominal lumber) posts are in place. When bolted to the wide trench walls, the inserts reduce the width of the trench by 760 mm (30 in.).

Dimensions for each trench condition are illustrated in fig. 4.3. Values are given as a function of the outside diameter of the pipe. The ranges are between concrete and metal pipe,

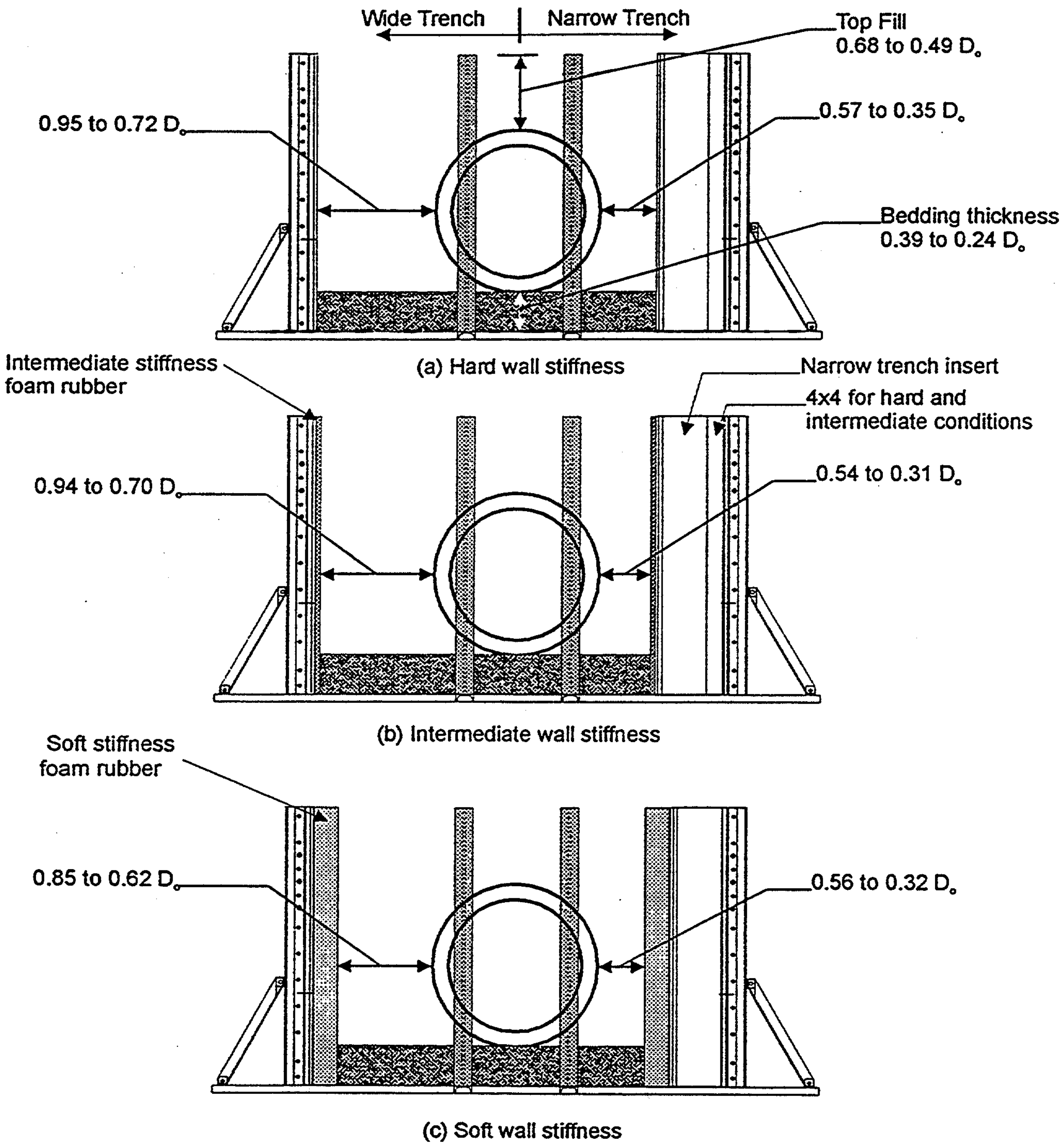



**Figure 4.3 Trench Box Wall Conditions**

which had the largest and smallest outside diameters, respectively, of the three pipe tested. The posts behind the narrow trench inserts are removed in the soft wall setup to compensate for the thickness of the foam.

### 4.1.3 Instrumentation

The behavior of the test pipe and the surrounding soil were monitored with several types of instrumentation during backfill placement. These instruments are described in more detail by Zoladz, (1995) and McGrath and Selig, (1996). Instruments included:

- A profilometer, using an LVDT, to measure pipe deflections and overall changes in pipe shape at 1-degree intervals around the pipe circumference.
- Visual extensometers mounted in the plastic pipe to measure changes in the pipe's diameter and verify the accuracy of the profilometer.
- Strain gages mounted in the plastic pipe.
- Pipe-soil interface pressure cells installed in the concrete (fluid filled earth pressure cells mounted in the pipe wall) and metal pipes (custom designed wall cutouts supported on instrumented support beams).
- Pressures cells mounted in the trench walls to measure horizontal soil stresses.
- Inductance coil strain gages mounted on the soft foam liner to measure soft wall displacements.
- A nuclear density gage to measure backfill moisture and soil density.
- A Proctor needle to measure soil strength in the haunch and bedding.
- Spring clamps mounted on the soil box were used to monitor gross pipe movements.

### 4.1.4 Backfill Materials and Compaction Equipment

Tests were conducted with pea gravel and rewash, characterized as Soil Nos. 4 and 6 in chapter 3. Hand tampers and shovel slicing were used to compact backfill in the pipe haunch zone.

Two types of hand-operated compaction equipment were used to compact the backfill: a rammer compactor (rammer) and a vibratory plate compactor (vibratory plate). The rammer is a Wacker model BS 60Y powered by a 1900 Watt (2.7 horsepower), twocycle engine (Wacker Corporation). The 280 mm (11 in.) wide and 330 mm (13 in.) long ramming shoe is driven into contact with the soil at a percussion rate of about 10 blows per second. The operating mass of the rammer is 60 kg (132 lb). The manufacturer's literature indicates that the generated dynamic force per blow is 10.2 kN (2,300 lb).

The vibratory plate is a Wacker model VPG 160B (Wacker Corporation) powered by a 3000 Watt (4 horsepower), four-cycle engine driving counter-rotating eccentric weights producing about 5,700 vibrations per minute. The vibratory plate compactor has an operating mass of 78.5 kg (173 lb) and, per the manufacturer's literature delivers a centrifugal force of 10.5 kN (2,350 lb). The contact area of the plate is 535 mm by 610 mm, (21 in. by 24 in.).

Compactor calibration tests were conducted in the soil box with pea gravel and silty sand to determine the soil unit weight achieved by varying the number of coverages with each compactor (fig. 4.4). Based on these results, the pea gravel was compacted with one coverage of the rammer or three coverages of the vibratory plate, while the silty sand was compacted with three coverages of the rammer or five of the vibratory plate. The increased number of passes required for the vibratory plate is a function of the much lower contact pressures. Filz and Brandon (1993, 1994) tested almost identical compactors and found that the peak force applied by the rammer was about four times greater than that applied by the vibratory plate, even though the catalog values for dynamic force are equal. The vibratory plate applied one half of the catalog value while the rammer applied twice the catalog value.

For tests where compaction of the haunch zone was required, two types of haunching effort were used. With pea gravel backfill, a procedure called "shovel slicing" was used, where the blade of a standard dirt shovel was sliced into the haunch material repeatedly. For tests backfilled with rewash, both shovel slicing and "rod tamping" were used. Rod tamping consisted of striking the backfill in the haunch zone with a 150 mm by 300 mm (3 in. by 6 in.) steel plate attached to a 2.4 m (8 ft) long steel pipe.

### 4.1.5 Test Procedures

Test variables included pipe type, trench width, trench wall stiffness, backfill material, method of compaction, method of haunching, and bedding condition.

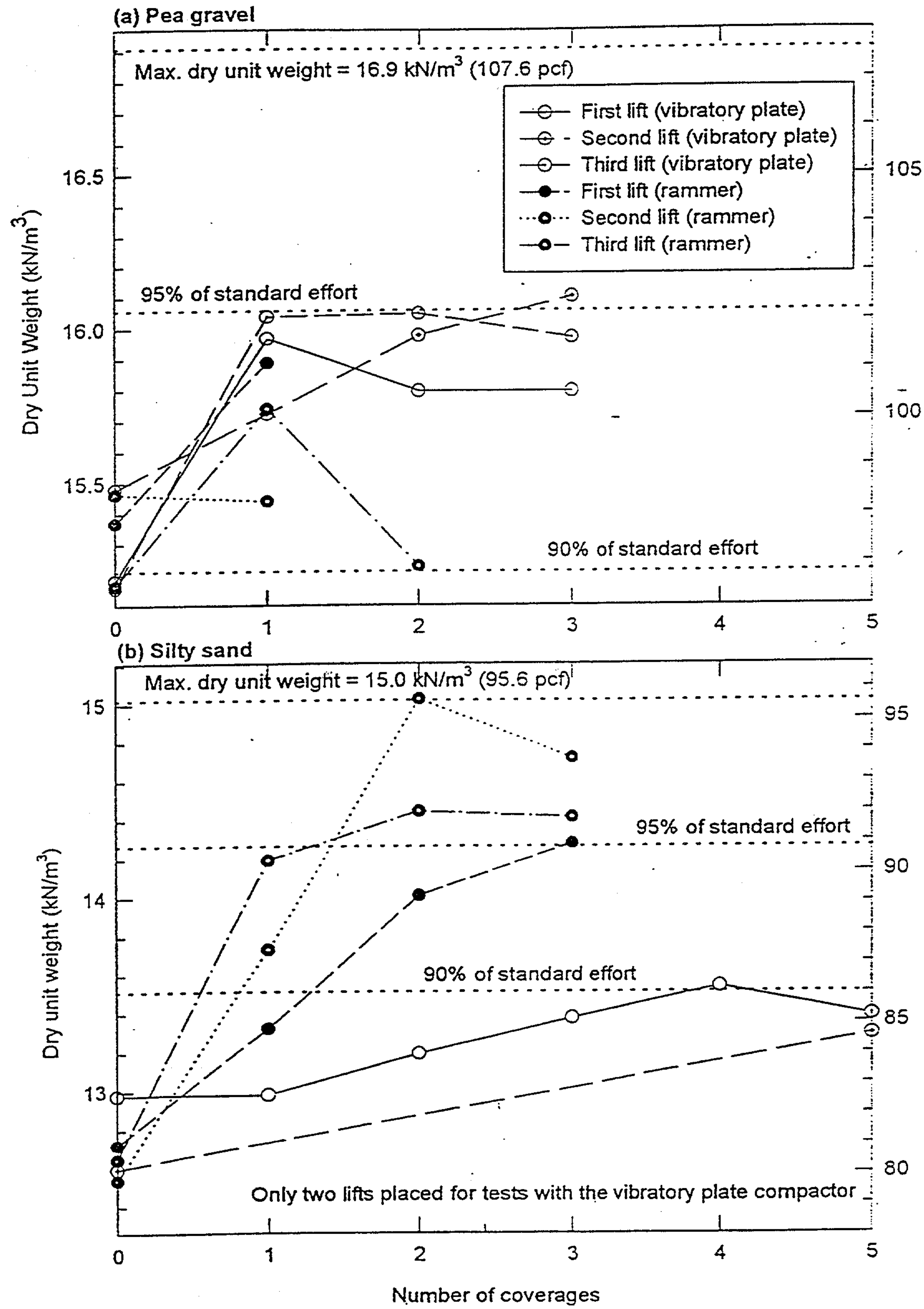



**Figure 4.4 Compactor Calibration Test Results**

The notation system, defined in table 4.5, was set up to identify test variables quickly. Figures and tables in this chapter use this system and identify variables in the order of test number, pipe type, trench condition, backfill, compactor, and haunching effort. Variables are removed from the label when indicated elsewhere in a figure. In addition to this notation, the backfill depth is often reported in terms of the normalized backfill depth, (NBD). This is the depth of the backfill relative to the top of the pipe divided by the outside diameter of the pipe. This simplifies interpreting the test results, as a normalized backfill depth of -1.0 is the bottom of the pipe, -0.5 is the springline, and 0.0 is the top of the pipe.

A total of 25 tests were conducted with the test variables listed in table 4.6. Because of the number of variables involved, it was impossible to test all combinations. The research team made selections of which combinations could provide the most information. Some tests were conducted primarily to evaluate the effects of compaction and haunch effort in the haunch zone. The backfill for these tests was brought only to a level at or near the springline. Other tests were backfilled to about 150 mm, (12 in.) over the top of the pipe.

**Table 4.5**
**Notation System for Laboratory Test Variables**

| Test variable | Symbol | Definition |
|---|---|---|
| Test No. | 1-25 | |
| Pipe type | CP | Concrete pipe |
| | MP | Metal pipe |
| | PP | Plastic pipe |
| Trench conditions | WH | Wide trench with hard walls |
| | WI | Wide trench with intermediate wall stiffness |
| | WS | Wide trench with soft wall stiffness |
| | NH | Narrow trench with hard walls |
| | NI | Narrow trench with intermediate wall stiffness |
| | NS | Narrow trench with soft wall stiffness |
| Backfill material | PG | Pea gravel |
| | SS | Silty sand |
| Method of compaction | RM | Rammer compactor |
| | VP | Vibratory plate compactor |
| | XC | No compaction |
| Haunching effort | RT | Rod tamping |
| | SH | Shovel slicing |
| | XH | No haunching |

**Table 4.6**
**Variables for Laboratory Tests**

| Test No. | Pipe | Trench condition | Backfill | Lift thickness mm, (in.) | Compactor | Haunch effort | Bedding | Final backfill depth (NBD) |
|---|---|---|---|---|---|---|---|---|
| 1 | CP | WH | PG | 305 (12) | XC | XH, SH | C | -0.68 |
| 2 | CP | WH | PG | 150 (6) | VP, RM | XH | C | -0.51 |
| 3 | PP | WH | PG | 305 (12) | XC | XH, SH | C | -0.33 |
| 4 | PP | WH | PG | 150 (6) | VP | XH | C | -0.33 |
| 5 | PP | WH | PG | 150 (6) | RM | XH | C | -0.33 |
| 6 | PP | NH | PG | 150 (6) | RM | XH | C | -0.33 |
| 7 | MP | WH | PG | 150 (6) | VP | XH | C | 0.65 |
| 8 | MP | WH | PG | 150 (6) | RM | XH | C | 0.65 |
| 9 | PP | WH | PG | 305 (12) | RM | XH | C | 0.50 |
| 10 | CP | WS | PG | 305 (12) | RM | XH | C | 0.30 |
| 11 | CP | WH | PG | 305 (12) | RM | XH | U | 0.30 |
| 12 | PP | WS | PG | 305 (12) | RM | XH | U | 0.33 |
| 13 | CP | NS | PG | 305 (12) | RM | XH | U | 0.30 |
| 14 | PP | NS | PG | 305 (12) | RM | XH | U | 0.33 |
| 15 | PP | NH | PG | 305 (12) | RM | XH | C | 0.33 |
| 16 | CP | NH | PG | 305 (12) | RM | XH | C | 0.30 |
| 17 | CP | WH | SS | 305 (12) | XC | XH, SH | C | -0.35 |
| 18 | CP | WH | SS | 305 (12) | VP, RM | XH | C | -0.35 |
| 19 | CP | WH | SS | 305 (12) | VP, RM | SH | U | -0.35 |
| 20 | MP | WH | SS | 305 (12) | VP | XH | C | -0.32 |
| 21 | MP | WI | SS | 305 (12) | VP | RT | C | -0.32 |
| 22 | MP | NH | SS | 305 (12) | RM | SH | U | -0.32 |
| 23 | CP | NH | SS | 305 (12) | RM | SH | U | -0.35 |
| 24 | CP | NI | SS | 305 (12) | RM | RT | C | -0.35 |
| 25 | MP | NI | SS | 305 (12) | RM | RT | C | -0.32 |

Tests were typically conducted in the following steps. Deviations from these procedures for specific tests are noted later.

1. Assemble soil box to required trench conditions.

2. Place and compact required bedding. Concrete and plastic pipes required a 230 mm (9 in.) bedding thickness, the metal pipe required a 305 mm (12 in.) thickness. Take density measurements at sidefill and invert locations.

3. Place pipe in trench and center the pipe between the lateral posts. The concrete and metal pipes required "in-air" readings of the interface pressure cells prior to placement. Take initial readings of all other instruments after placement.

4. Place first lift 305 mm (12 in.) deep for the concrete and metal pipes and 230 mm (9 in.) deep for the metal pipe. If haunching is to be conducted, place half the layer and haunch, then place the rest of the backfill.

5. Level off the lift and take uncompacted backfill readings. Uncompacted backfill readings are taken for the horizontal soil stresses, pipe-soil interface pressures, and soft wall displacements only.

6. Compact backfill as required and take compacted backfill readings. Compacted backfill readings are taken for all the instruments.

7. Repeat sequence of placing backfill, taking uncompacted readings, compacting, and taking compacted backfill readings until the final desired backfill depth is reached.

8. Remove backfill to at least 250 mm (10 in.) below springline and inspect the haunch zone. For tests with pea gravel, this consisted of carefully excavating under the pipe by hand. For tests with rewash, the pipe was removed and the backfill stiffness was evaluated with the Proctor penetrometer.

**Deviations from Typical Tests Procedures** – Variations from the standard procedures included the following:

- *Tests 1, 2, 3, 17, 18, 19* – Tests were conducted with a different compactors and/or different haunching method on each side of the pipe. Five of these tests were conducted with concrete pipe as it was felt that the compaction effects on one side of the pipe would not have any effect on the other side. The other test was conducted with polyethylene pipe with no mechanical compaction but with different haunching technique on each side of the pipe.

- *Instrumentation* – Electrical problems resulted in tests 3, 4, and 5 being conducted without the profilometer. Profilometer measurements were not conducted for the concrete pipe after test 16, as the concrete pipe did not show any measurable deflections. Horizontal soil stress cells were not installed in the trench walls until after test 9.

### 4.1.6 Results

This section presents and compares results from the 25 laboratory tests. Section 4.1.6.1 presents examples of each type of measurement taken, presented as a function of backfill depth. Complete results of each test are presented separately in Zoladz, et al. (1995). Subsequent sections compare results from different tests to demonstrate significant findings from the tests.

#### 4.1.6.1 Examples of Test Results

**Backfill Unit Weight, Pipe Deflections, and Gross Pipe Movement** – Figs. 4.5a to 4.5e show examples of the variations in several monitored parameters with increasing depth of backfill for test 9, conducted with pea gravel backfill and compaction with the rammer. Fig. 4.5 (a) indicates that the dry unit weight of the backfill was relatively uniform for each layer placed. Fig. 4.5 (b) shows the deflection versus depth of fill and indicates that while placing sidefill at elevations between the springline and the crown the pipe peaked (increased in vertical diameter and decreased in horizontal diameter), and deflected only slightly due to backfill over the top of the pipe. Figs. 4.5 (c) and 4.5 (d) show the lateral pipe movement at the springline relative to the soil box and indicates that the pipe springlines moved inward as backfill was placed from the springline to the crown. This is consistent with the deflections reported in Fig. 4.5 (b). Fig. 4.5 (e) indicates the change in elevation of the pipe invert as backfill is placed and indicates that the pipe is lifted up off the bedding as backfill is placed from the invert to about the springline level.

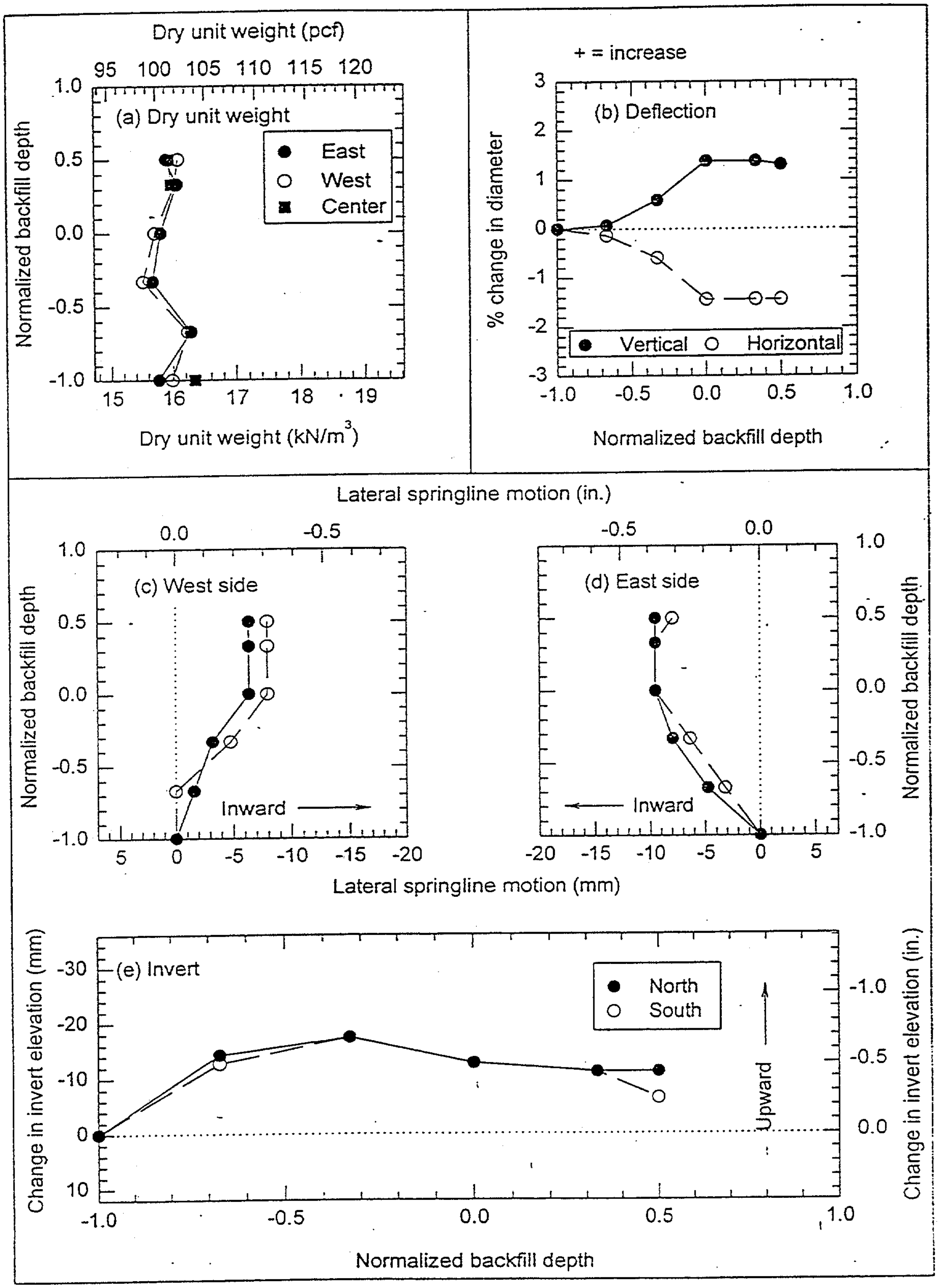



**Figure 4.5 Soil Unit Weight, Pipe Deflections, and Pipe Movement (Lab Test 9)**

**Profilometer Data** – Fig. 4.6 illustrates results of the profilometer measurements. The data from each profile measurement was smoothed by computing a running average of five degrees over the entire circumference of the pipe. The deformed shape is magnified ten times to improve readability. After magnification, the figures were aligned at the invert. Profilometer data were also used to determine changes in vertical and horizontal deflection.

**Horizontal Soil Stresses at the Trench Wall** – Fig. 4.7 presents average horizontal soil stresses at the trench wall, before and after compaction, from test 11 which was conducted using the concrete pipe placed in a wide trench with hard walls, pea gravel backfill, compaction with the rammer, and no haunching effort.

**Pipe-Soil Interface Pressures** – Fig. 4.8(a) presents the concrete pipe-soil interface pressures at the springline and 45 degrees below the springline (called the haunch in the figure) from test 11, both before and after compaction of each backfill lift. The figure suggests that even without haunching, when the rammer compactor is used with a free flowing material such as the pea gravel, significant radial pressures can develop at the haunch.

Further, Fig. 4.8(b) suggests that the rammer compactor is capable of lifting the concrete pipe sufficiently to lower the invert pressures, during compaction of the first lift. This is beneficial toward developing a uniform pressure distribution around the pipe.

**Plastic Pipe Strains** – Fig. 4.9 presents the plastic pipe strains measured during test 15, conducted with the plastic pipe placed in a narrow trench with hard walls, pea gravel backfill compacted with the rammer, and no haunching effort. Positive strains indicate tension. The strains are consistent with the other data, i.e., they indicate very little deformation during backfilling below the springline and then indicate that the pipe is being squeezed inward at the sides during compaction above the springline. The outside strains are higher than the inside strains which is consistent with the location of the neutral axis. Longitudinal strains are about 50 percent of the magnitude of the circumferential strains.

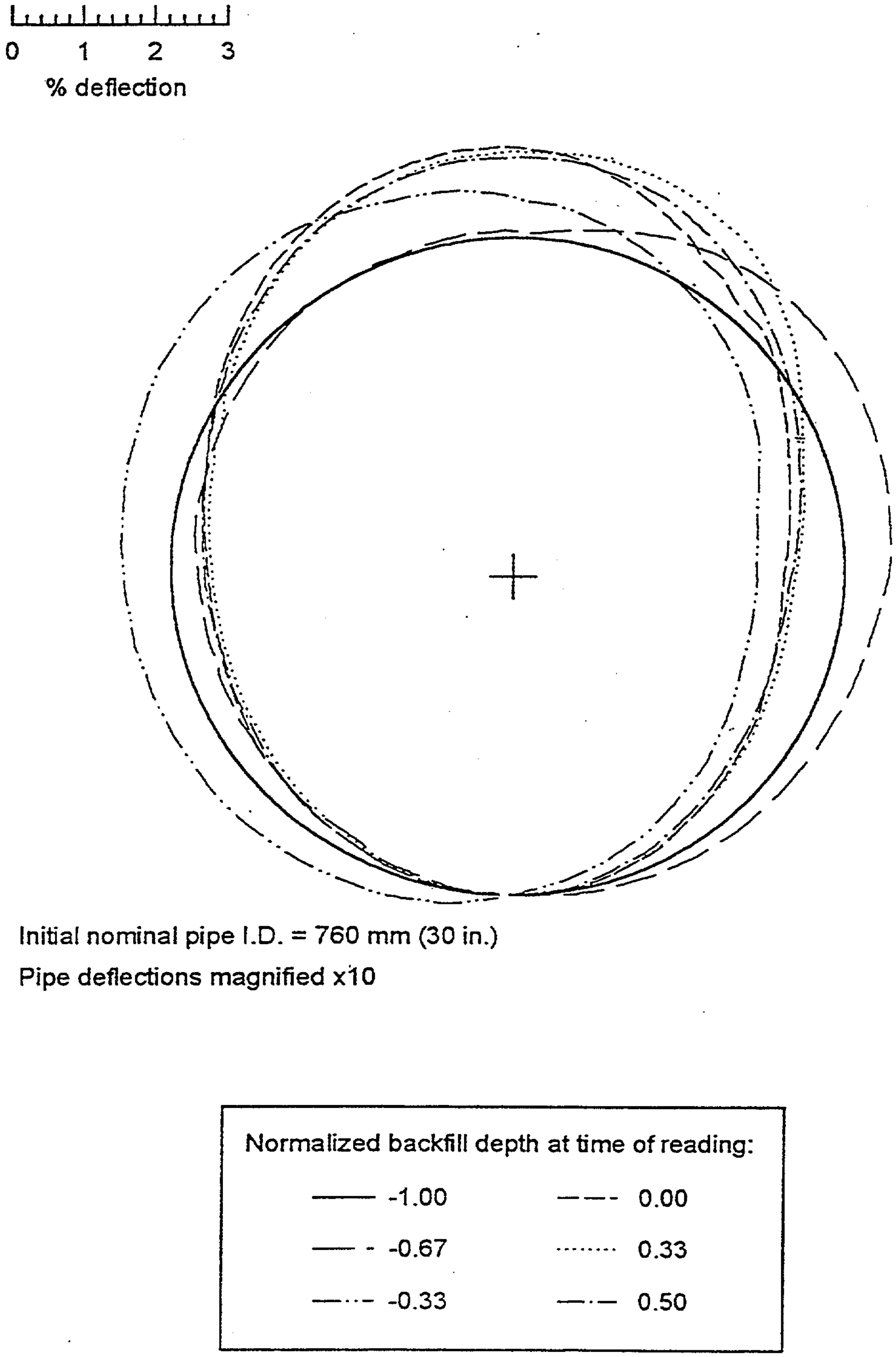
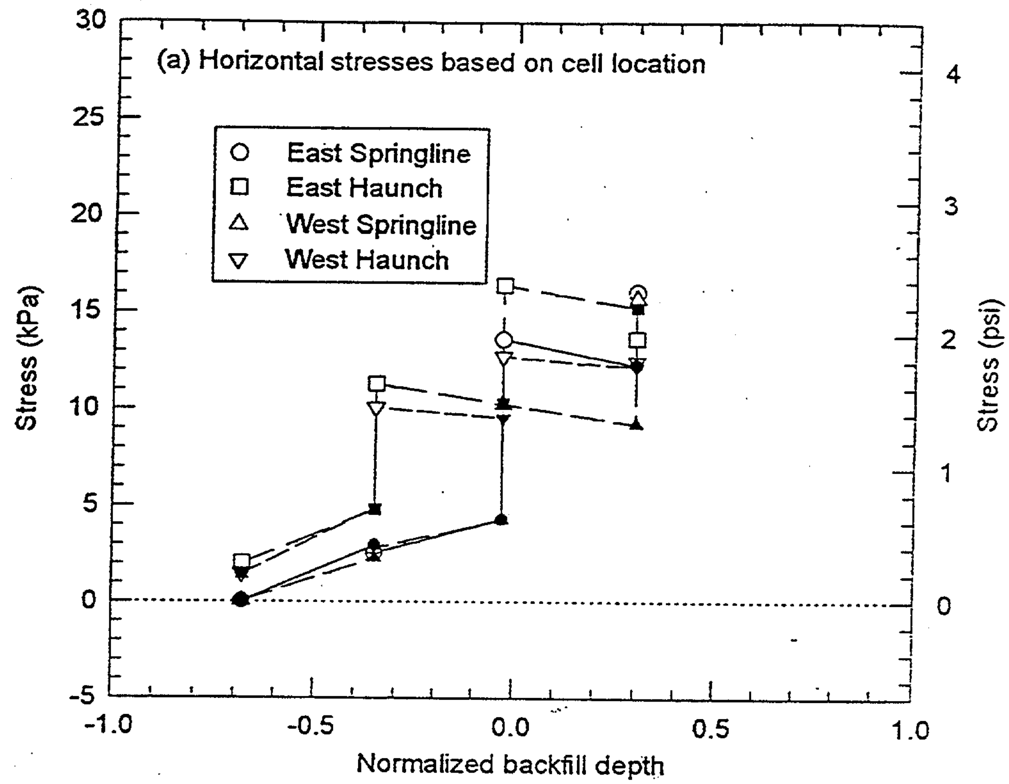



**Figure 4.6 Magnified Plastic Pipe Profiles (Lab Test 9)**




**Note: Filled symbols represent readings taken prior to compaction of backfill**

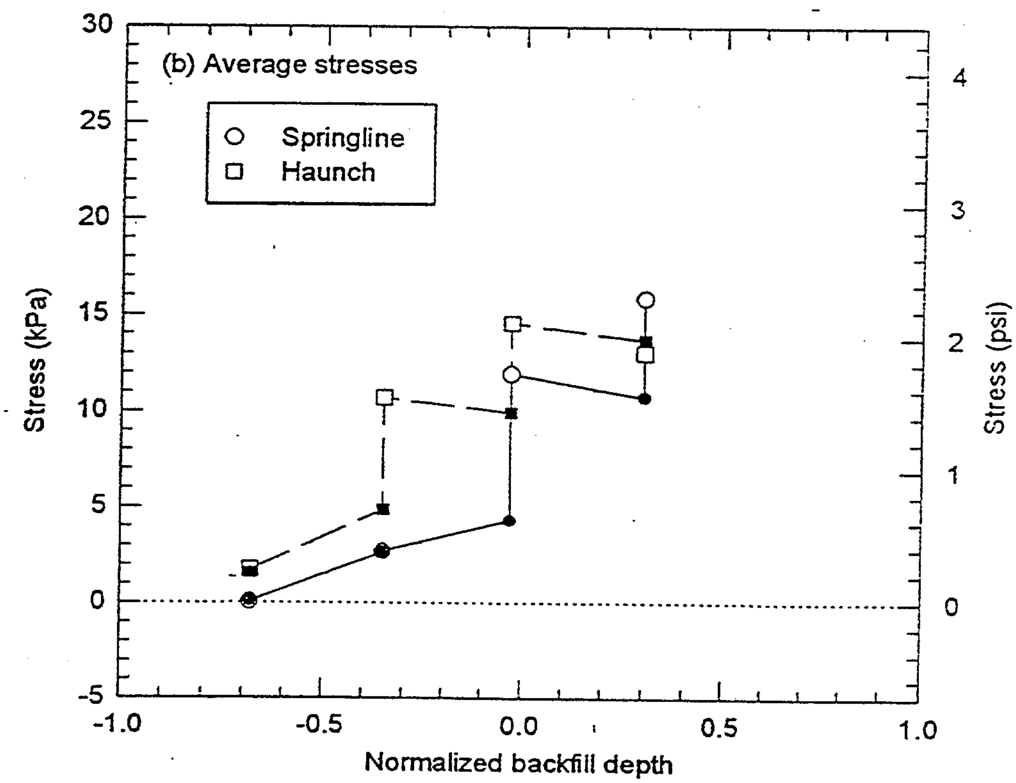



**Figure 4.7 Horizontal Soil Stresses at the Trench Wall (Lab Test 11)**

Note: Filled symbols represent readings taken prior to compaction of backfill

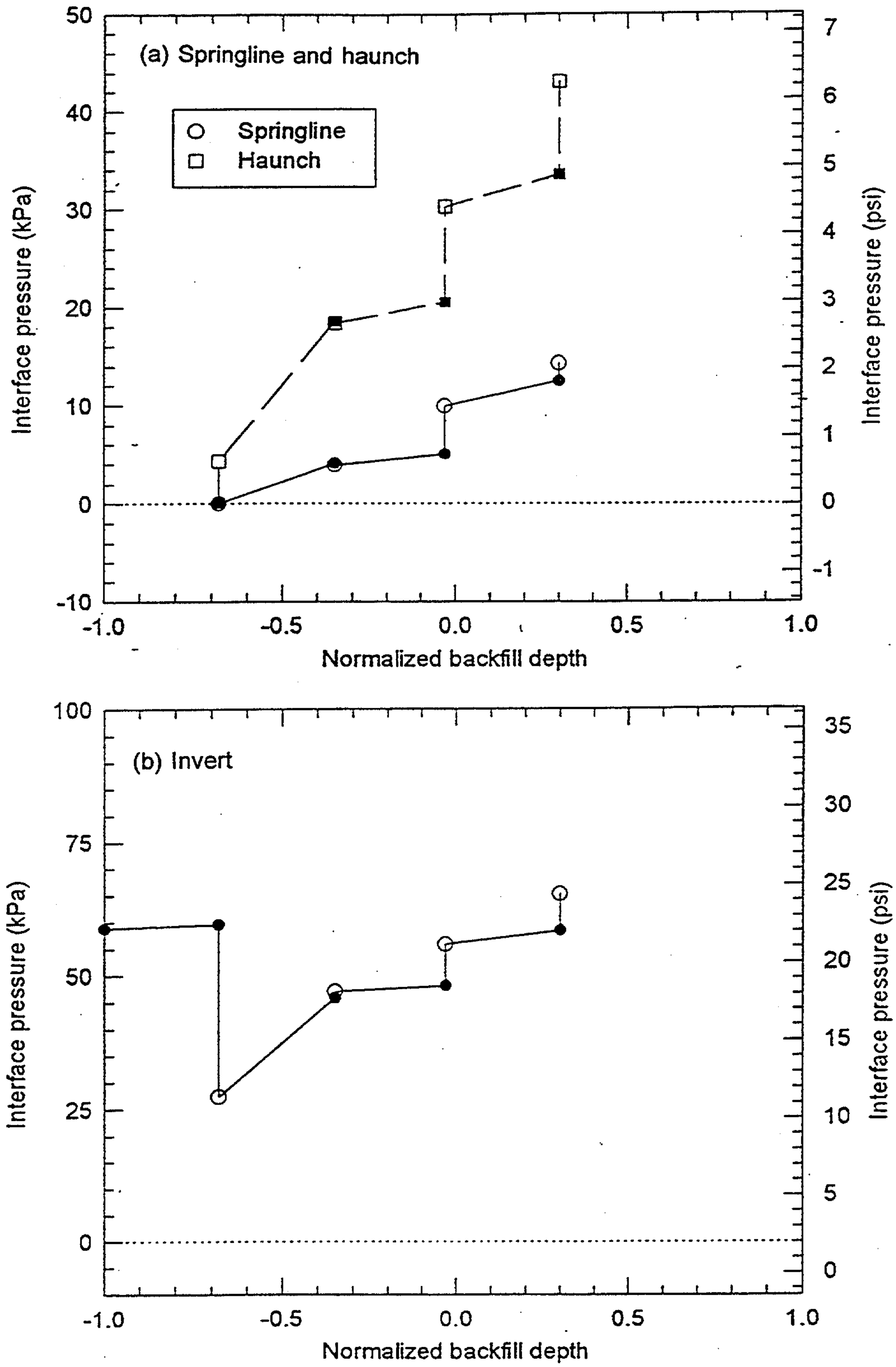



**Figure 4.8 Concrete Pipe-Soil Interface Pressures (Lab Test 11)**

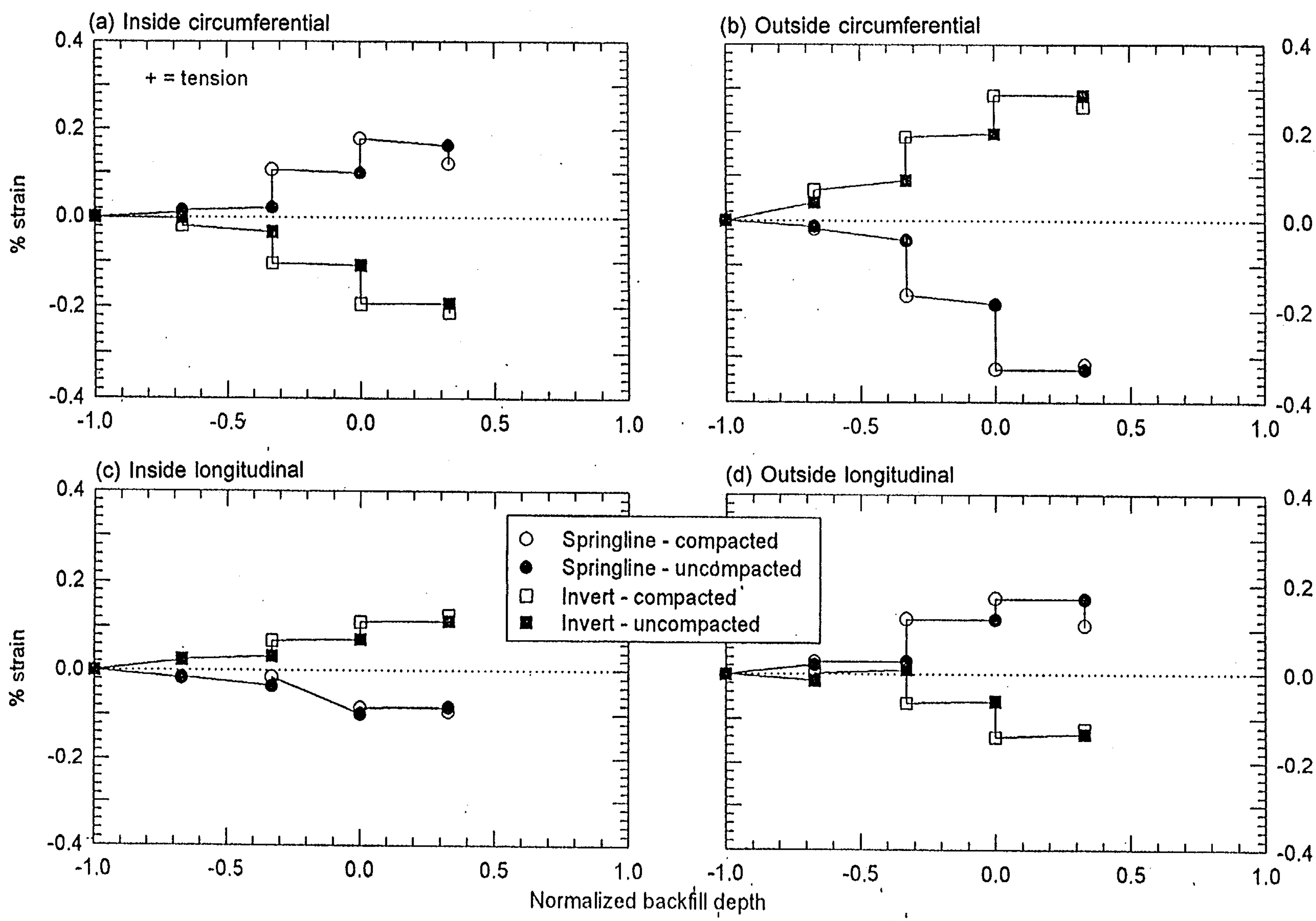



**Figure 4.9 Plastic Pipe Strains (Test 15)**

**Proctor Penetration Resistance** – Fig. 4.10 presents the results of penetrometer testing taken from test 21, performed with the metal pipe in a wide trench with intermediate stiffness walls, silty sand backfill, compacted with the vibratory plate, and the haunches compacted with the rod tamper. Data are presented for penetration depth of 25 mm (1 in.) and 50 mm (2 in.). The bedding soil was compacted for this test, and the invert showed the highest resistance. The penetration resistance at 30 and 60 degrees was similar, suggesting that the rod tamping used in the haunch zone was effective.

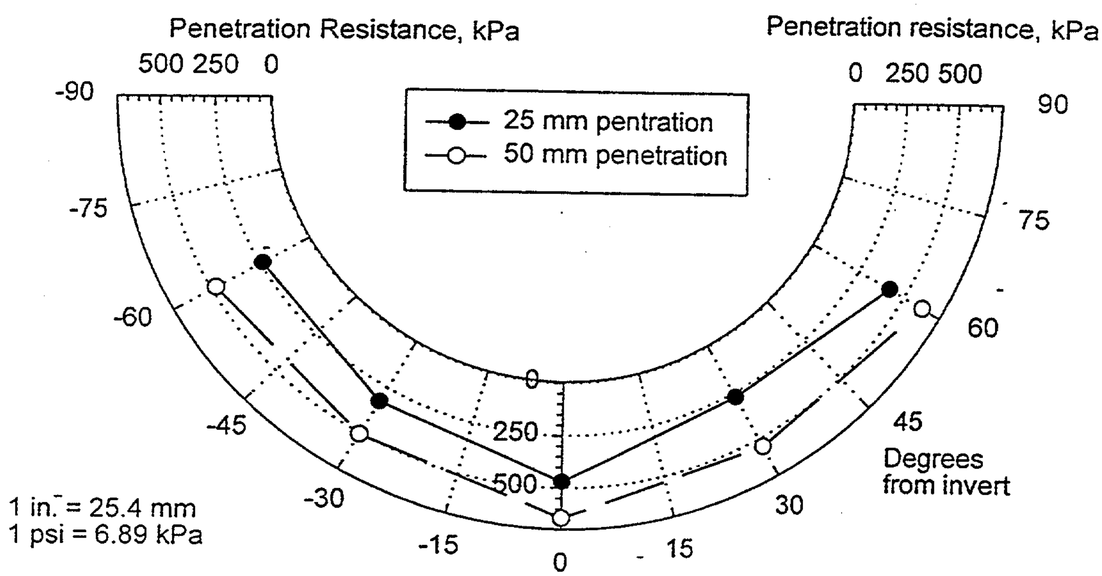
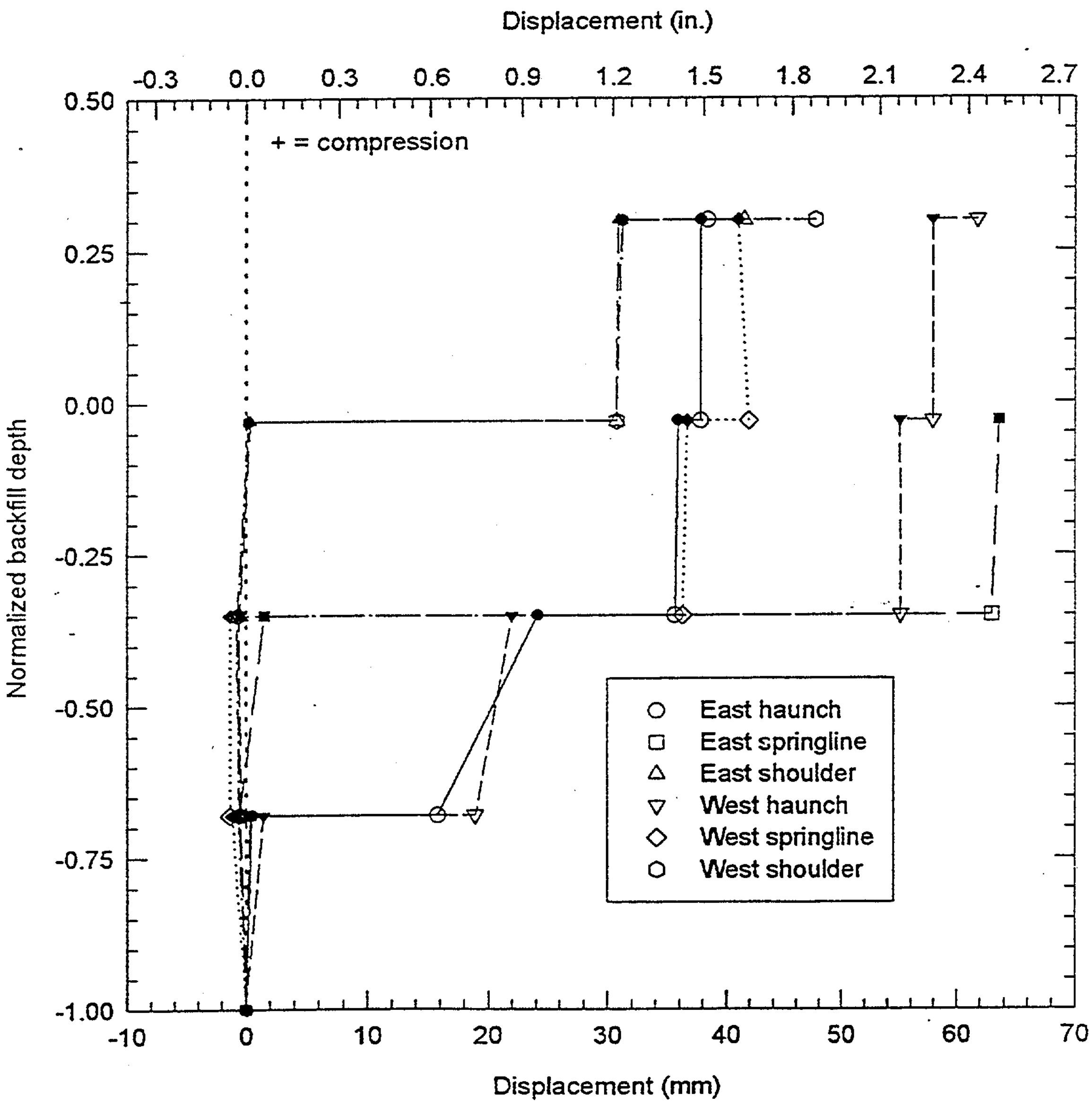



**Figure 4.10 Penetration Resistance of Bedding After Lab Test 21 in Silty Sand Metal Pipe, Vibratory Plate, Compaction, and Rod Tamping**

**Trench Wall Displacements** – Soft wall displacements for test 13 which was conducted with the concrete pipe placed in a narrow trench with soft walls, pea gravel backfill compacted with the rammer, and no haunching effort are presented in fig. 4.11. Most of the displacement in the wall occurred after the first layer was compacted near the inductance coils. As can be seen in fig. 4.11, as the first layer (NBD = -0.67) was compacted the walls at the haunch elevation compressed. As the second backfill layer (NBD = -0.33) was compacted, the walls at the springline elevation showed displacement and the walls in the haunch elevation continued to compress. This trend continued as the backfilling proceeded.




**Note: Filled symbols represent readings taken prior to compaction**

**Figure 4.11 Soft Trench Wall Displacements (Lab Test 13)**

#### 4.1.6.2 Vertical Pipe Movement

The data on vertical pipe movement show that the plastic and metal pipe lifted up from 15 to 25 mm (0.6 to 1.0 in.) when compacted with the rammer and from 0 to 12 mm (0.0 to 0.5 in.) when compacted with the vibratory plate. As noted above, this difference further emphasizes the significant difference in the applied stresses under the two types of compaction equipment. Only a small percentage of the uplift was recovered as fill was placed above the springline. The uplift is greater in silty sand than in pea gravel. When no compaction was applied the pipe dropped during placement of the sidefill. Uplift was significantly reduced when the trench walls were soft.

The values reported here should not be taken as indicative of actual field uplift values because the test lengths of pipe were short. In the field, the uplift would be resisted by the weight of pipe adjacent to the section being compacted (see section 4.2 for actual field data). However, the tests do suggest that compaction of the sidefill below the springline has the beneficial effects of reducing the invert pressure under a pipe. The reduced uplift noted when trench walls are soft indicate that the compactive energy deforms the trench wall and is less effective in forcing backfill into the haunch zone.

Only limited data were collected for the concrete pipe, and no uplift was noted. The pipe had settled downward 1 to 2 mm (0.04 to 0.08 in.) when backfill was at the springline level and up to 5 mm when backfill was placed to 300 mm (12 in.) over the top of the pipe. When trench walls were soft, the settlements at the springline level and at the final level were about twice the settlements measured for similar conditions with hard trench walls.

#### 4.1.6.3 Pipe Profiles and Deflections

The presentation of pipe profile and deflection data is limited to the tests with the plastic and metal pipes as the concrete pipe did not measurably deflect. The general trend of the deflections versus depth of fill is shown in fig. 4.12. The figure indicates the following:

- Most upward deflection occurs during compaction of backfill between the springline and crown level;

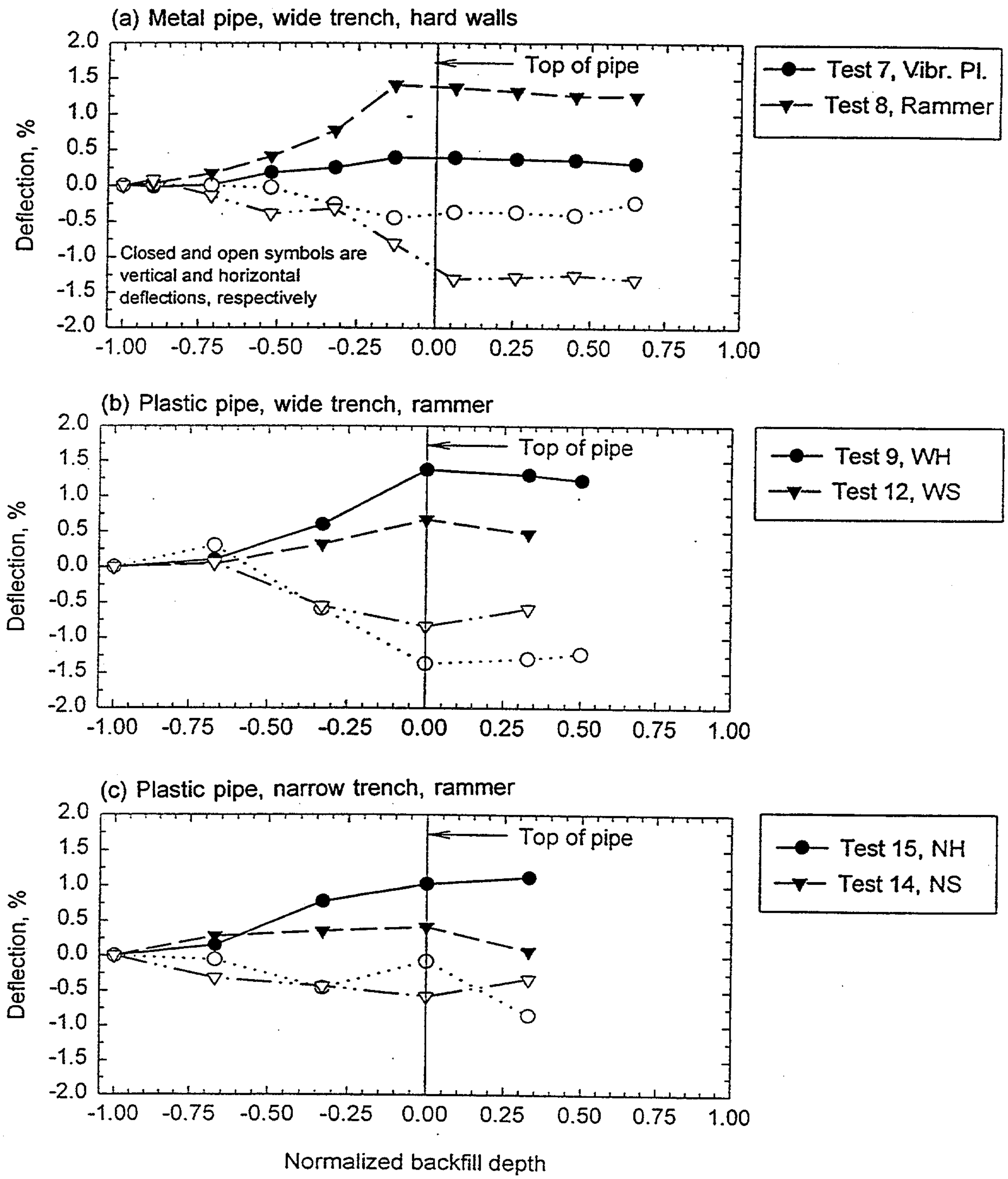



**Figure 4.12 Pipe Deflections in Laboratory Tests**

- The rammer creates much more upward deflection during compaction than the vibratory plate (fig. 4.12(a)); and
- Much more upward peaking occurs with the hard trench walls than with the soft trench walls, suggesting that some compaction energy is deforming the trench walls rather than densifying the soil.

Deflection data for a wider range of variables are presented in fig. 4.13 which shows the deflection magnitude when the backfill was at a level 150 mm (6 in.) above the springline. This figure also shows trends similar to those in fig. 4.12, and shows that pipe backfilled with silty sand deflects more during compaction than pipe backfilled with pea gravel.

Deflections when backfill is at the springline, the top of pipe, and at the end of the test, 300 mm (12 in.) or more over the top of the pipe for tests with pea gravel backfill are presented in fig. 4.14. The figure again shows the significant difference in peaking between the rammer and the vibratory plate, less peaking for installations with soft trench walls and increased downward deflection for tests with soft trench walls, even with only about 300 mm (12 in.) of backfill over the pipe. This indicates that compaction against soft trench walls is far less effective than against hard trench walls.

Profilometer and deflection data are shown in figs. 4.15 and 4.16 also demonstrate the effect of compaction method and trench wall stiffness respectively. Fig 4.15 shows that the rammer compactor produces more upward peaking than the vibratory plate. This suggests that the energy delivered by the rammer compactor is more concentrated than that delivered by the vibratory plate, which is consistent with the compactor calibrations that showed compaction to a specific density is achieved with fewer passers of the rammer relative to the vibratory plate. Fig. 4.16 shows that compaction when trench walls are soft results in substantially less peaking than when the walls are hard. This suggests that in the field contractors installing pipe in soft native soils will need to pay extra attention to the compaction procedures.

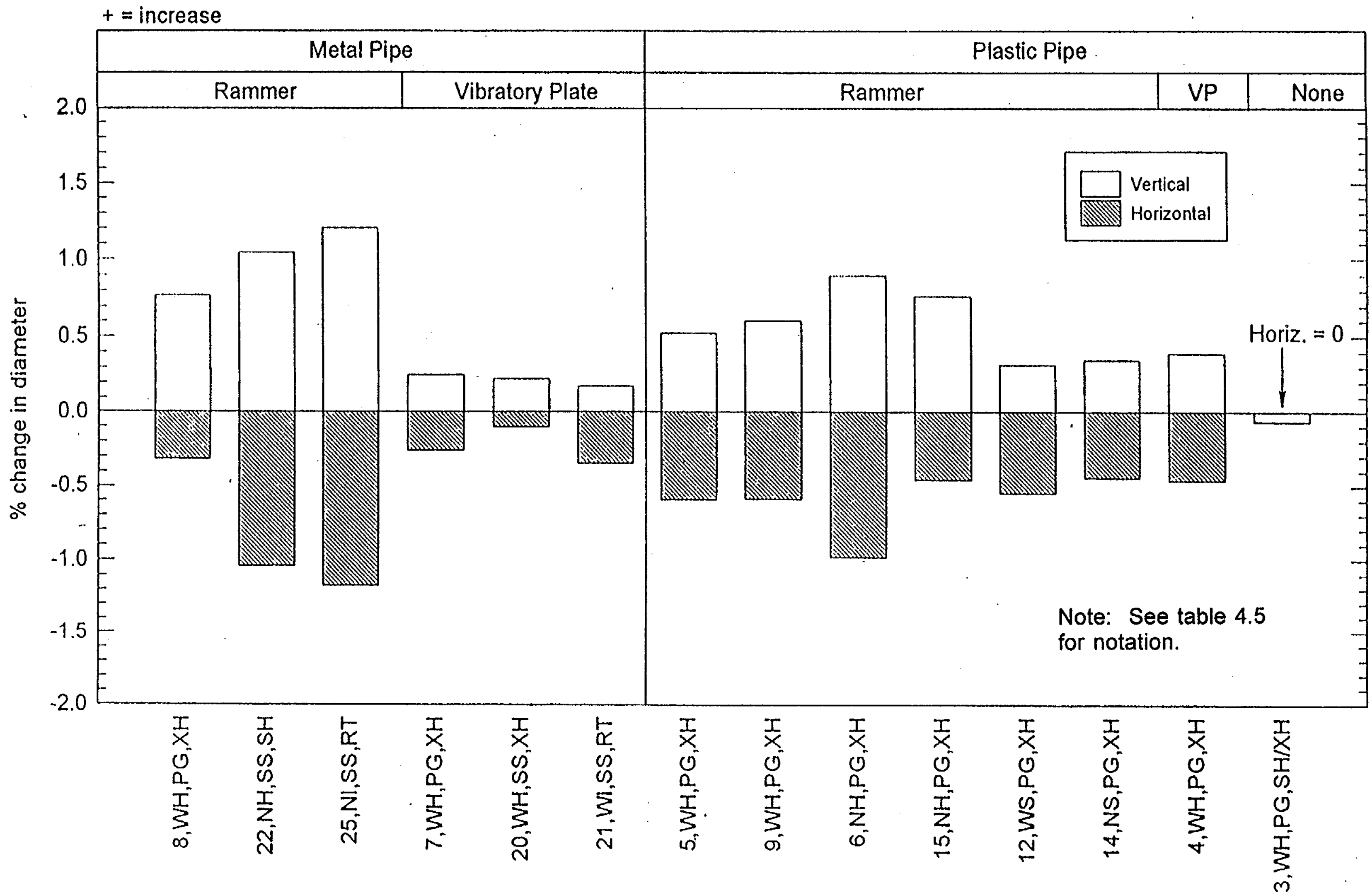



**Figure 4.13 Pipe Deflections, Backfill Placed, and Compacted to the Springline Lift**

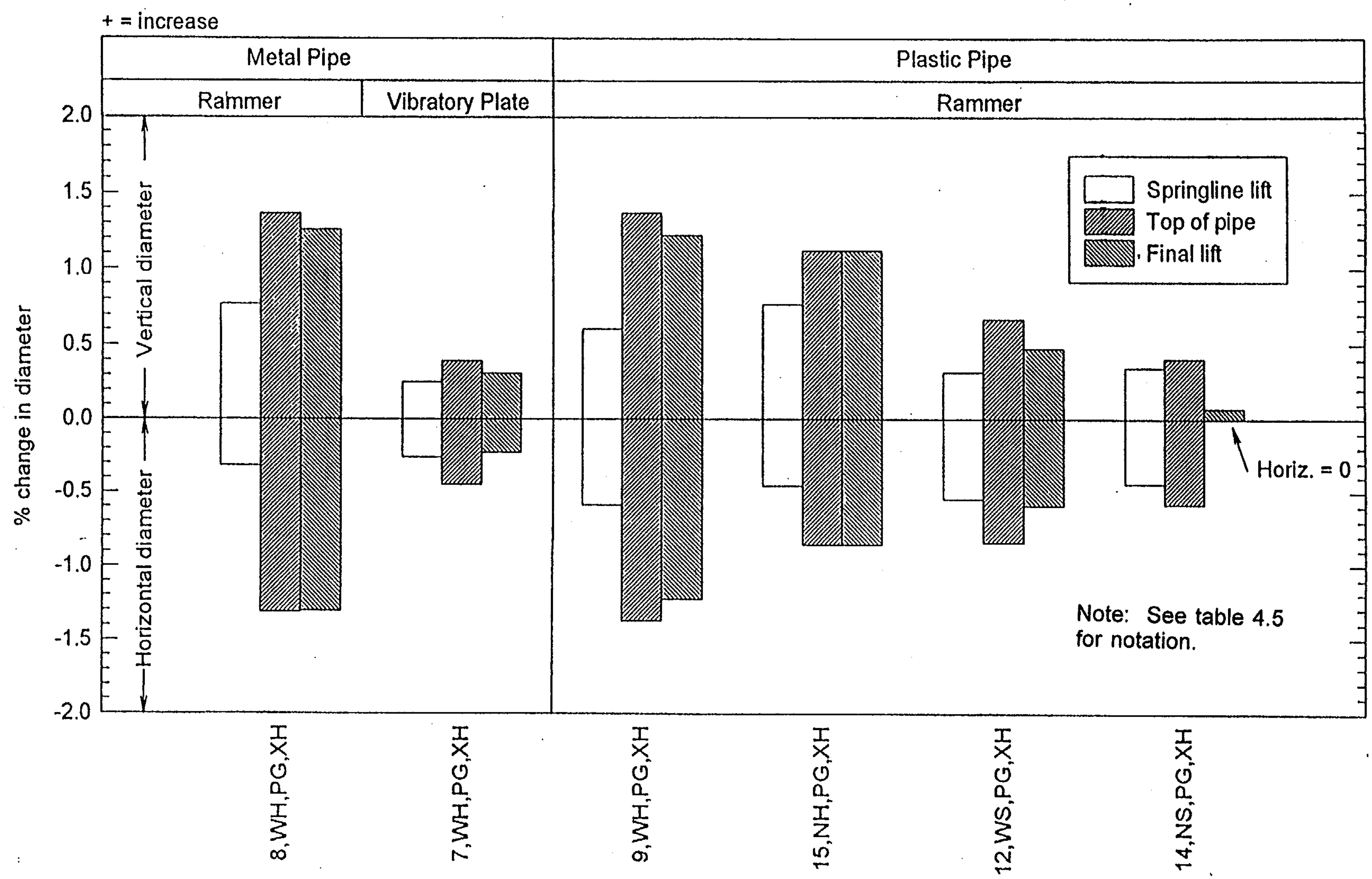



**Figure 4.14 Pipe Deflections, Backfill Placed and Compacted to the Springline Lift, the Top of the Pipe, and the Final Lift**

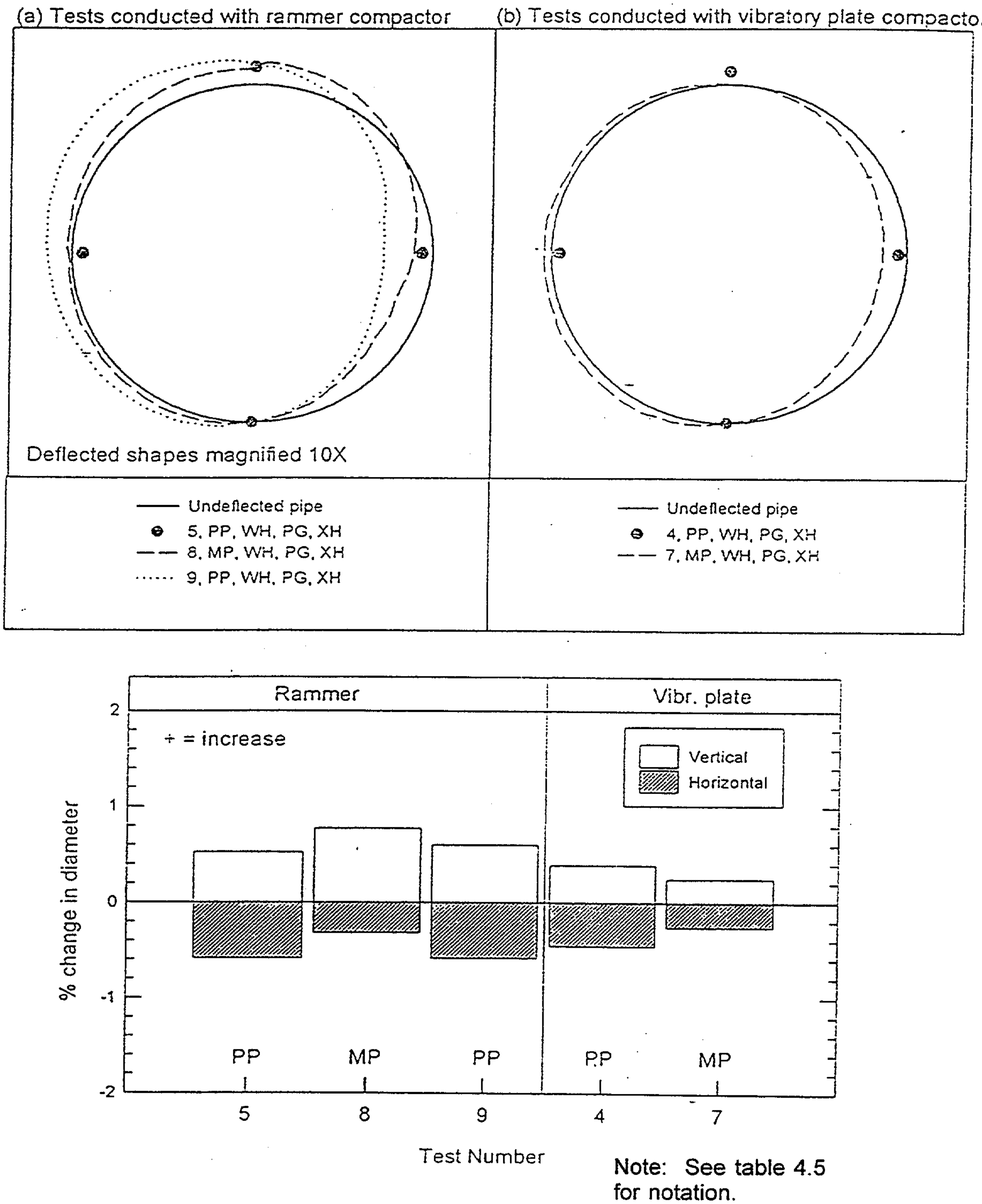
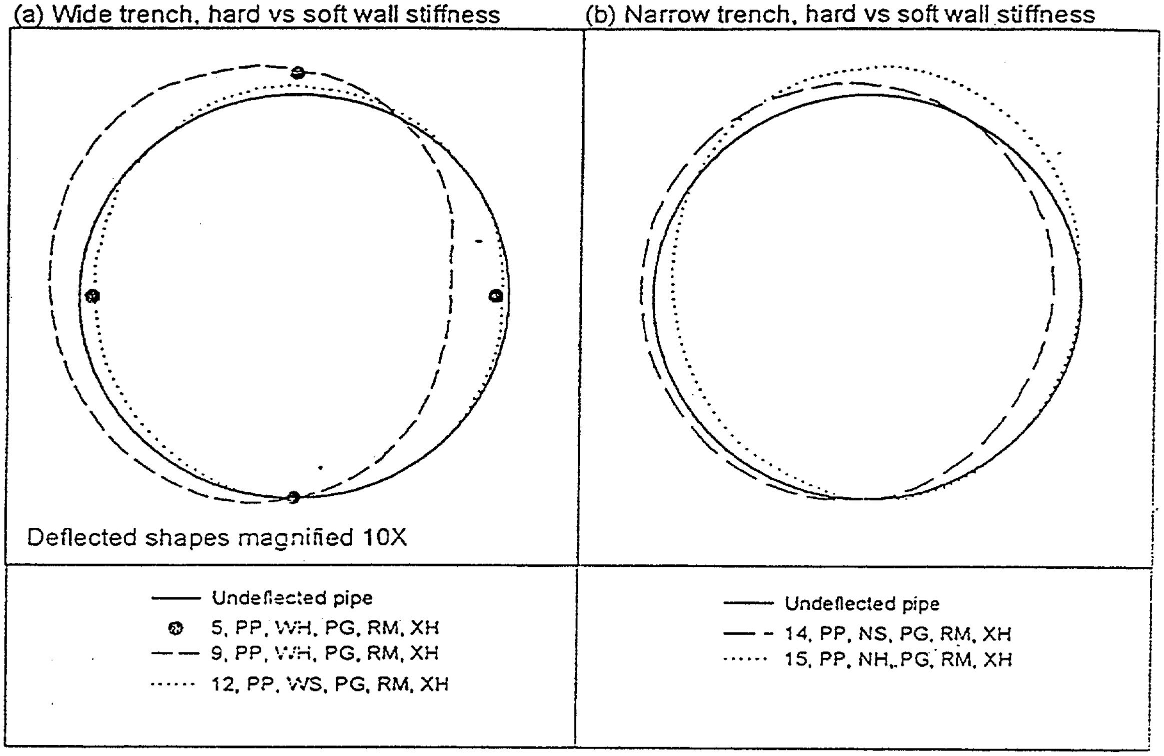



**Figure 4.15 Comparison of Pipe Deflections with Pipe Type and Method of Compaction, Backfill Compacted to the Springline Lift**



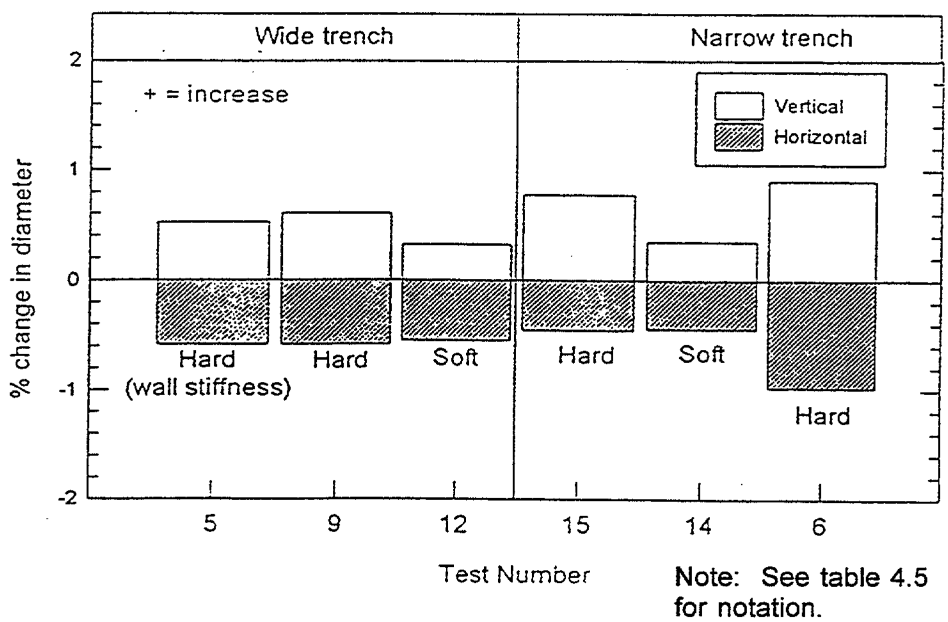




**Figure 4.16 Comparison of Pipe Deflections with Trench Wall Stiffness, Backfill Compacted to the Springline Lift**

### 4.1.6.4 Haunch Zone Pipe Support

Haunch zone pipe support is evaluated by both the pipe-soil interface pressures and the penetration resistance. Interface pressure readings were made for the concrete and metal pipe with both backfill materials while the penetration resistance was only measured for tests backfilled with the silty sand.

The initial invert pressure, i.e., when the pipe is first placed on the bedding, is somewhat random as it is very sensitive to small deviations in the grade along the length of the pipe. Changes in the invert interface pressure during backfilling, however, indicate the change in pipe support that results from compaction and haunching effort below the springline. Fig. 4.17 shows the invert pressure under the concrete pipe for two tests backfilled with pea gravel and compacted with the rammer. Test 10 was conducted with compacted bedding and soft trench walls while test 11 was conducted with the central third of the bedding uncompacted and hard trench walls. Neither test incorporated any effort at compacting material in the haunch zone. Pressures before and after compacting each lift of backfill are shown. Both figures show significant reduction in invert pressure when the first lift, below the springline, is compacted. This confirms observations made in other tests that the rounded pea gravel backfill readily flows under compaction and no specific effort is required to compact it in the haunch zone (see below). However, when backfill is placed above the springline, the pipe with soft trench walls and hard bedding shows large increases in invert pressure while the invert pressure under the pipe with soft bedding and hard trench walls returns to the pretest pressure. Both the trench wall and bedding stiffness are thought to contribute to the reduced invert pressure. Fig. 4.18 shows a similar trend in the invert pressure under the metal pipe.

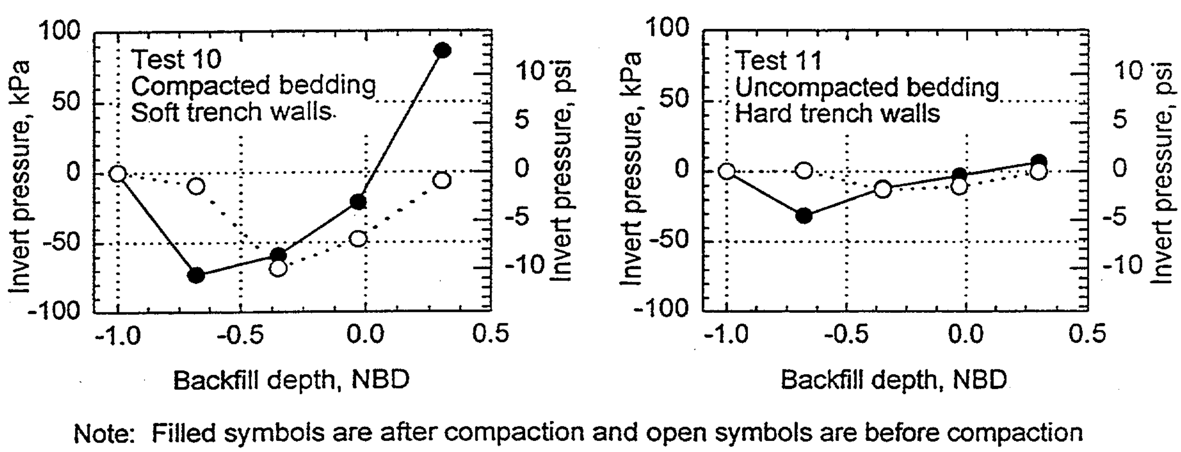



**Figure 4.17 Invert Interface Pressure, Concrete Pipe with Pea Gravel Backfill**

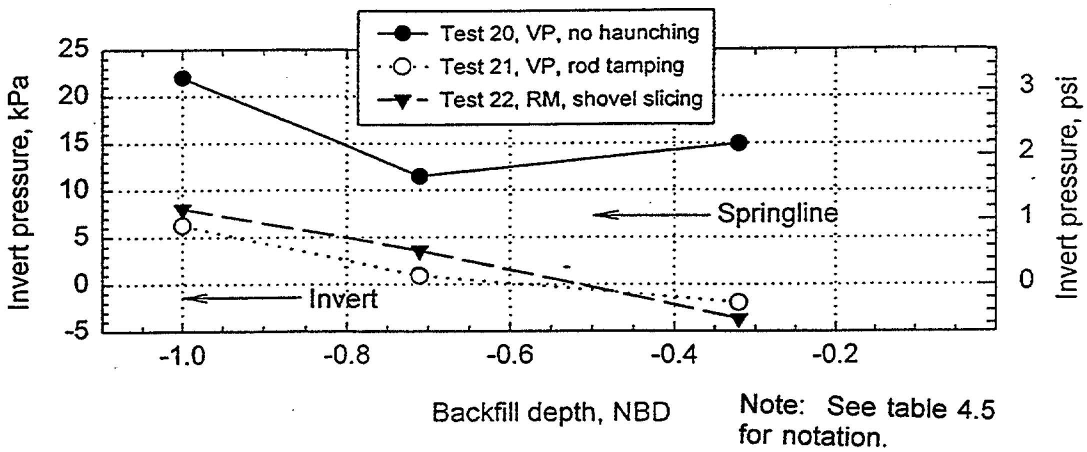



**Figure 4.18 Invert Interface Pressure, Metal Pipe with Silty Sand Backfill**

The radial pressures around the concrete pipe for Tests 23 and 24, backfilled with silty sand and compacted with the rammer when backfill was at a level 150 mm (6 in.) above the springline are presented in Fig. 4.19. For Tests 23 and 24 the backfill was worked into the haunch zone by shovel slicing and rod tamping respectively. These tests show the following:

- Neither type of haunching effort produces significant radial pressure on the pipe at an angle 22.5 degrees from the invert.
- The two types of haunching effort appear to provide equivalent pipe support at angles of 45 degrees and more from the invert.
- Both tests showed essentially zero invert pressure after placing backfill; however, the pressure for both tests was quite low when the pipe was placed, thus, the low pressures are not a result of the haunch effort or compaction.

The interface pressures with backfill compacted up to the springline lift for a metal and concrete pipe under similar installation conditions are presented in fig. 4.20. The figure suggests that the metal pipe develops lower interface pressures at 45 degrees from the invert; this seem consistent with the low weight and stiffness of the metal pipe.

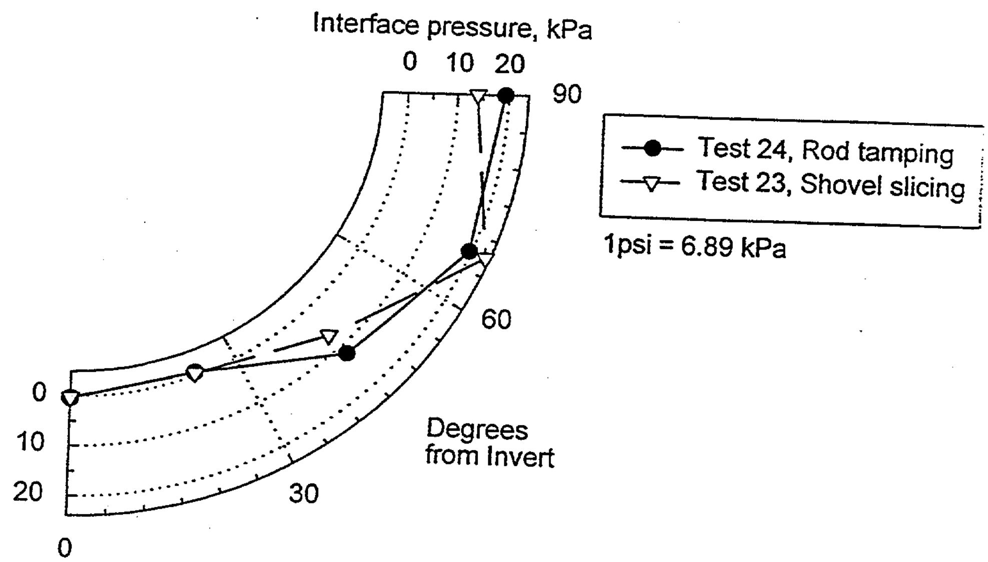



**Figure 4.19 Radial Pressure Against Concrete Pipe**

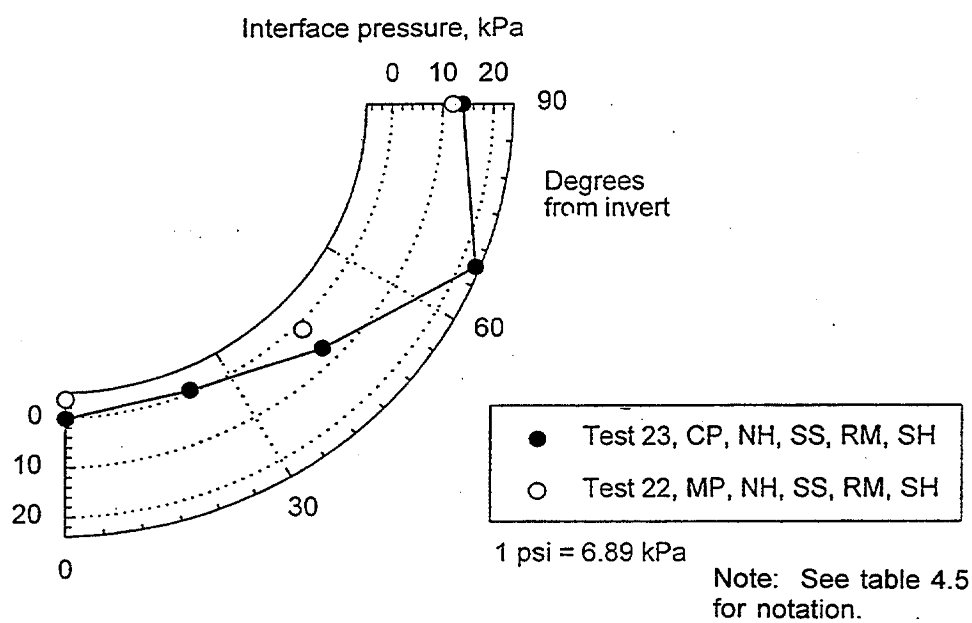



**Figure 4.20 Comparison of Radial Pressure Against Concrete and Metal Pipe**

Proctor penetration tests were conducted only in the silty sand backfill because the penetrometer is used only in fine-grained materials (ASTM D 1558). Penetration tests for tests 20 to 25 were conducted after testing with the pipe removed. Measurements were conducted at the invert and 30 and 60 degrees from the invert. Tests 20 and 21 were measured with a 640 $mm^2$ (1 $in.^2$) tip, and tests 22 through 25 were conducted with a 480 $mm^2$ (0.75 $in.^2$) tip.

The penetration resistance for tests 20 and 21, both conducted with the metal pipe are compared in fig. 4.21. Test 20 was conducted without haunch effort while in test 21 the haunch was compacted using rod tamping. The lower strength of the soil in the haunch region is evident, which is consistent with the interface pressure data. The soil strength under the concrete pipe for tests 23 and 24, which had soft bedding and compacted bedding, respectively are compared in fig. 4.22. The data is consistent with the interface pressures for the same conditions and shows that the soil strength is lower when the backfill is left uncompacted. This is significant because it shows that the soft bedding remains relatively soft even after pipe and backfill are placed.

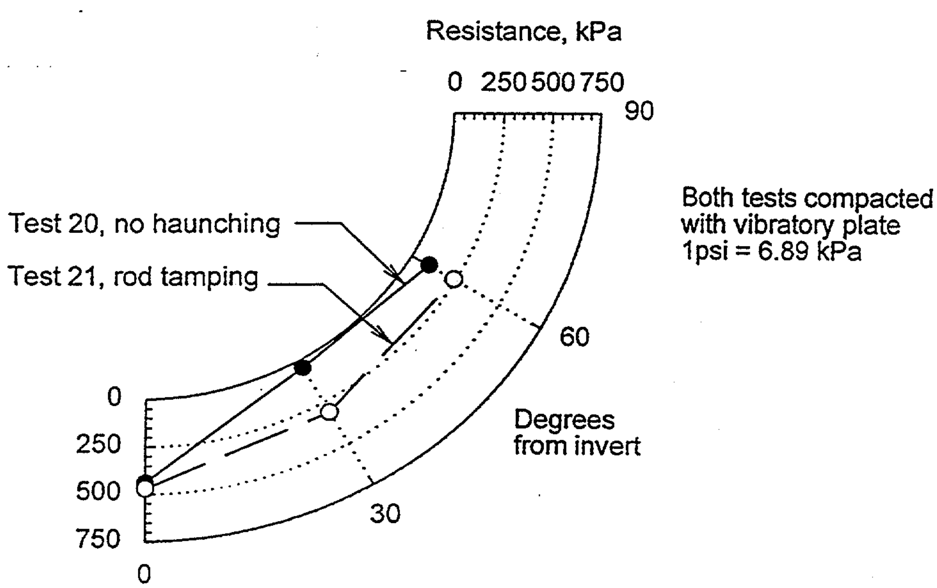



**Figure 4.21 Penetration Resistance of Backfill Under Metal Pipe**

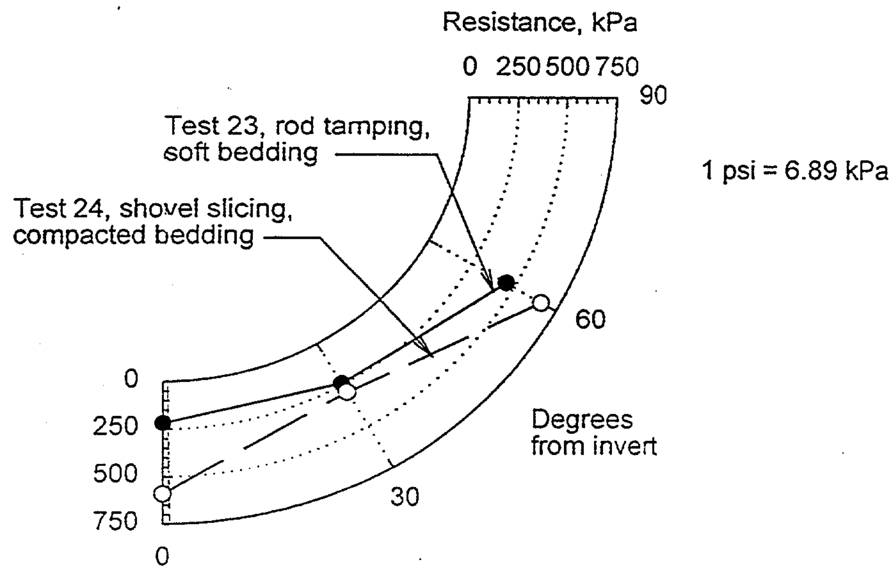



**Figure 4.22 After Test Penetration Resistance of Backfill Under Concrete Pipe**

#### 4.1.6.5 Horizontal Soil Stresses at the Trench Wall

Horizontal backfill stresses were measured on both sides of the trench at the pipe springline and haunch elevations. Horizontal soil stresses when the backfill is placed and compacted to the springline lift for specific test variables are presented in figs. 4.23 to 4.25. The horizontal stresses at the haunch elevation are greater than the stresses at the springline elevation, which is consistent with the depth of fill. The horizontal soil stresses are generally lower for the concrete pipe than for the plastic pipe, and the stresses were higher with the hard and intermediate trench wall stiffness than with the soft wall stiffness. In both the wide and narrow trench conditions, the horizontal soil stresses were, on average, four times greater with the hard wall. The silty sand resulted in higher horizontal stresses than the pea gravel. Horizontal stresses were, on average, 35 percent higher with the silty sand material.

The horizontal stresses at the springline and haunch level for tests where backfill was brought over the top of the pipe are shown in fig. 4.26. This figure also shows the geostatic lateral pressure, assuming a $K_0$ value of 0.4, when the backfill was at the final elevation. This figure demonstrates the significant loss of lateral support when the trench walls are soft.

Trench wall displacement measurements show that large compression occurred in the soft trench wall, on the order of 30 to 50 mm. Compression of the intermediate trench wall was on the order of 0.5 mm to 1.0 mm (0.02 in. to 0.04 in.).

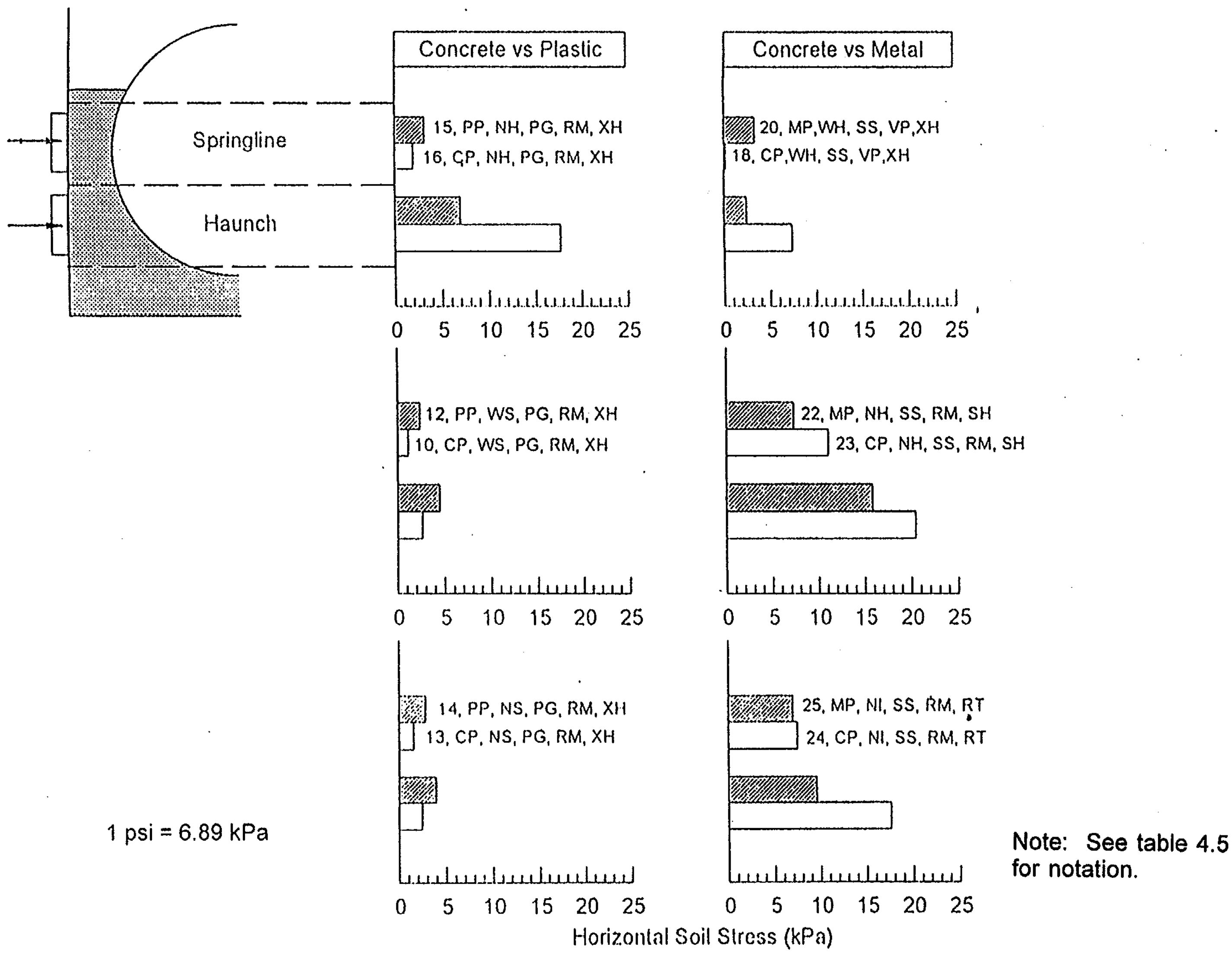



**Figure 4.23 Comparison of Horizontal Soil Stresses at the Trench Wall Due to Pipe Type, Backfill Placed and Compacted to the Springline Lift**

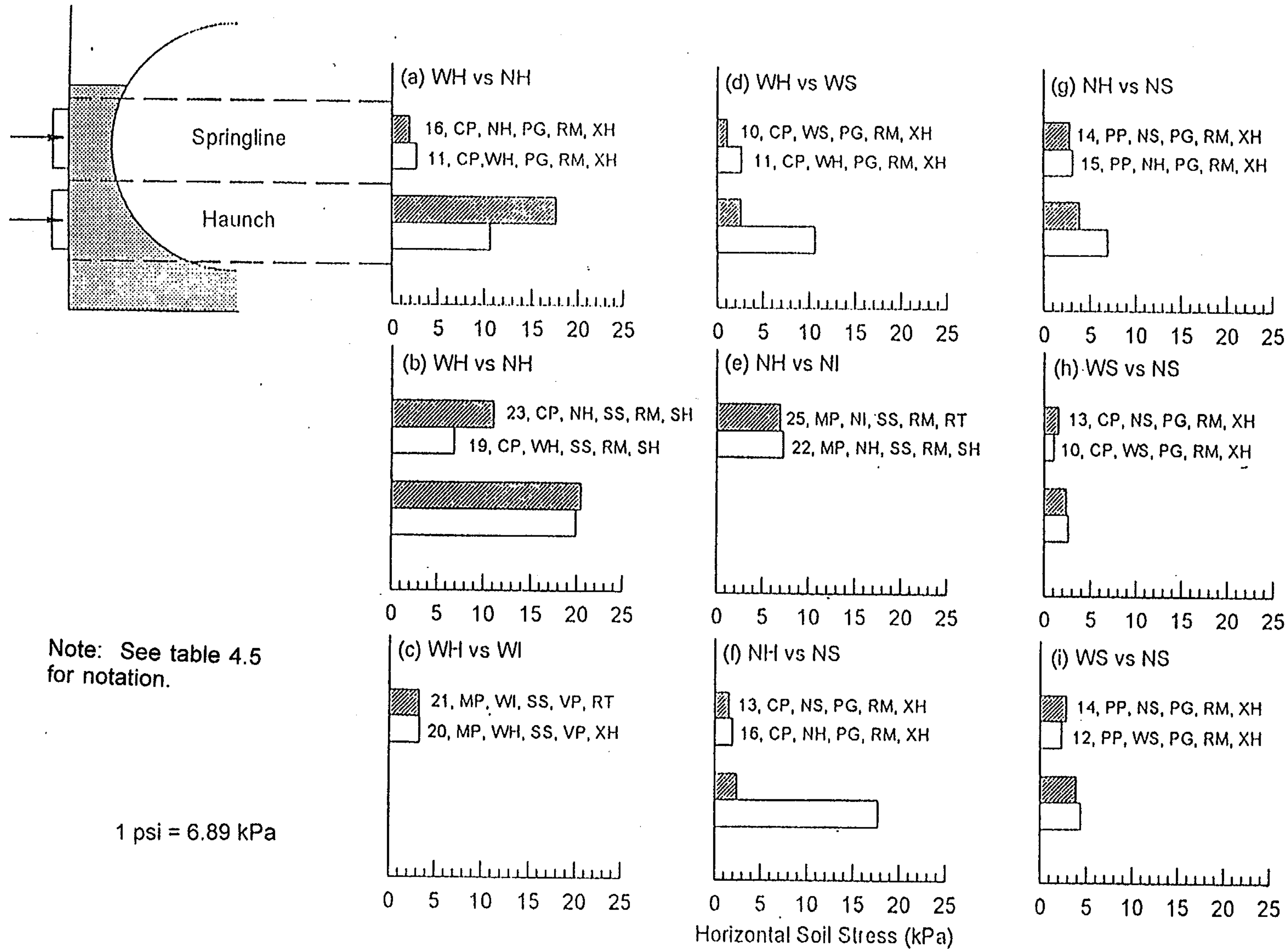



**Figure 4.24 Comparison of Horizontal Soil Stresses at the Trench Wall Due toTrench Condition, Backfill Placed and Compacted to the Springline Lift**

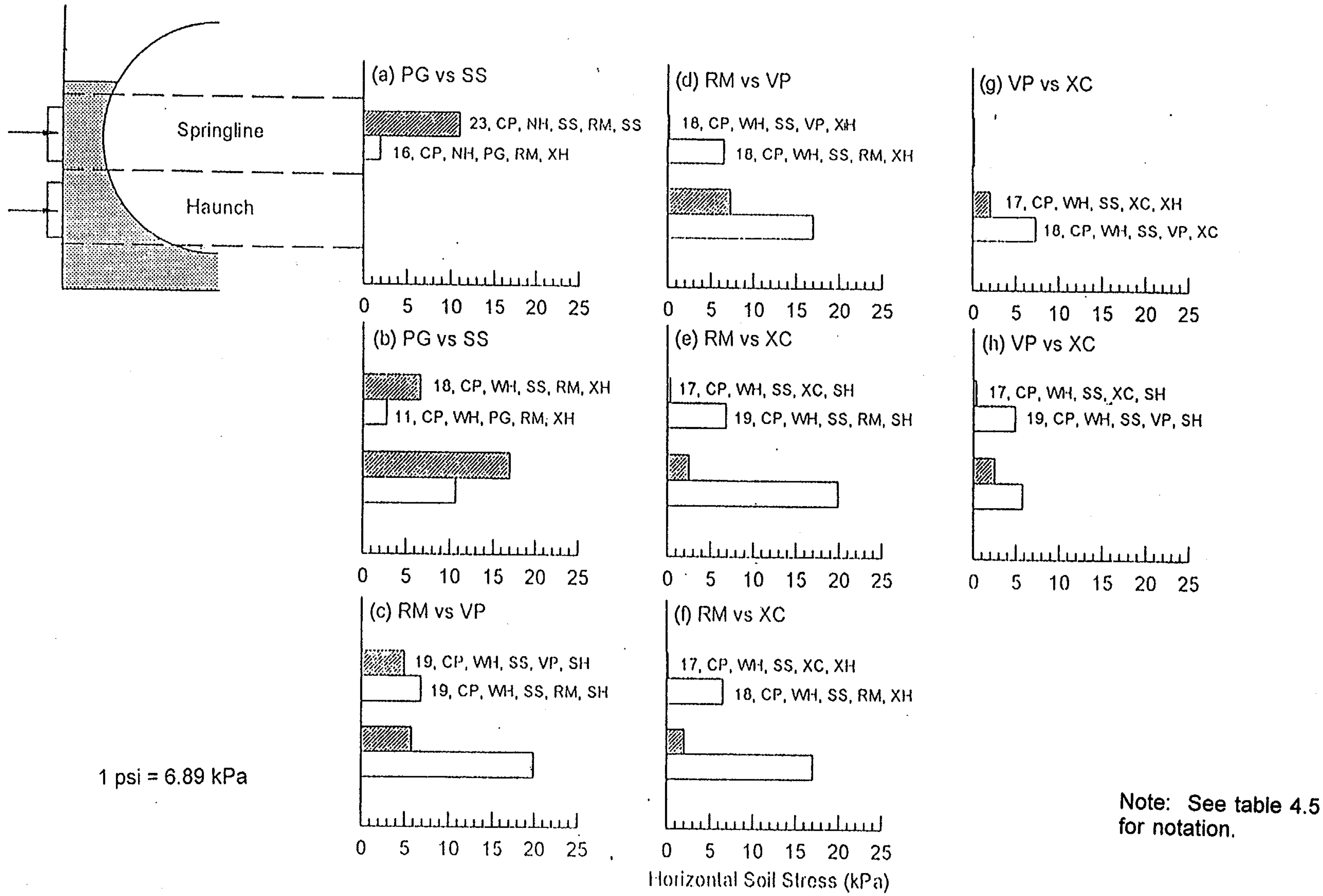
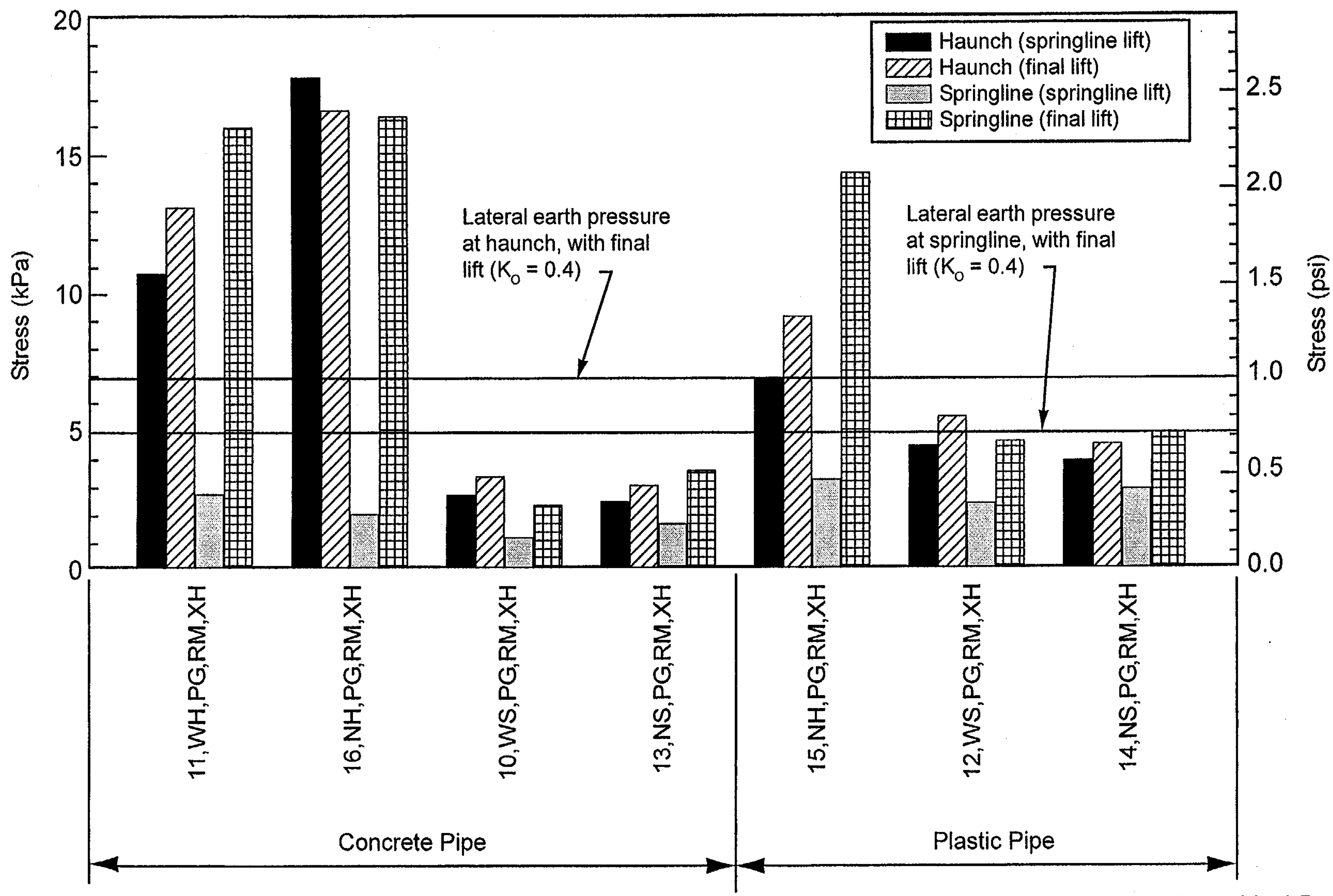



**Figure 4.25 Comparison of Horizontal Soil Stresses at the Trench Wall Due to Backfill Material and Method of Compaction, Backfill Placed and Compacted to the Springline Lift**




Note: See table 4.5 for notation.

**Figure 4.26 Horizontal Soil Stresses at the Trench Wall, Backfill Placed and Compacted to the Springline and the Final Lift**

#### 4.1.6.6 Pipe Strains

Strains were measured for only three tests conducted with the plastic pipe and the results are presented as strain versus normalized depth of fill in fig. 4.27. Gages were located at the springline and invert both on the inner and outer walls of the pipe. Positive readings indicate tension. Note that for all of these tests the backfill was compacted with the rammer. The circumferential strains (fig. 4.27(a) and (b)) are consistent with the deflection and other data collected, i.e., upward peaking of the pipe during compaction but reduced in magnitude when the trench walls are soft. The outside wall strains were larger than strains in the inside wall, which is consistent with the location of the centroidal axis. The longitudinal strains are of opposite sign from the circumferential strains at the same location.

Plots of strain versus deflection at every depth of fill, with the best fit regression curve and correlation coefficient, r, and slope, m, are presented in fig. 4.28. The data are relatively linear, with coefficients of correlation always greater than 0.74 except for the longitudinal strain at the springline. The best fit curves generally pass through the origin of the plot. The ratios of the slopes, presented in table 4.7, indicate the relative magnitude of the longitudinal strain compared to the circumferential strain. The ratio is higher at the invert than at the springline.

**Table 4.7**

**Strain Versus Deflection in Plastic Pipe**

| Location | Circumferential strain | Longitudinal strain | Ratio: long./circumf. |
|---|---|---|---|
| | (% strain/%defl.) | (% strain/%defl.) | |
| Springline, inside | 0.16 | -0.07 | -0.44 |
| Springline, outside | -0.31 | 0.14 | -0.45 |
| Invert, inside | -0.18 | 0.11 | -0.61 |
| Invert, outside | 0.21 | -0.14 | -0.67 |

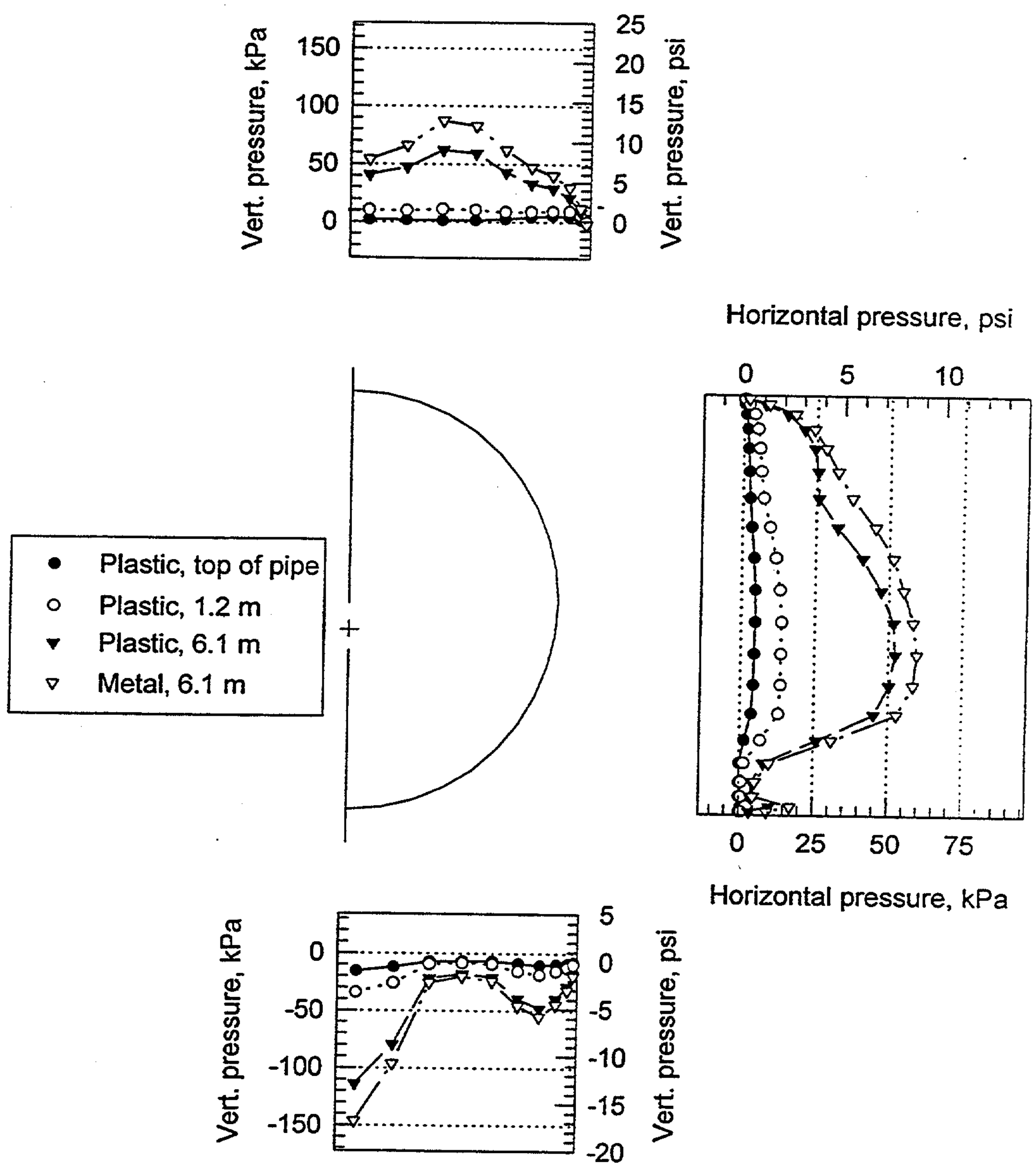



**Figure 5.11 Vertical and Horizontal Pressures on Plastic and Metal Pipe, CANDE Analysis, Test 5 – Rammer Compaction, Soft Bedding, No Haunching, Sandy Silt Backfill**

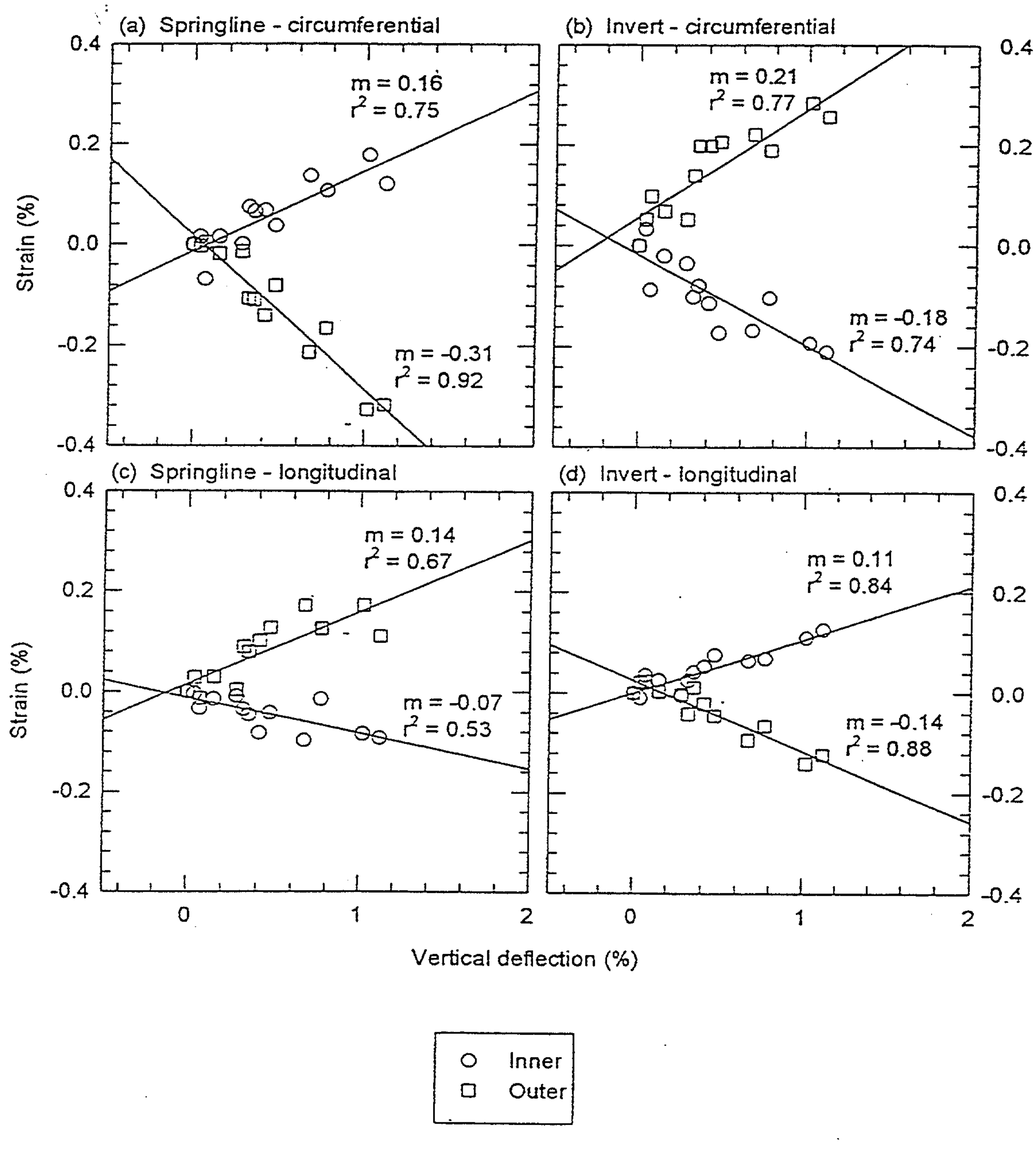



**Figure 4.28 Strain Correlated with Deflection After Compaction of Backfill**

## 4.2 Field Tests

Full-scale field tests were conducted to gather data on the stresses, strains, and deformations in pipe and the surrounding soil embedment as the pipe-soil system is being constructed. The test program was developed to provide information that could improve our understanding of the response of a pipe and the surrounding soil to installation variables. The test program has been reported in detail in Webb (1995). Tables and figures of all of the raw data are reported in Webb et al.(1995) and Zoladz et al. (1995).

A total of 14 tests were conducted. Each test included a reinforced concrete, corrugated or profile wall polyethylene, and a corrugated steel pipe. Tests variables for each test are described in table 4.8. Because of the number of variables involved, it was not possible to test every possible combination of parameters. The specific combinations selected were based on the judgement of the research team.

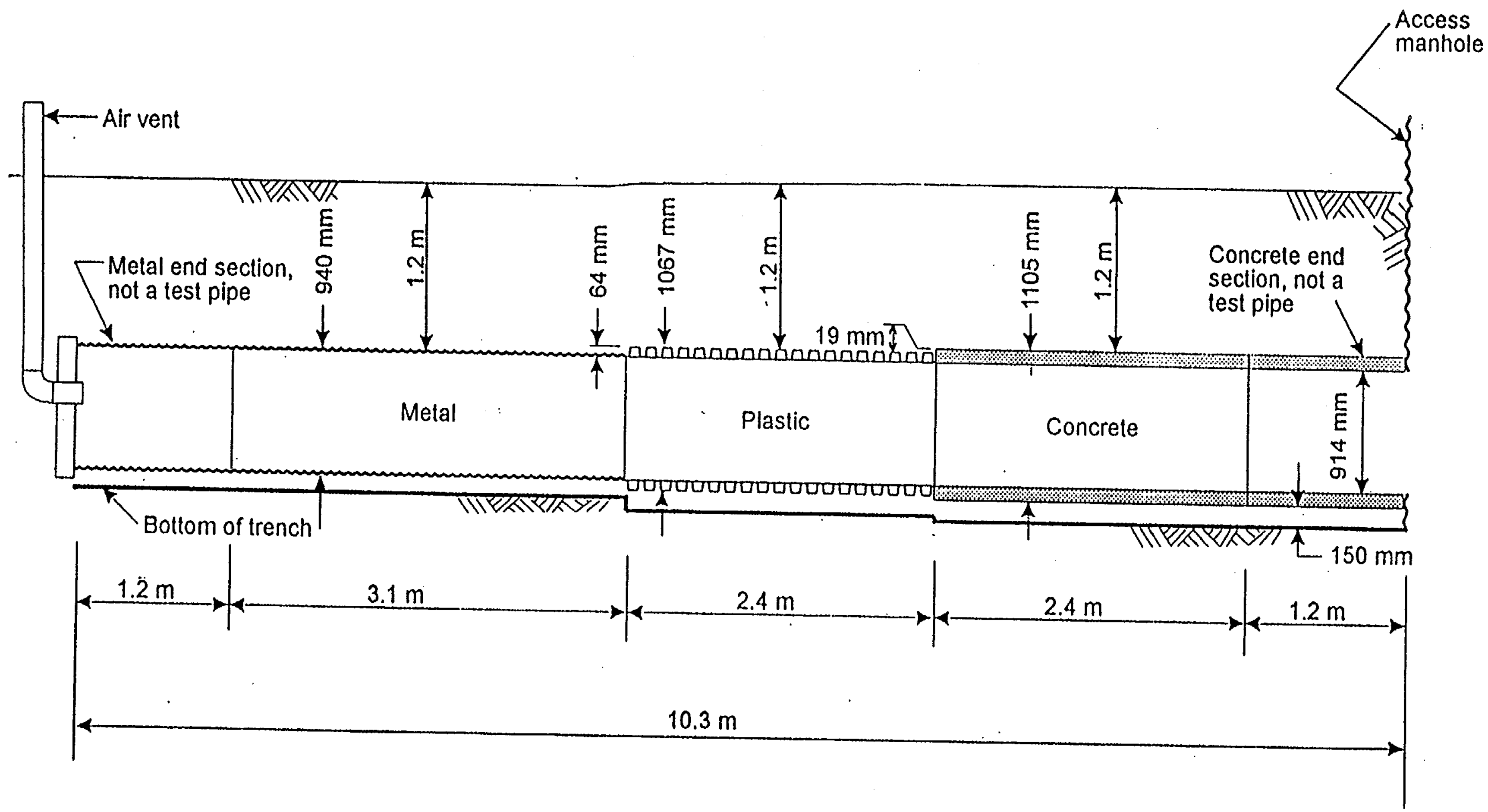
The general configuration for each test consisted of one length each of concrete, plastic, and metal pipe installed end to end as shown in fig. 4.29 for the 900 mm (36 in.) diameter pipe. The configuration for the 1,500 mm (60 in.) diameter pipe was similar. All the pipes were backfilled to a depth of 1.2 m (4 ft) over the top of the pipe.

More detailed information on pipe, backfill, test sites, and other variables is provided in the following sections.

**Table 4.8**
**Summary of Variables for Field Tests**

| Test No. | Trench Width (1) | In situ soil | Pipe diameter mm (in.) | Backfill material | Sidefill compaction | Haunch (2) | Bedding compaction (3) |
|---|---|---|---|---|---|---|---|
| 1 | N | Sand | 900 (36) | Stone | Rammer | SS | Fully compacted |
| 2 | N | Sand | 900 (36) | Stone | None | N | Fully compacted |
| 3 | W | Sand | 900 (36) | Stone | Rammer | SS | Sides compacted |
| 4 | W | Sand | 900 (36) | Stone | | N | Sides compacted |
| 5 | N | Sand | 900 (36) | Silty sand | None | N | Fully compacted |
| 6 | N | Sand | 900 (36) | Silty sand | Rammer | SS | Fully compacted |
| 7 | W | Sand | 900 (36) | Silty sand | Vibr. plate | N | Sides compacted |
| 8 | W | Sand | 900 (36) | Silty sand | Rammer | SS | Sides compacted |
| 9 | N | Clay | 900 (36) | Stone | Rammer | SS | Fully compacted |
| 10 | N | Clay | 900 (36) | CLSM | Rammer | -- | Fully compacted |
| 11 | W | Clay | 900 (36) | Stone | Vibr. plate | N | Sides compacted |
| 12 | N | Clay | 1,500 (60) | Stone | None | RT | Fully compacted |
| 13 | W | Clay | 1,500 (60) | Stone | Vibr. plate | RT | Sides compacted |
| 14 | I | Clay | 1,500 (60) | Silty sand | Vibr. plate | RT | Sides compacted |

Notes: 1. N = narrow (O.D. +0.6 m), W = wide (O.D. plus 1.8 m), and I = intermediate (O.D. plus 0.9 m).
2. SS = shovel slicing, RT = rod tamping and N = none.
3. Bedding was compacted with the vibratory plate. Fully compacted means the bedding was compacted over the full trench width. Sides compacted means that a strip directly under the pipe, one third of the pipe outside diameter in width, was left uncompacted.




Note: Dimensions shown for 914 mm (36 in.) diameter pipe; some dimensions change for the 1524 mm (60 in.) diameter pipe.
1 in. = 25.4 mm
1 ft = 0.305 m

**Figure 4.29 Schematic of Layout of Test Trenches for 900 mm Diameter Pipe**

### 4.2.1 Test Pipe

Eleven tests were conducted with 900 mm (36 in.) nominal inside diameter pipe, and three tests were conducted with 1,500 mm (60 in.) nominal inside diameter pipe. The 900 mm diameter plastic pipe had a corrugated pipe wall with a liner to provide a smooth inside surface. The 1,500 mm plastic pipe had a smooth pipe wall with a spiral rib on the outside. The test pipe are referred to herein as the concrete, metal, and plastic pipes, respectively. Pipe were supplied with no joints, allowing them to be laid end to end in the test trenches. These pipes were selected to provide a range of pipe bending and hoop stiffnesses that is typical in actual culvert applications.

The geometric, material, and stiffness parameters of the test pipe are summarized in table 4.9. In this table, the nominal short term modulus of the polyethylene is reported and used to calculate the pipe stiffnesses. Depending on the duration of an applied load, other values of the modulus may be appropriate; however, since the tests discussed in this paper are all of relatively short duration, the short-term modulus was deemed most appropriate. The pipe stiffnesses are calculated values, rather than test values. Test values for plastic and metal pipes are often lower than the calculated values.

**Table 4.9**
**Summary of Properties of Test Pipe**

| Pipe type | Diameter mm | E GPa | A $mm^2/mm$ | I $mm^4/mm$ | $PS_H$ $kN/m^2$ | $PS_B$ kN/m/m |
|---|---|---|---|---|---|---|
| Concrete | 900 | 25 | 119 | 140,000 | $5,800 \times 10^3$ | 170,000 |
| | 1,500 | | 169 | 402,000 | $5,000 \times 10^3$ | 111,000 |
| Plastic | 900 corrugated | 0.8 | 10.2 | 8,470 | $16 \times 10^3$ | 390 |
| | 1,500 profile | | 11.3 | 3,180 | $11 \times 10^3$ | 36 |
| Metal | 900 | 205 | 1.64 | 31 | $720 \times 10^3$ | 410 |
| | 1,500 | | 1.88 | 142 | $500 \times 10^3$ | 420 |

1 mm = .039 in., 1 GPa = $145 \times 10^3$ psi, 1 $kN/m^2$ = 0.15 psi

Table 4.9 shows that the concrete pipe has high hoop and bending stiffness relative to both the metal and plastic pipe, while the plastic pipe has low flexural and hoop stiffnesses. However, the metal pipe has a low bending stiffness, which is consistent with its traditional treatment as a flexible pipe but an intermediate hoop stiffness. Thus, each of the three pipes represents a different regime of pipe stiffnesses. Low hoop stiffness has been shown to cause significant reductions in load on buried pipe (Hashash and Selig, 1990).

### 4.2.2 Test Sites

Tests were conducted at two sites. At the first site, called here the "sand" site, the soils were glacial deposits of coarse to medium sand (SP, SW-SM). Samples of these soils were incorporated into the backfill test program reported in chapter 3 as Soils Nos. 11 and 12. In its natural condition, this sand was compact and partially cemented, providing a stiff stable material to excavate trenches in and compact soil against. The ground water table was near the bottom of the excavations for some of the tests and pumps were used to keep the excavation reasonably dry. Seepage from the trench walls also affected some of the tests.

The second site consisted principally of a sedimentary varved clay deposit (CL). Samples of these soils were incorporated into the backfill test program reported in chapter 3 as Soils No. 9 and 10. This formation is generally quite soft and was selected to represent a poor in situ soil condition, unfortunately the specific area selected proved to be stiffer than anticipated. Penetrometer readings suggest unconfined compression strength values between 190 kPa and 380 kPa (2 tsf and 4 tsf), with values as low as 100 kPa (1 tsf) in some areas. Some water seeped into the trenches during the tests; however, the rate was low enough that positive action to control the water was not required.

### 4.2.3 Backfill

Thirteen of the fourteen tests were completed with either of two soil backfill materials, in a 19 mm (3/4 in.), broadly graded crushed stone, called stone herein and characterized as Soil No. 3 in chapter 3, and a poorly graded silty sand characterized as Soil No. 6 in chapter 3.

One test was backfilled to the pipe springline with CLSM. The batch design of the flowable fill, shown in table 4.10, was selected based on the material study reported in chapter 3. The target strength for the mix was 690 kPa (100 psi) at 28 days. The material was delivered in two batches, and although the ready mix supplier reported that both batches were identical, the strengths and stiffnesses of the two batches varied significantly, as shown in table 4.11. This backfill above the springline was the in situ clay material which is discussed in a subsequent section.

**Table 4.10**
**CLSM Backfill Mix Design**

| Material | Mass<br>$kg/m^3$ ($lb/yd^3$) |
|---|---|
| Concrete sand | 1606 (2707) |
| Cement | 46 (78) |
| Class F fly ash | 247 (416) |
| Water | 274 (462) |

**Table 4.11**
**CLSM Strength Test Results**

| Batch No. | Strength, kPa (psi) | | Modulus of elasticity, MPa (psi) | |
|---|---|---|---|---|
| | 7 day | 28 day | 7 day | 28 day |
| 1 | 420 (61) | 779 (113) | 165 (24,000) | 234 (34,000) |
| 2 | 248 (36) | 434 (63) | 70 (10,000) | 145 (21,000) |

### 4.2.4 Instrumentation

Extensive instrumentation was used to monitor the behavior of the test pipe and surrounding soil as the backfill was placed and compacted at the sides of the pipe. The instrumentation was largely the same as used in the laboratory tests and described in detail in McGrath and Selig (1996). The instruments included a profilometer to monitor pipe deflections and overall changes in the pipe shape, strain gages mounted on the metal and plastic pipe, interface pressure cells on the concrete and metal pipe, and earth pressure cells to monitor horizontal soil stresses at the trench wall-backfill interface and vertical soil stresses in a plane 150 mm (6 in.) over the top of the pipe. In addition, inductance coil soil strain gages that were not used in the laboratory tests were installed to monitor horizontal soil displacements between the springline of the pipe and the trench wall. Instrument layouts for each type of pipe are shown in figures 4.30 to 4.35.

Strain gages were mounted on the springlines, crown, and invert of the plastic and metal pipes. At each position gages were mounted on the inside and outside surfaces in both the circumferential and longitudinal directions.

Soil stresses were monitored with 230 mm (9 in.) diameter, fluid filled, earth pressure cells with vibrating wire transducers. The cells mounted in the trench wall at the springline (see figures 4.30, 4.32, and 4.34) had heavy backplates to minimize the effect of non-uniform support against the trench wall. The cells over the top of the pipe were sensitive to pressure on both faces.

In addition to the above instruments, standard survey equipment was used to monitor pipe and backfill elevations. Observations were used to supplement measurements whenever appropriate. Most instruments were read electronically using a computerized data acquisition system.

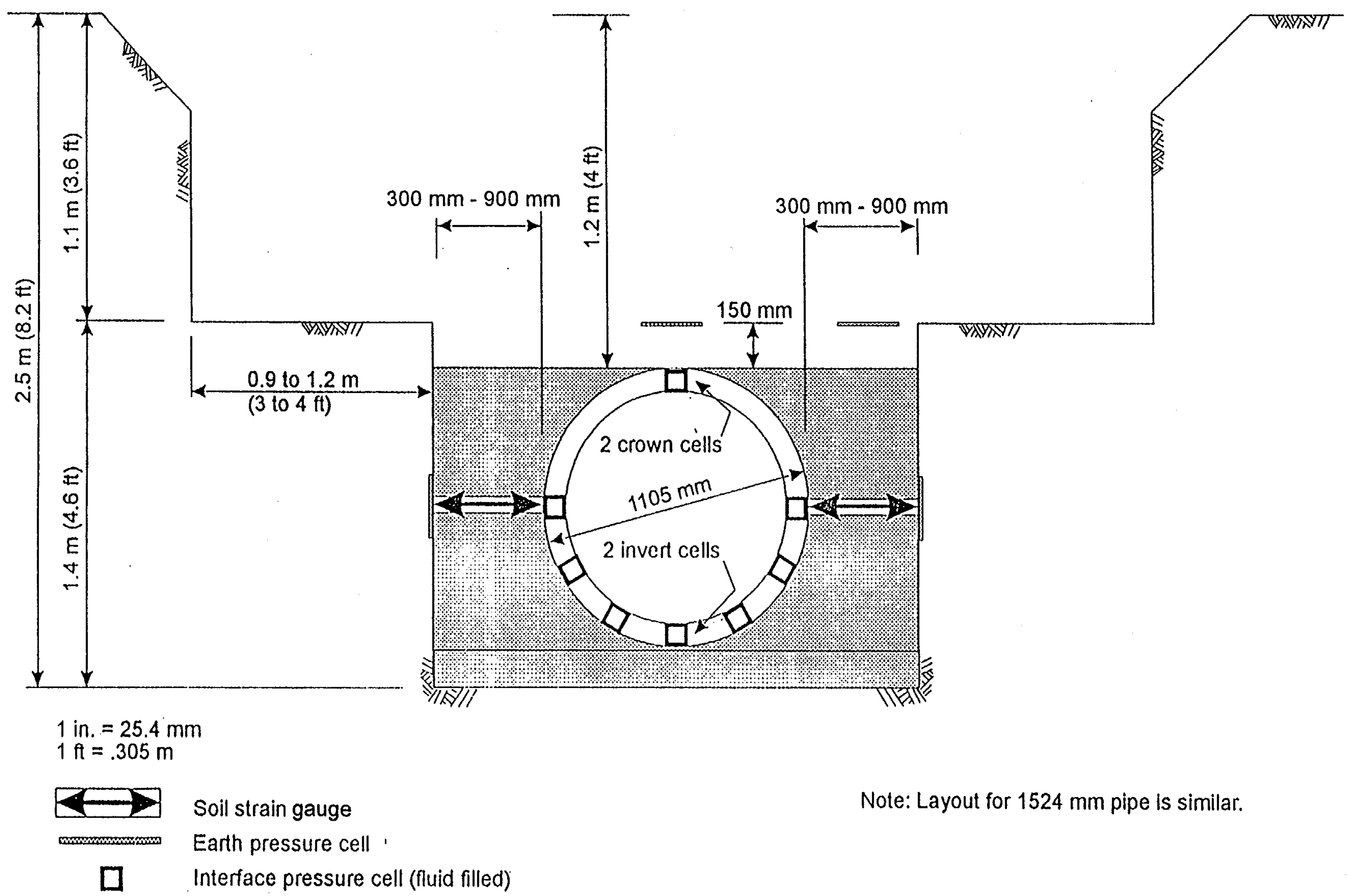



**Figure 4.30 Cross-Section of Concrete Pipe in Trench with Instrumentation**

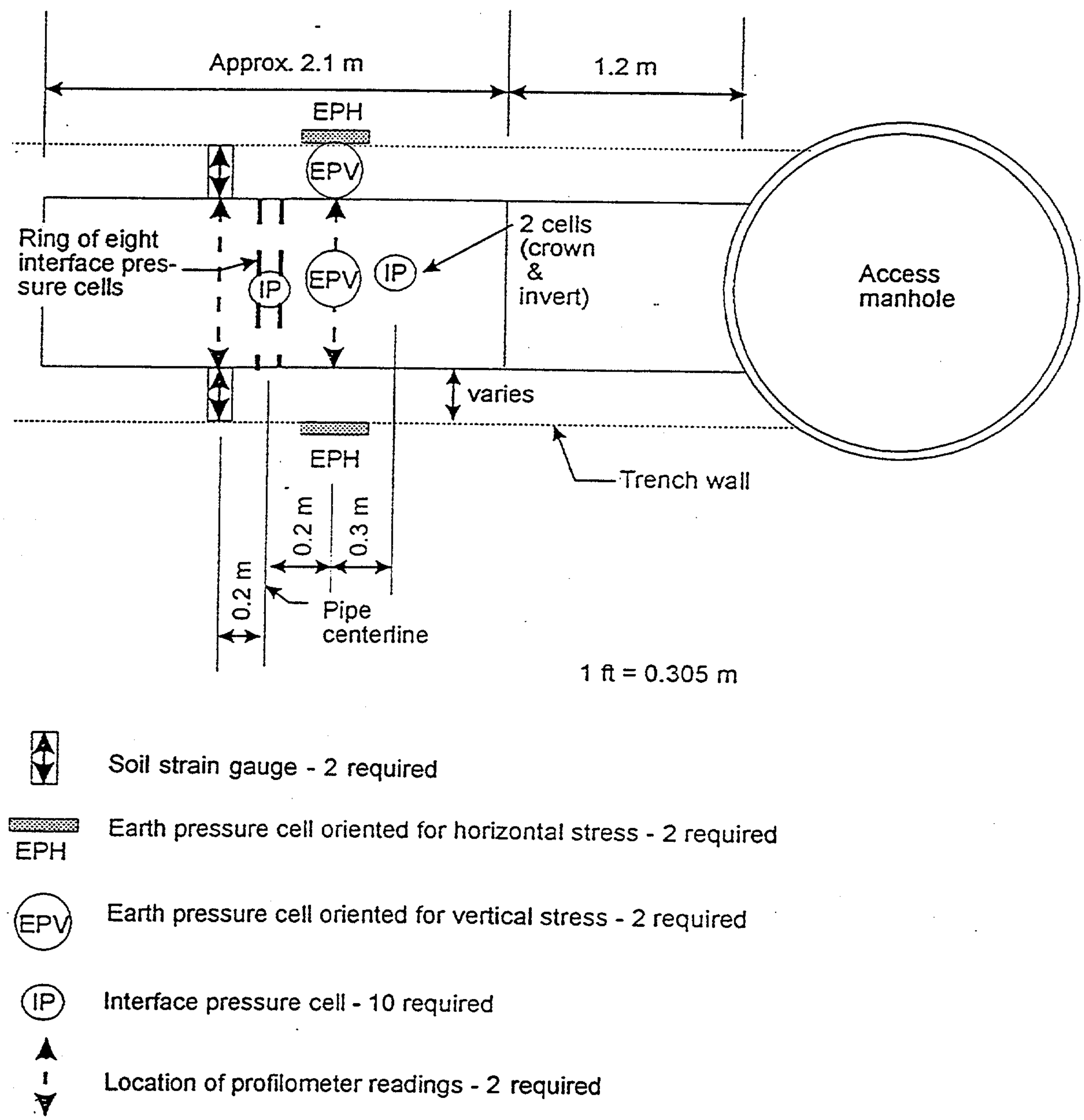



**Figure 4.31 Longitudinal Instrumentation Layout for the Concrete Pipe**

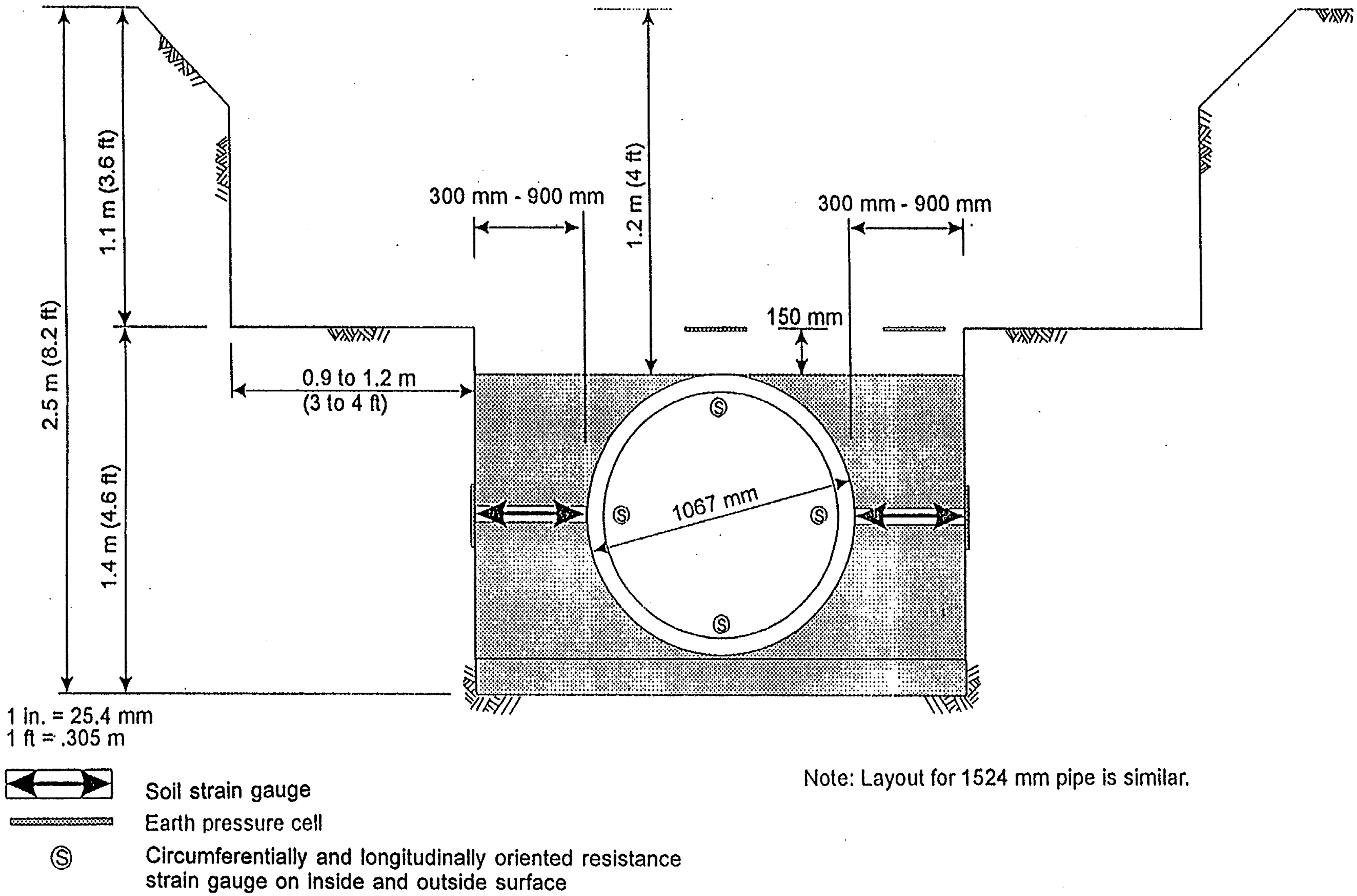



**Figure 4.32 Cross-Section of Plastic Pipe in Trench with Instrumentation**

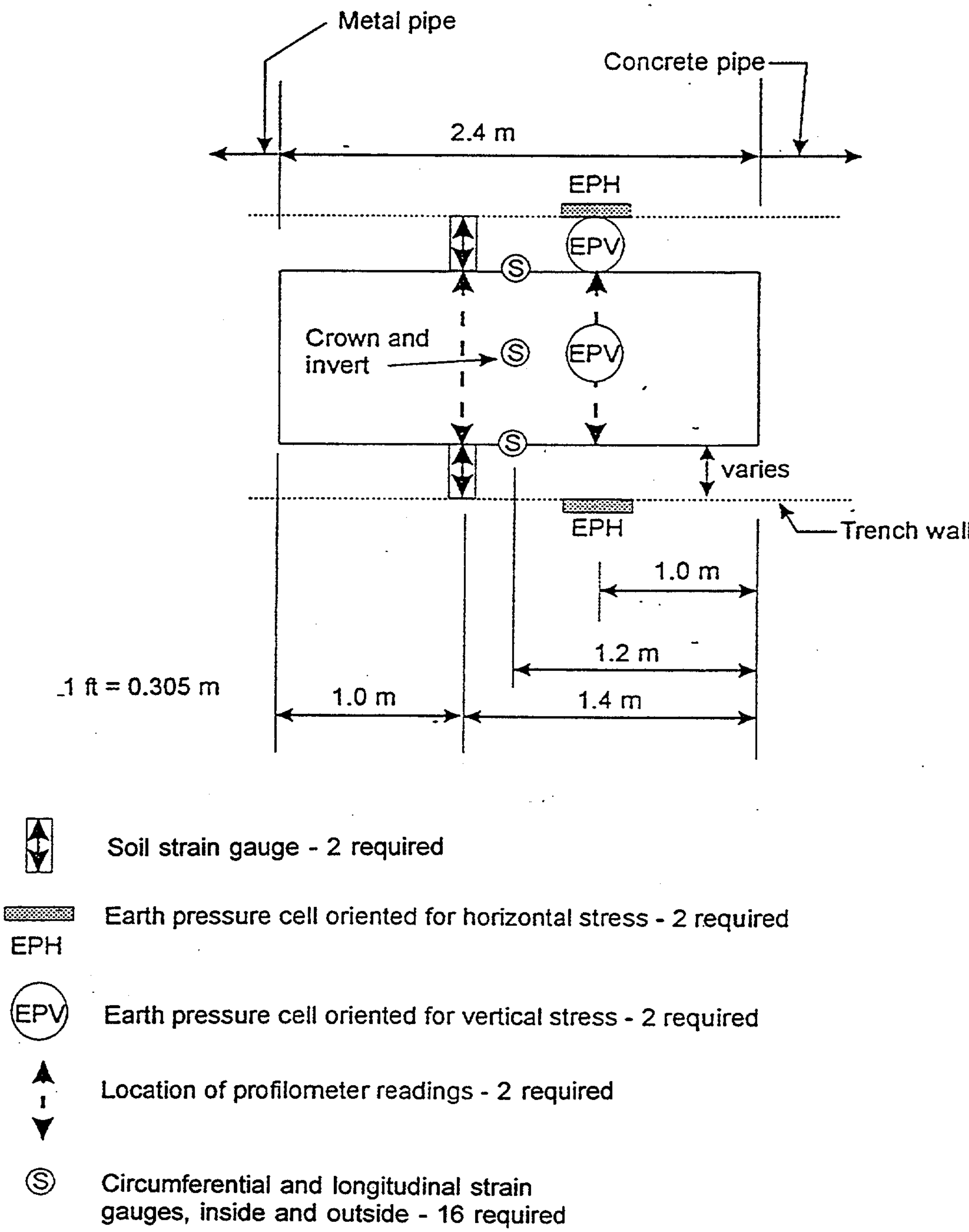



**Figure 4.33 Longitudinal Instrumentation Layout for the Plastic Pipe**

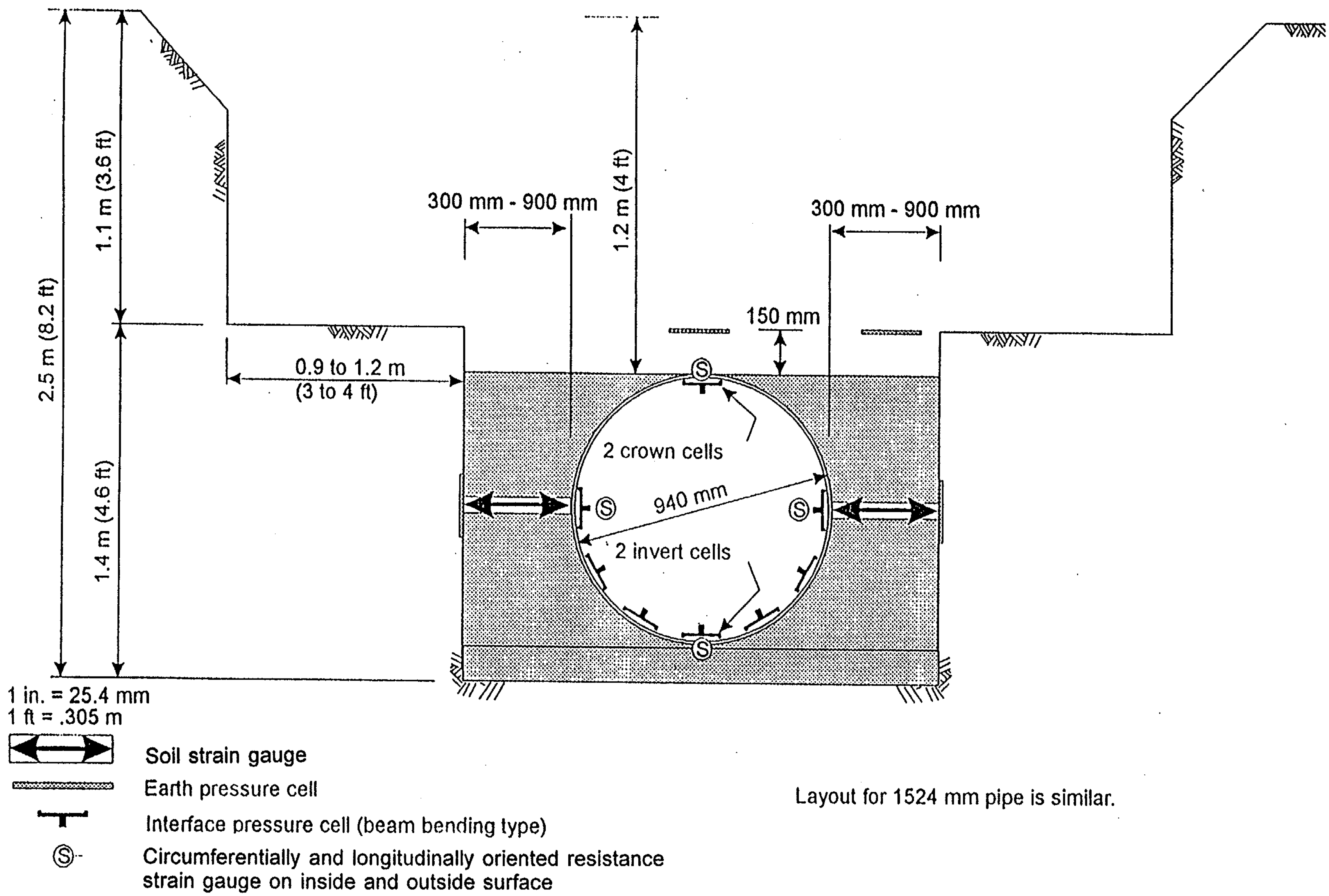



**Figure 4.34 Cross-Section of Metal Pipe in Trench with Instrumentation**

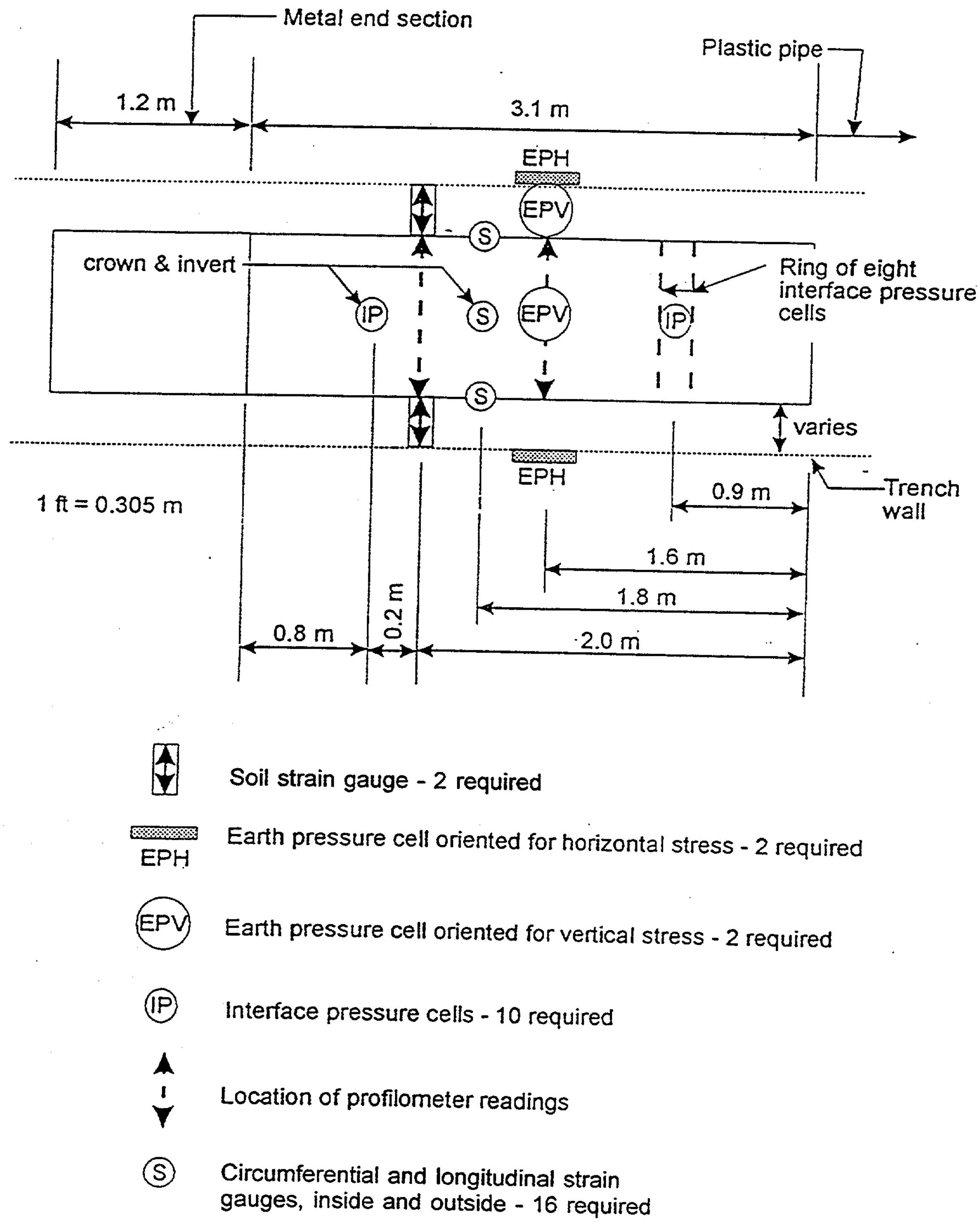



**Figure 4.35 Longitudinal Instrumentation Layout for the Metal Pipe**

### 4.2.5 Test Procedures

The principal purpose of the test was to closely monitor the pipe and soil behavior that take place during the installation and backfilling process. This was accomplished by taking measurements after nearly every layer of backfill was placed at the sides of the pipe. Backfill was placed to a depth of 1.2 m (4 ft) over the pipe for all tests. At the end of a test, the site was immediately re-excavated to retrieve instruments and pipe and to inspect the condition of the bedding.

If the protocol for a test called for compacting the bedding, then this was done with the vibratory plate. Compaction of the backfill was accomplished with the same vibratory plate and rammer compactors that were used for the laboratory tests (see section 4.1.4). If the test plan called for compaction, then two coverages were always used. Backfill over the top of the pipe was compacted with a Bomag, double drum, walk behind, and vibratory roller. The soil unit weights for each type of material and compaction equipment was quite consistent. The data are summarized in table 4.12 for the stone and silty sand materials, expressed as a percentage of maximum dry density (AASHTO T-99), and in table 4.13 for the CLSM and the in situ materials over the pipe, expressed as wet unit weight.

**Table 4.12**
**Soil Compaction Test Results and Moisture Contents**

| Soil type | Compactor | Test Nos. | Compaction Test Results | | Average Moisture Content |
|---|---|---|---|---|---|
| | | | Ave. % of Max. Unit Weight (AASHTO T99) | Stand. Dev. $kN/m^3$ (No. of measurements) | |
| Stone | Rammer | 1,3,9 | 92 | 0.5 (26) | 2 |
| | Vibr. plate | 4,11,13 | 85 | 0.5 (14) | 3 |
| | None | 2,12 | 79 | 0.4(8) | 4 |
| Silty sand | Rammer | 6,8 | 95 | 0.2 (11) | 8 |
| | Vibr. plate | 7,14 | 89 | 0.2 (13) | 7 |
| | None | 5 | 82 | 0.5 (6) | 5 |

1 $kN/m^3$ = 6.4 $lb/ft^3$

**Table 4.13**
**Compaction Test and Moisture Content Results for In Situ Soils**

| Soil type | Compactor | Test Nos. | Ave. Wet Unit Weight kN/m³ | Stand. Dev. kN/m³ (No. of test measurements) |
|---|---|---|---|---|
| In situ sand | Bomag | 1,3,4,6-8 | 20.1 | 0.6 (48) |
| | None | 2,5 | 17 | 0.5 (6) |
| In situ clay | Bomag | 9-14 | 18.7 | 0.8 (28) |
| CLSM | – | 10 | 20.9 | 0.2 (2) |

1 kN/m³ = 6.4 lb/ft³

In general water contents during compaction were below optimum. Only a minimal effort was made to introduce moisture to improve compactibility, as this was deemed more closely related to actual practice. Moisture was added only when the material became dusty and difficult to work with.

Note that although the vibratory plate compactor has a greater mass, the rammer compactor produces substantially higher soil stresses during compaction because of the smaller plate area and impact type of compaction. Table 4.12 shows that the rammer produced significantly higher soil unit weights than the vibratory plate when the same number of coverages were applied.

### 4.2.5.1 Trench Layout

As noted for each test, the concrete, plastic, and metal pipes were laid end to end as shown in fig. 4.29. Most trenches were excavated twice, the first test was conducted in a trench as wide as the pipe outside diameter plus 0.6 m (24 in.), called the narrow condition, and then, while retrieving the pipe from the first test, the trench was widened to equal the pipe outside diameter plus 1.8 m (6 ft) for the second test. For test 14, an intermediate width of the pipe outside diameter plus 0.3 m (3 ft) was used. This trench was only excavated once. Test 10, with CLSM backfill was conducted in a narrow trench that was also excavated only once.

At each trench location, a custom fabricated manhole was set to provide access to the test pipe. Test trenches were excavated in both directions, allowing a total of four tests to be conducted without resetting the manhole. This arrangement allowed excavation to be ongoing in one trench while readings were being taken during backfilling of the trench on the other side of the manhole, thus optimizing the use of the construction equipment.

All trenches were benched, as shown in figs. 4.36, 4.37, and 4.38. The benching resulted in a negative projection ratio of about 0.15 for the 900 mm (36 in.) pipe and a positive projection ratio of about 0.36 for the 1,500 mm (60 in.) diameter pipe.

The concrete pipe was backfilled to the springline with the selected material for a given test (see table 4.8). Excavated in situ material, compacted in the same fashion as the select backfill was used above this level. The selected backfill material was placed to 150 mm (6 in.) above the top of the plastic and metal pipe. For all pipe, the excavated in situ material was used as final backfill from a level 150 mm (6 in.) above the top of the pipe to the ground surface.

#### 4.2.5.2 Typical Test Sequence

Tests were typically conducted in the following steps. Trench configurations and lifts are shown in figs. 4.37 to 4.38. Deviations from these procedures for specific tests are noted in the following subsections.

1. Trenches were excavated to 150 mm (6 in.) below the bottom of the test pipe. The same backfill to be used for the test was placed as bedding and compacted according to the requirements of that particular test. Pipes were set in place, and all instrumentation that was in place was read.

2. Backfill was placed in layers approximately 300 mm (12 in.) thick after compaction. Some adjustments were made to the thickness to allow layers to come to certain target elevations and to accommodate the different outside diameters of the test pipe. After compaction, all in-place instrumentation was read.

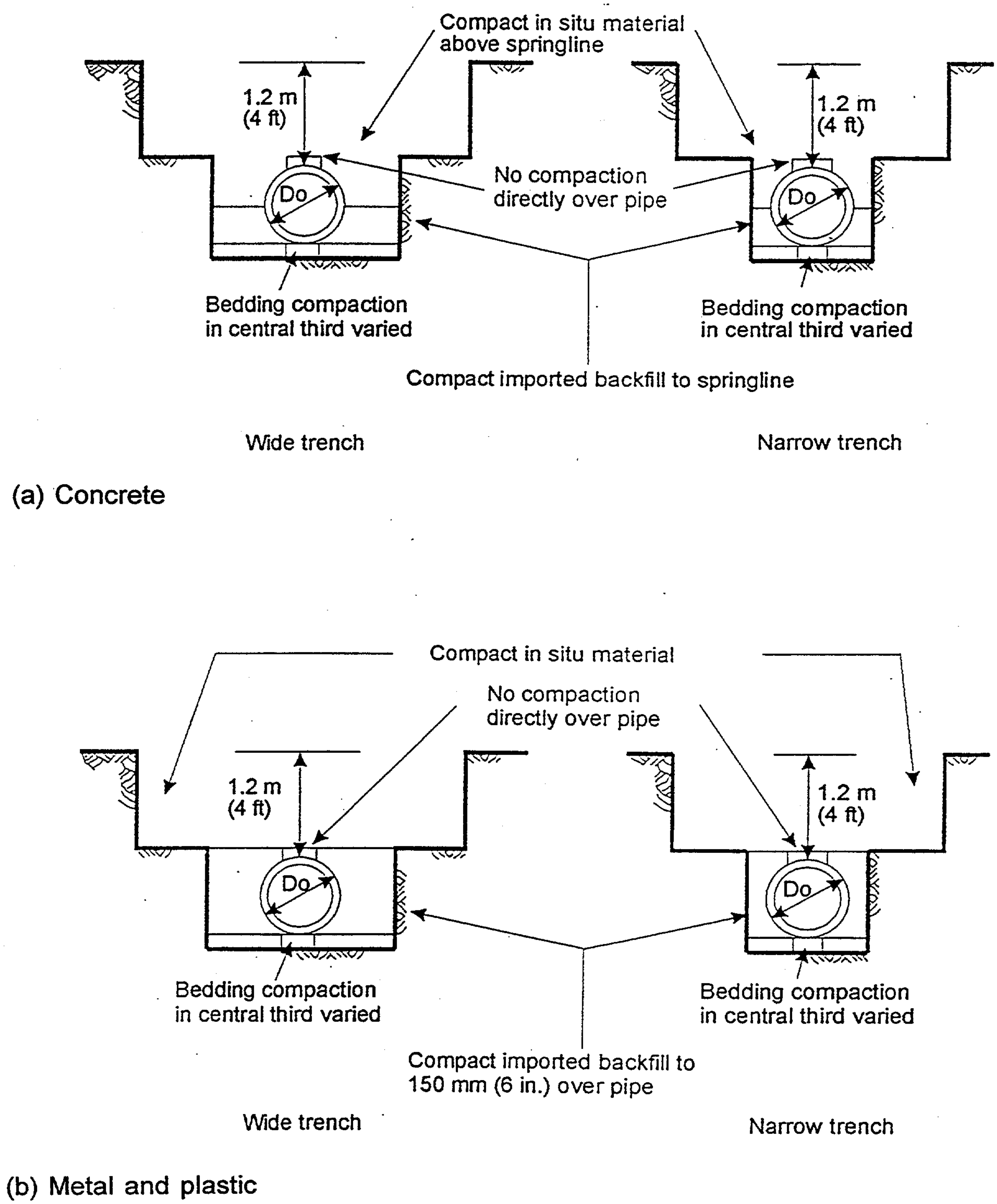



**Figure 4.36 Backfill Configurations for Rigid and Flexible Pipes**

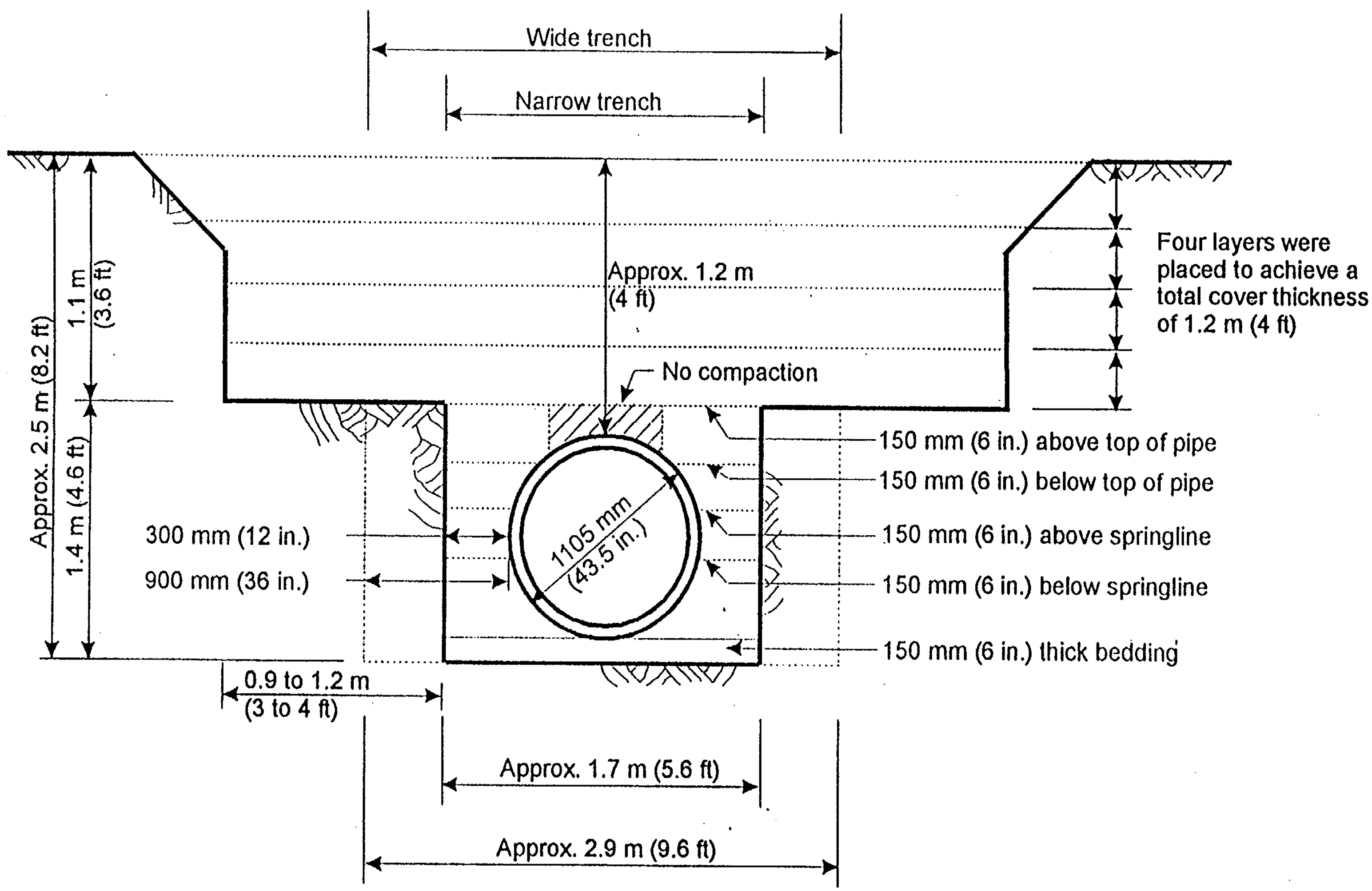



**Figure 4.37 Typical Trench Cross-Section and Backfill Layer Thicknesses for 900 mm (36 in.) Diameter Concrete Pipes (Section for Plastic and Metal Pipes Similar)**

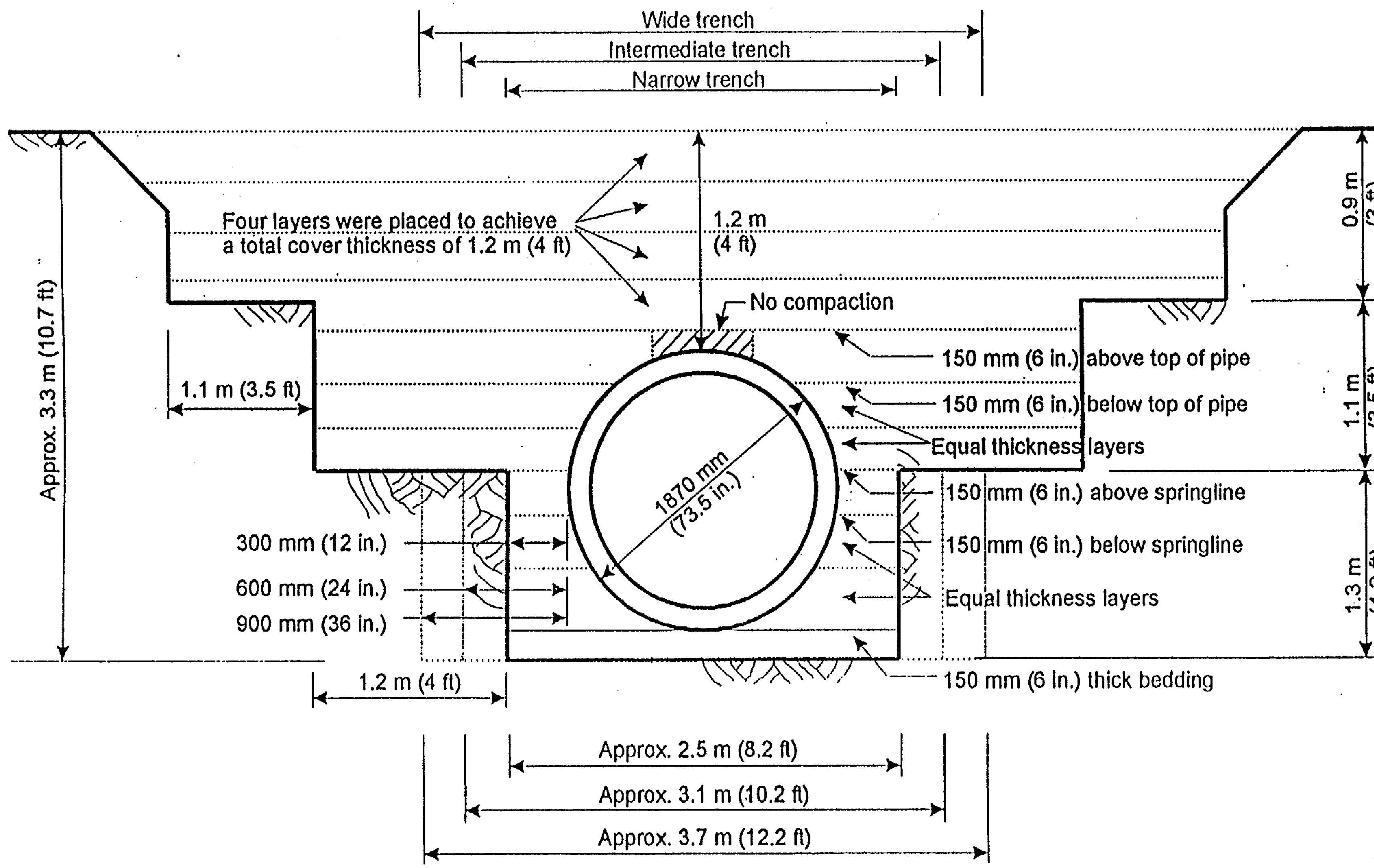



**Figure 4.38 Typical Trench Cross-Section and Backfill Layer Thicknesses for 1,500 mm (60 in.) Diameter Concrete Pipe (Section for Plastic and Metal Pipe Similar)**

3. The trench wall earth pressure cells and the soil strain gages were installed after placing, but before compacting, the backfill layer that came to 150 mm above the springline. The instruments were installed by digging small holes in the backfill. The trench wall was smoothed as much as possible prior to placing instruments up against it. Sand was tamped into any space that was left behind the instrument. After placing the instruments the holes were refilled, initial readings were taken, then the layer was compacted according to the requirements of the plan.

4. The backfill layer that came to 150 mm (6 in.) above the top of the pipe was left uncompacted for a width of 0.45 m (18 in.) centered over the test pipe. After the rest of this layer was compacted, the earth pressure cells used to measure vertical soil stresses were installed, and initial readings were taken.

5. Backfilling was completed with four approximately equal layers of in situ material, of approximately equal thickness, until the total cover over the pipe was about 1.2 m (4 ft). Most instruments were read after compacting each layer; however profilometer readings were taken only after the second and fourth layers.

6. When the fourth layer of in situ material was compacted the test was complete. The pipe were re-excavated to examine the bedding and haunching and to retrieve the test pipe and instruments for use on the next test.

#### 4.2.5.3 Deviations from Typical Test Procedures

The vagaries of the weather, the need to complete all of the tests in a short period of time, and a desire to maximize the information obtained from the tests resulted in deviations from the standard procedures. These deviations are summarized below.

***Test 4*** – While excavating to remove the test pipe after completion of the test, a thunderstorm flooded the trench and prevented inspection of the bedding under the plastic and metal pipe.

***Tests 5, 6, 7, 8, and 14*** – After placing and compacting the bedding for test 5, the trench was left overnight. During this time, groundwater seepage saturated the silty sand creating a running soil condition. The soft soil was excavated and replaced in the worst areas. To avoid this problem, the bedding material was changed to a concrete sand.

***Test 11*** – After placing and compacting the first layer of in situ material over the top of the pipe, heavy rains occurred for several days, flooding the trench and filling the test pipe with water. The water was pumped out and the instruments dried. Work was restarted after a delay of 7 days.

***Test 10*** – CLSM backfill was used for test 10. For this test, imported bedding was not used. The pipe were set on bags of gravel to hold them off of the trench bottom and allow the CLSM to flow underneath. Bags of gravel were also placed on top of the plastic and metal pipe to minimize the risk of flotation. The CLSM was produced at a concrete batching plant and delivered to the site in a concrete truck. The flowability of the mix was checked using a 75 mm (3 in.) diameter, 150 mm (6 in.) long tube. CLSM was placed and leveled in the tube which was then raised. The CLSM had to spread to a diameter of at least 225 mm (9 in.) to indicate proper flow characteristics. CLSM was received in two deliveries. The first delivery was used to bring the fill to about 150 mm (6 in.) above the invert. About 2 hours later, the second lift was placed to just above the pipe springline. While the second lift was being placed, the metal pipe came free and raised up about 40 mm (1.6 in.). The plastic pipe, even though it was lighter, did not lift. Apparently the deep corrugations allowed the plastic pipe to develop an anchorage to the first pour that prevented flotation. The morning after the CLSM was placed, the trench backfilling was completed. For all pipe, the in situ clay material was placed and compacted with the rammer compactor to a level 150 mm (6 in.) above the crown. Backfill above this point followed the standard test procedures. Because of the nature of the test and the plan to leave the pipe in the ground for a period of time, the soil strain gages and earth pressure cells were not installed for this test. The CLSM test pipe were left in the ground for 22 days before excavation.

### 4.2.6 Results

Measurements taken during the field test program covered a wide range of behavior. Complete data are presented in Webb (1995), Webb et al. (1995), and Zoladz et al. (1995).

#### 4.2.6.1 Pipe Deflections

Plots of deflection versus depth of fill are presented in fig. 4.39 for 9 of the 14 tests. The deflections generally reflect the effects of the compaction method used and the soil unit weights that were achieved. Tests compacted with the rammer, which creates the highest soil stresses during compaction, showed the most peaking (upward deflection when the backfill is at the top of the pipe, (depth of fill equal to 0.0 m), and the least downward deflection as backfill was placed over the crown. The final deflected shape for pipe with rammer compacted backfill was always ovalled upward at the end of the test. The vibratory plate compactor produced less peaking and more downward deflection as backfill was placed over the top of the pipe. This is consistent with the lower density produced by the vibratory plate. Most pipe in tests where the vibratory plate was used for compaction were deflected downward at the end of the test. Tests with no compaction applied to the backfill showed about the same peaking as tests compacted with the vibratory plate; however, these tests with no compaction showed more downward deflection due to backfilling over the pipe. One exception to the above trends is test 7 (Fig. 4.39c and 4.39d). Even though backfill was compacted with the vibratory plate, the deflection profile appears to follow that of test 5 which had no compaction. The backfill material for test 7 was the silty sand, and no haunching effort was applied. As noted above, this material is very sensitive to moisture. When this test was backfilled to a level 150 mm (6 in.) over the pipe, it was left overnight. On the following morning, several instruments showed that the backfill had softened overnight. The earth pressure and several pipe-soil interface pressure cells showed drops in stress levels, and the invert interface pressure cell showed an increase. It is believed that the silty sand took up moisture from the surrounding native material and flowed into the voids in the haunch zone, causing the drop in pressure and the increased deflections. Also, the deep corrugations of the plastic pipe, which are not filled with backfill in the lower region of the pipe may have provided a larger void, relative to the metal pipe, which could explain part of the increased deflection in the plastic pipe for this test.

The metal pipe showed less peaking than the plastic pipe. This is expected because of the higher metal pipe bending stiffness. Peaking behavior is affected more by this pipe stiffness than is downward deflection due to backfilling over the pipe. Downward deflection is controlled more by soil stiffness. This is also reflected in the higher peaking deflections in the 1,500 mm (60 in.) diameter plastic pipe than in equivalent tests in the 900 mm (36 in.) diameter plastic pipe. The 1,500 mm (60 in.) plastic pipe had the lowest pipe bending stiffness of all of the pipe tested.

The smaller deflection change during the last backfill increment for the tests with no compaction of the backfill indicates a reduction in the rate of deflection. This could suggest that the pipe deflected sufficiently to mobilize support from the trench walls, which were much stiffer than the backfill or that the low compactive effort left voids in the backfill around the pipe which closed up, resulting in a higher rate of deflection during the first increments of backfill.

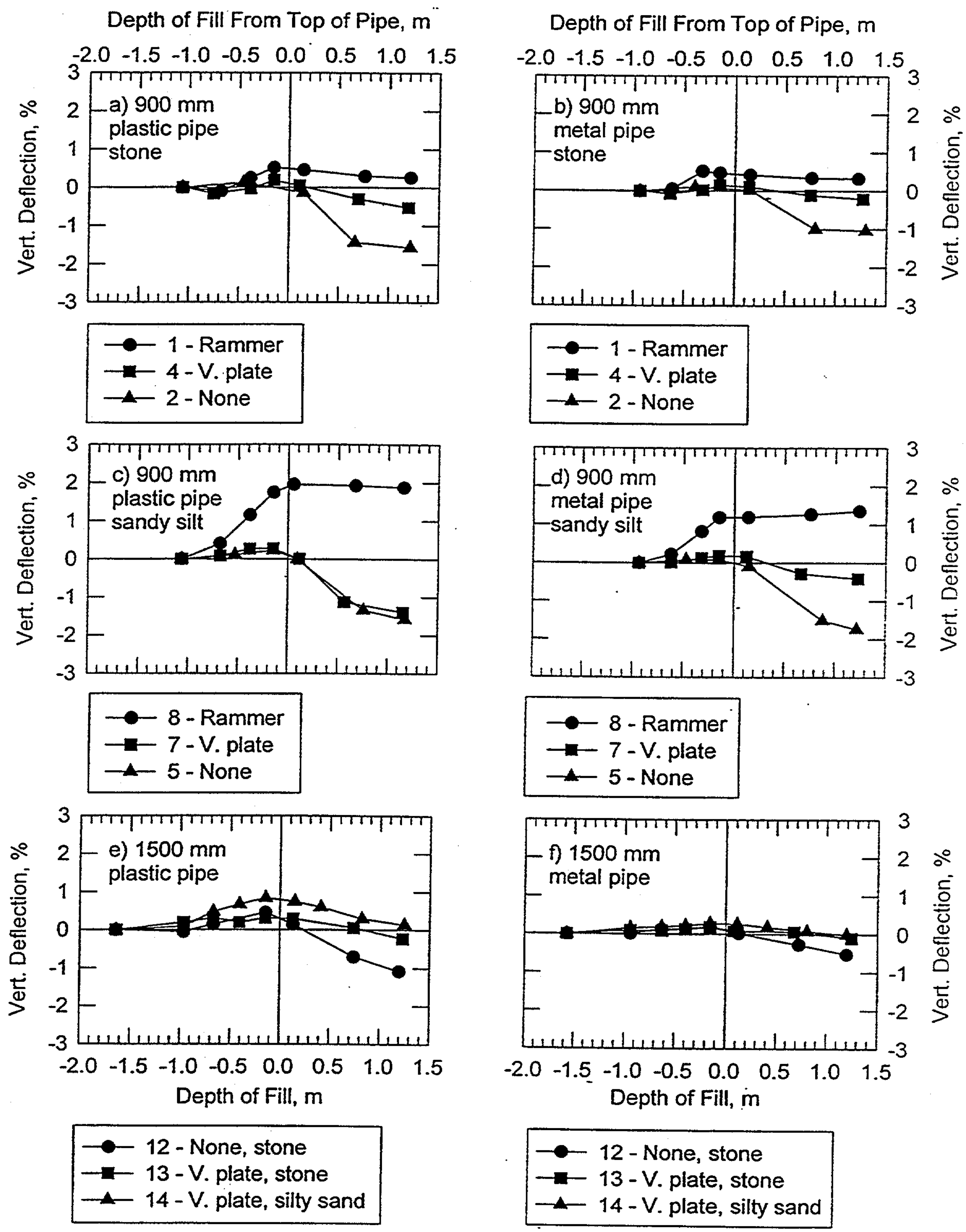



**Figure 4.39 Typical Plots of Vertical Deflection Versus Depth of Fill**

Vertical deflections for all tests are summarized in figs. 4.40a and 4.40b which show the peaking deflection, the change in deflection during backfilling over the top of the pipe, and the final deflection at the end of the test. Fig. 4.40(c) shows the ratio of change in vertical deflection to change in horizontal deflection caused by backfilling over the crown. Together, Figs. 4.39 and 4.40 show:

- Significantly more peaking occurred with the silty sand backfill than the stone backfill. This is probably because of the higher lateral pressures generally exerted by the lower strength of finer grained soils and the reduced pressures due to the higher strength from the interlocking of the stone particles.

- The downward deflection in test 11 was higher than expected based on other results. This was particularly true of the plastic pipe. Test ll was flooded during the backfilling process, and the flooding apparently softened the backfill and the trench walls. This was the only test where the soil strain gages showed significant outward movement of the trench walls during backfilling over the top of the pipe.

- Tests with wide trenches show slightly more peaking during backfilling to the top and slightly less downward deflection due to backfilling over the top of the pipe than equivalent tests in narrow trenches. Tests 1 and 3 and tests 6 and 8 are used for this comparison.

- The ratio of the vertical to horizontal deflection due to backfilling over the crown is generally larger in absolute magnitude for the plastic pipe than for the metal pipe, particularly when backfill was compacted with the rammer, where the ratios were substantially larger than 1.0. This is thought to be due, at least in part, to the lower hoop stiffness of the plastic pipe. This type of pipe has been shown to undergo substantial circumferential shortening relative to traditional flexible pipe, when subjected to earth load. This shortening is seen as a decrease in vertical and horizontal diameter, hence the higher ratios of vertical to horizontal deflections.

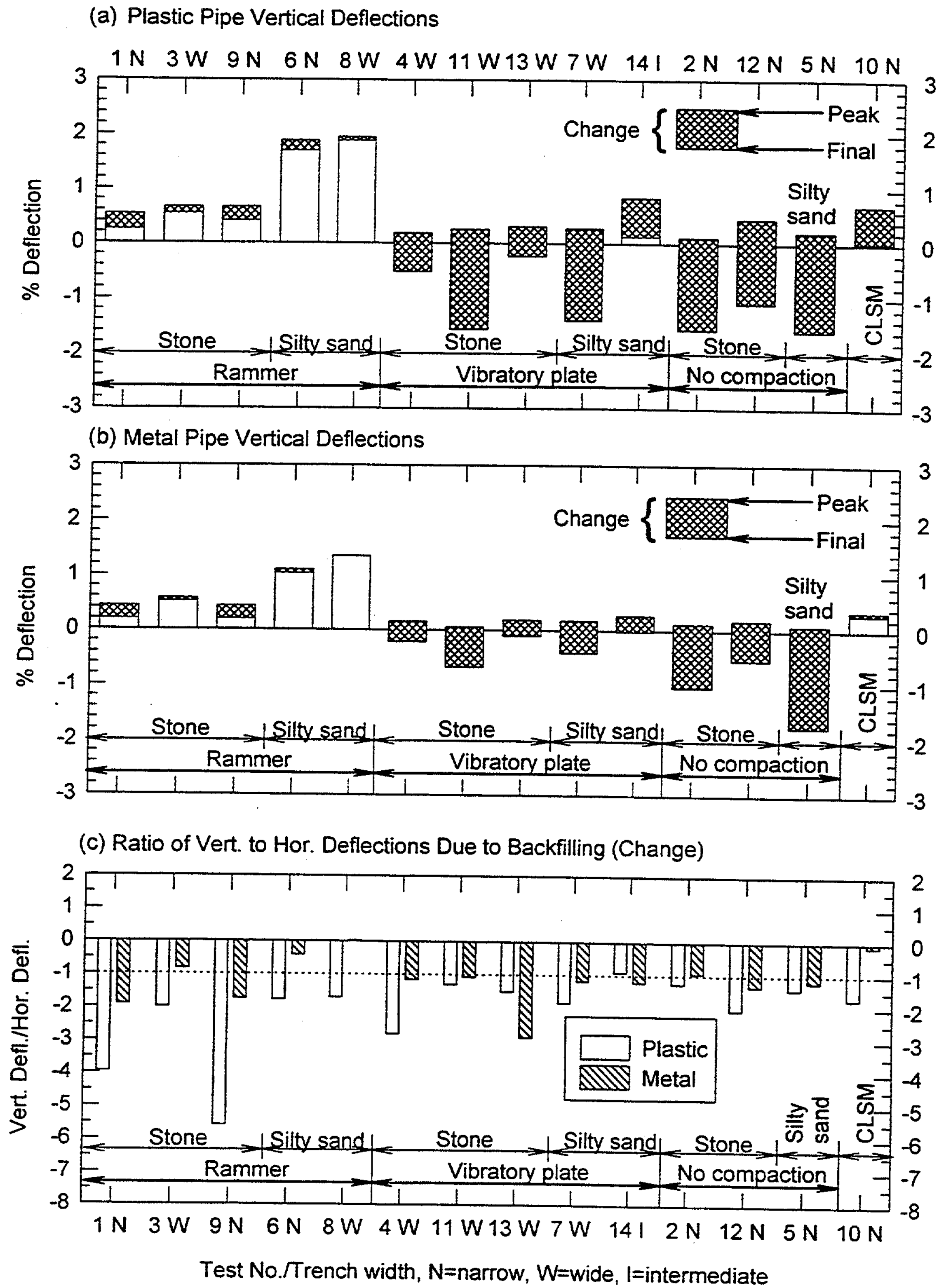



**Figure 4.40 Summary of Field Test Deflections**

#### 4.2.6.2 Pipe-Soil Interface Pressures

The development of interface pressure on the concrete pipe for tests 1 to 4, with stone backfill, and partial data for tests 5 to 8, with silty sand backfill are presented in fig. 4.41. The end of test interface pressures for tests 1 to 4 in a radial plot are presented in fig. 4.42. In both figures, the invert interface pressures are the changes after the pipe was set in place, thus the weight of the pipe is not reflected.

The highest invert pressure occurs for test 2 where no haunching or compactive effort was provided. Test 1, compacted with the rammer and haunched, shows a decrease in invert pressure as the sidefill was placed and compacted, suggesting that the compactive effort actually lifted the pipe off the bedding. Tests 3 and 4 show intermediate results.

Interface pressures at thirty degrees from the invert are low regardless of compactive effort or haunching effort. This suggests that design should always consider a region of the haunch as unsupported after backfilling.

The benefit of higher compactive effort is clearly seen in the interface pressures at 60 degrees from the invert. The two tests where the backfill was compacted with the rammer show high pressures. This is beneficial for pipe performance as it indicates more uniform support for the pipe. Interface pressures at this location for test 4, compacted with the vibratory plate, showed very little difference from the pressures in test 2, where no compactive effort was applied.

For tests 5 to 8, with silty sand backfill, the data is similar to that for the tests with stone backfill. The tests where the rammer compactor was used show higher interface pressures. Of interest are the drops that occur for tests 6 and 8 at a backfill depth of about 0.1 m (4 in.) over the top of the pipe. This drop occurred overnight. As discussed previously for the deflections of test 7, the silty sand is sensitive to moisture and the overnight delay in backfilling may have allowed the material to take up water and soften. For tests 6 and 8, the drop in the radial pressure does not appear to be paralleled with an increase in deflection for the plastic and metal pipe, as was the case with test 7. This is likely because tests 6 and 8 had backfill with higher unit weights, from the rammer compaction and haunching during backfilling.

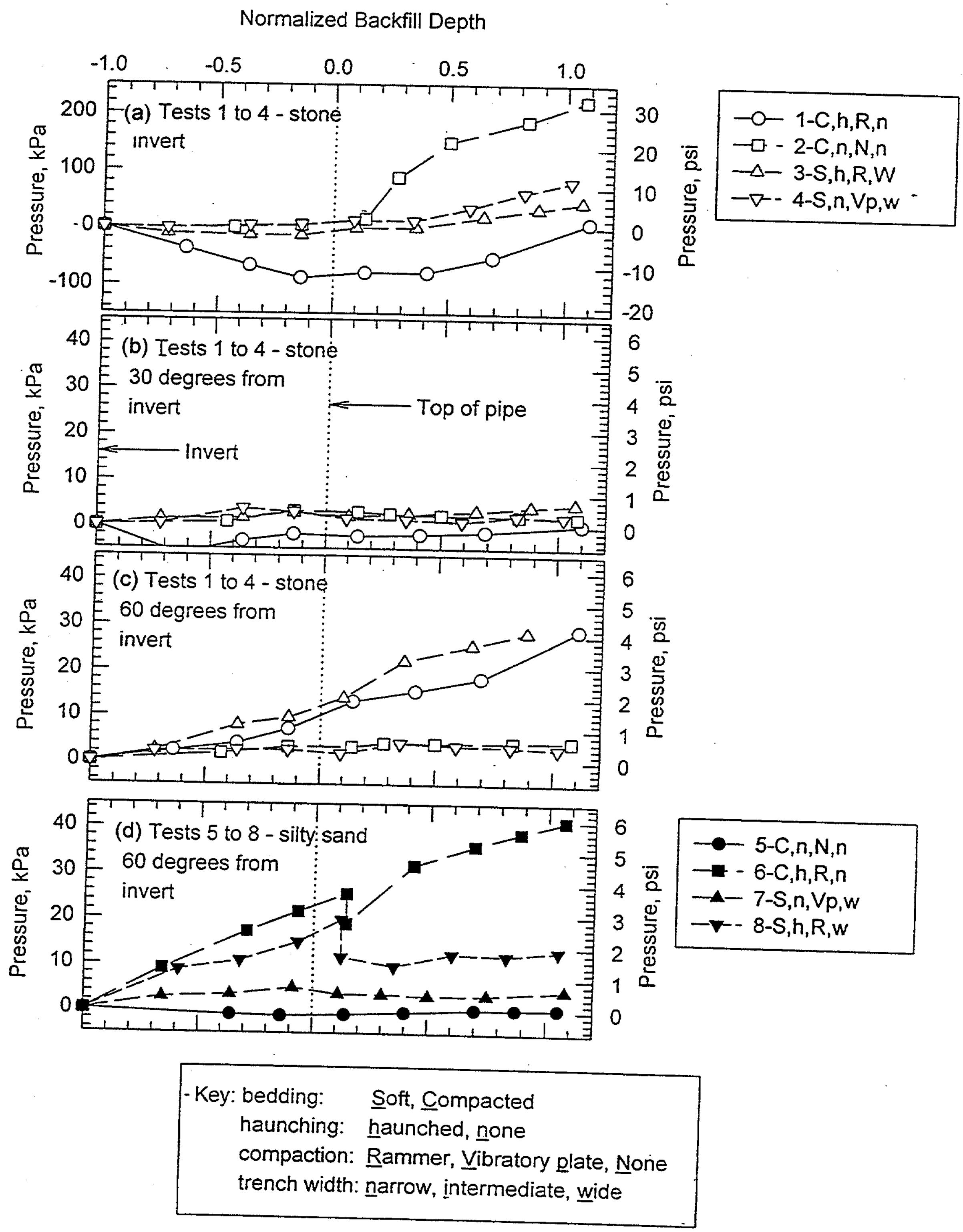
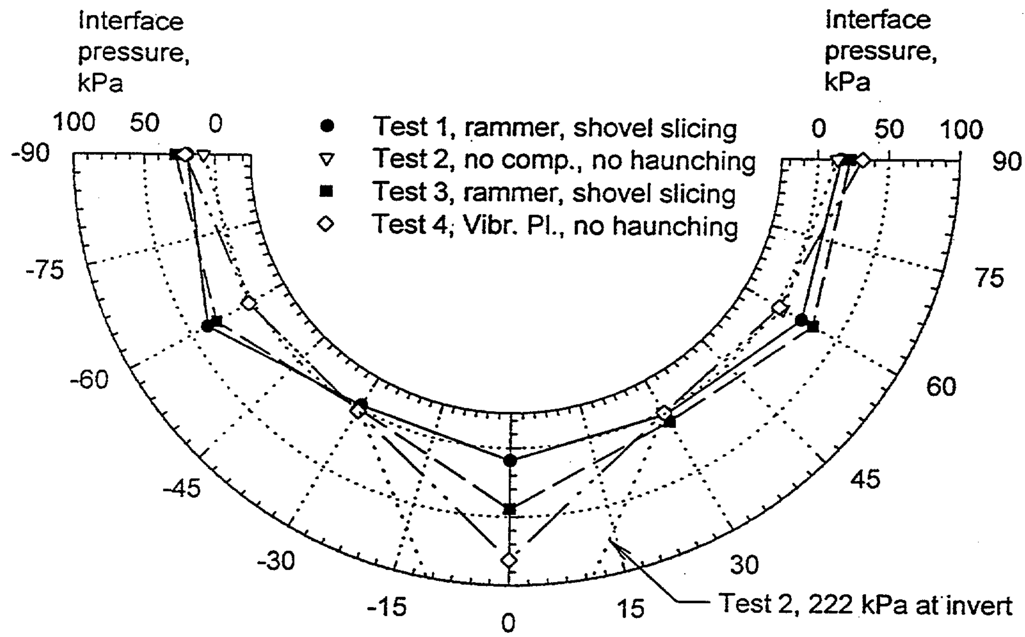



**Figure 4.41 Concrete Pipe Interface Pressures**




1 psi = 6.89 kPa

**Figure 4.42 Radial Pressures, 900 mm (36 in.) Diameter Concrete Pipe, Stone Backfill**

Interface pressure data for the other tests was similar. The end-of-test invert interface pressures under the 1,500 mm (60 in.) pipe (tests 12 to 14, all with haunching) were between 100 and 200 kPa (14.5 and 29 psi), which were all less than the pressure under the concrete pipe in test 2 without haunching.

### 4.2.6.3 Trench Wall Soil Stresses

Earth pressure cells were installed at the trench wall at the springline level to monitor the soil stress at this location as backfill was placed. Fig. 4.43 presents the data from tests 5, 6, and 7 in the form of stress versus depth of fill. Figure 4.44 is a bar chart showing, for all tests where data was taken, the trench wall stress when the backfill was at the top of the pipe, and at the end of the test. Typical trends, as displayed by the figures include:

- In tests with no compaction, lateral stresses do not develop at the springline level of any type of pipe until the backfill level rises over the top of the pipe. During backfilling above the crown, trench wall interface stresses develop beside the plastic and metal pipe, but stresses next to the concrete pipe are never greater than about 5 kPa. The trench wall stress beside the flexible pipe develops because the pipe is deflecting outward into the soil.

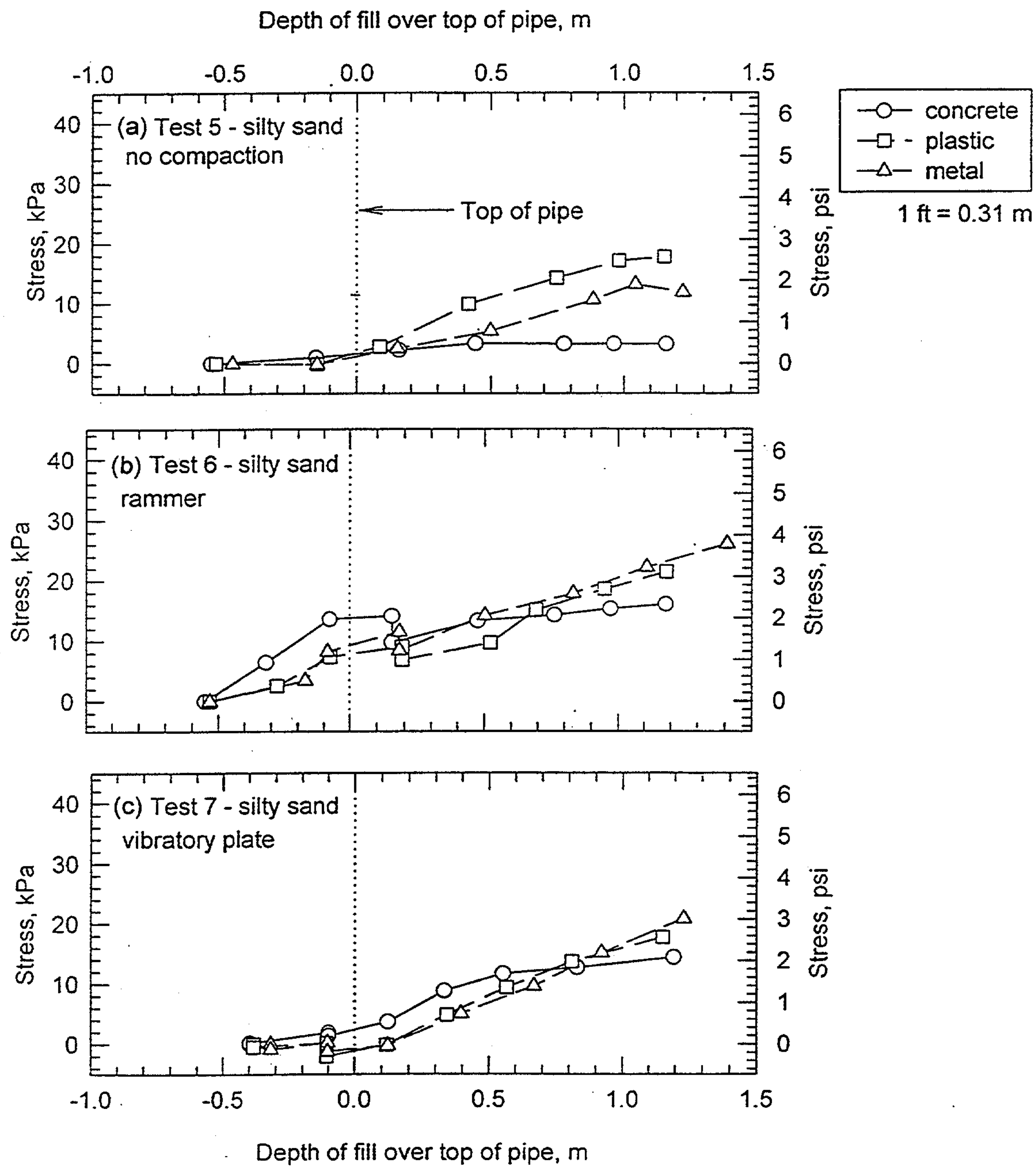



**Figure 4.43 Horizontal Soil Stresses at Springline at Trench Wall-Backfill Interface**

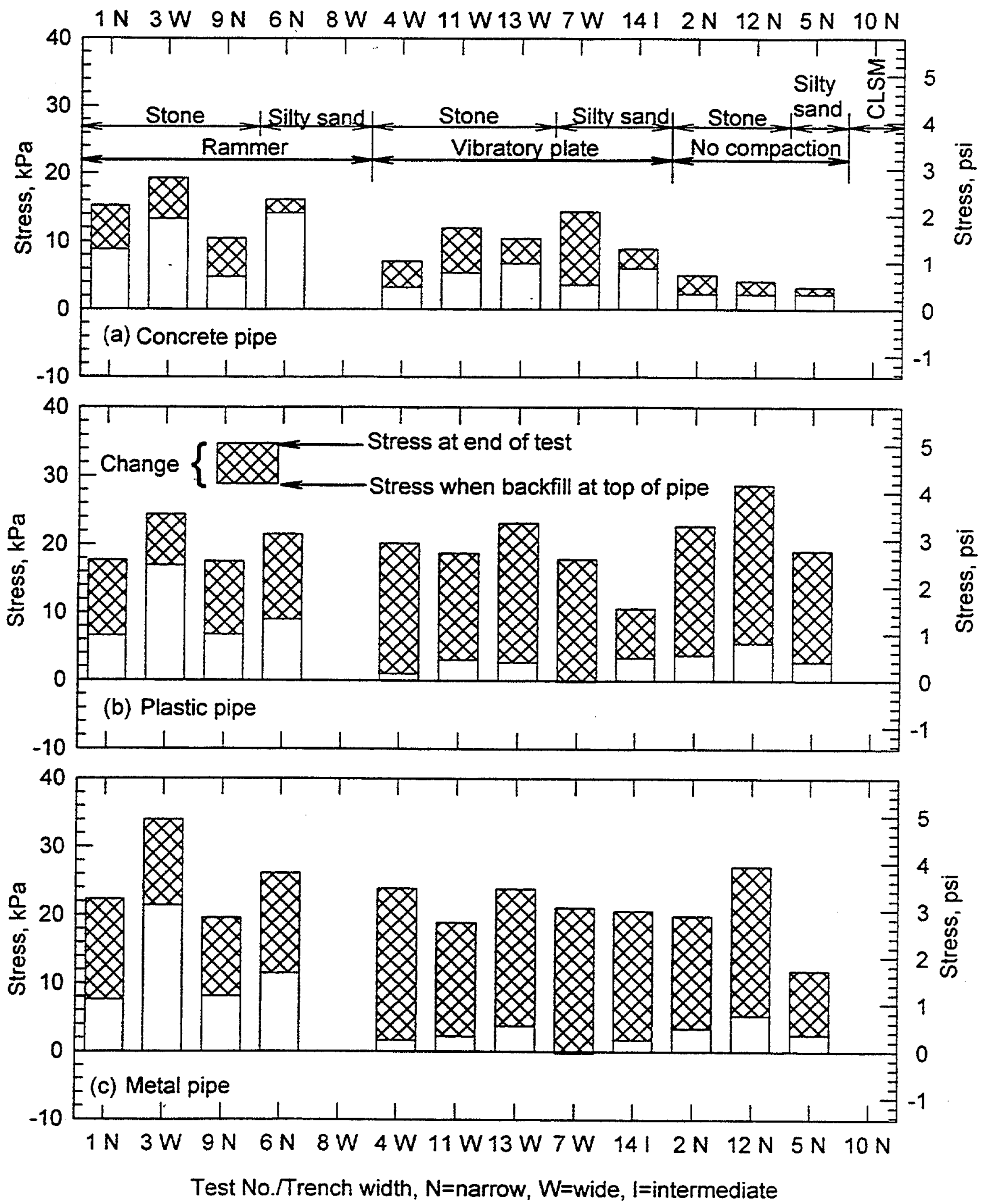



**Figure 4.44 Summary of Horizontal Stresses at Trench Wall**

- For concrete pipe in tests with compactive effort applied, horizontal stresses develop during compaction; however, as backfill is placed over the pipe the rate of increase in lateral stress at the trench wall is reduced.

- While the sidefill is placed, the plastic and metal pipe only develop lateral pressure when the sidefill is compacted with the rammer. When the sidefill is compacted with the vibratory plate only small trench wall stresses develop. These observations are consistent with the development of peaking deflections as the sidefill is compacted with the rammer, but not with the vibratory plate.

- The only direct comparison to evaluate trench wall stresses developed in narrow and wide trenches are tests 1 and 3. For all three pipe the trench wall stress developed while placing the sidefill was greater for test 3, the wide trench. The change in horizontal stress as the backfill was placed over the pipe was the same in test 3 as in test 1. The net effect was that all three pipe developed more lateral stress when installed in the wide trench.

- For the tests with no compaction, less trench wall stress developed in test 5, with silty sand backfill, than in tests 2 and 12 with stone backfill.

- The only instances in which no trench wall stresses developed while placing sidefill was with the flexible pipe in test 7. Actually, as shown in fig. 4.43, a small stress developed during placement of the sidefill, but it dissipated overnight. This is consistent with the previous hypothesis that the sandy silt backfill in this case softened while testing was stopped for the night.

- For test 11, during which the backfill became flooded, trench wall stresses developed to about the same magnitude as during tests 4 and 13, even though higher deflections developed during those tests.

- For the plastic and metal pipe the final trench wall pressures are generally the same at the end of all tests, regardless of type of compaction, backfill type or trench width, even though as noted above, the deflections varied widely.

#### 4.2.6.4 Vertical Soil Stresses Over Pipe

Vertical soil stresses directly over the pipe and sidefill are summarized in table 4.14. The stresses are normalized by the geostatic soil stresses at the elevation of the gages based on the soil unit weights in table 4.12. The ratio of the crown to sidefill stress is not the arching factor but is indicative of the arching of load onto, or off of, the pipe. No trend was noted based on diameter or trench width, thus the data is presented by type of compaction.

**Table 4.14**
**Normalized Vertical Soil Stresses Over the Test Pipes**

| Location | Concrete | | Plastic | | Metal | |
|---|---|---|---|---|---|---|
| | Mean | Std. Dev. | Mean | Std. Dev. | Mean | Std. Dev. |
| **a. Rammer compactor** (Tests 1, 3, 6, 8, 9) | | | | | | |
| Crown<br>Sidefill | 0.96<br>1.03 | 0.10<br>0.26 | 0.91<br>1.19 | 0.21<br>0.19 | 1.06<br>1.21 | 0.08<br>0.17 |
| Crown /sidefill (%) | 94 | | 77 | | 88 | |
| **b. Vibratory plate compactor** (Tests 4, 7, 11, 13, 14) | | | | | | |
| Crown<br>Sidefill | 1.04<br>1.11 | 0.08<br>0.14 | 0.96<br>1.15 | 0.22<br>0.11 | 0.98<br>1.05 | 0.24<br>0.09 |
| Crown /sidefill (%) | 94 | | 83 | | 93 | |
| **c. No compaction** (Test 2, 5, 12) | | | | | | |
| Crown<br>Sidefill | 1.28<br>0.87 | 0.23<br>0.21 | 0.94<br>1.10 | 0.20<br>0.20 | 0.99<br>1.11 | 0.17<br>0.22 |
| Crown /sidefill (%) | 147 | | 85 | | 89 | |

Table 4.14 suggests the following:

- With one exception, the crown vertical pressure is highest over the concrete pipe, lowest over the plastic pipe and intermediate over the metal pipe. This is consistent with traditional load theory. The one exception, the metal and concrete pipes with the rammer used for compaction, is thought to be anomalous.

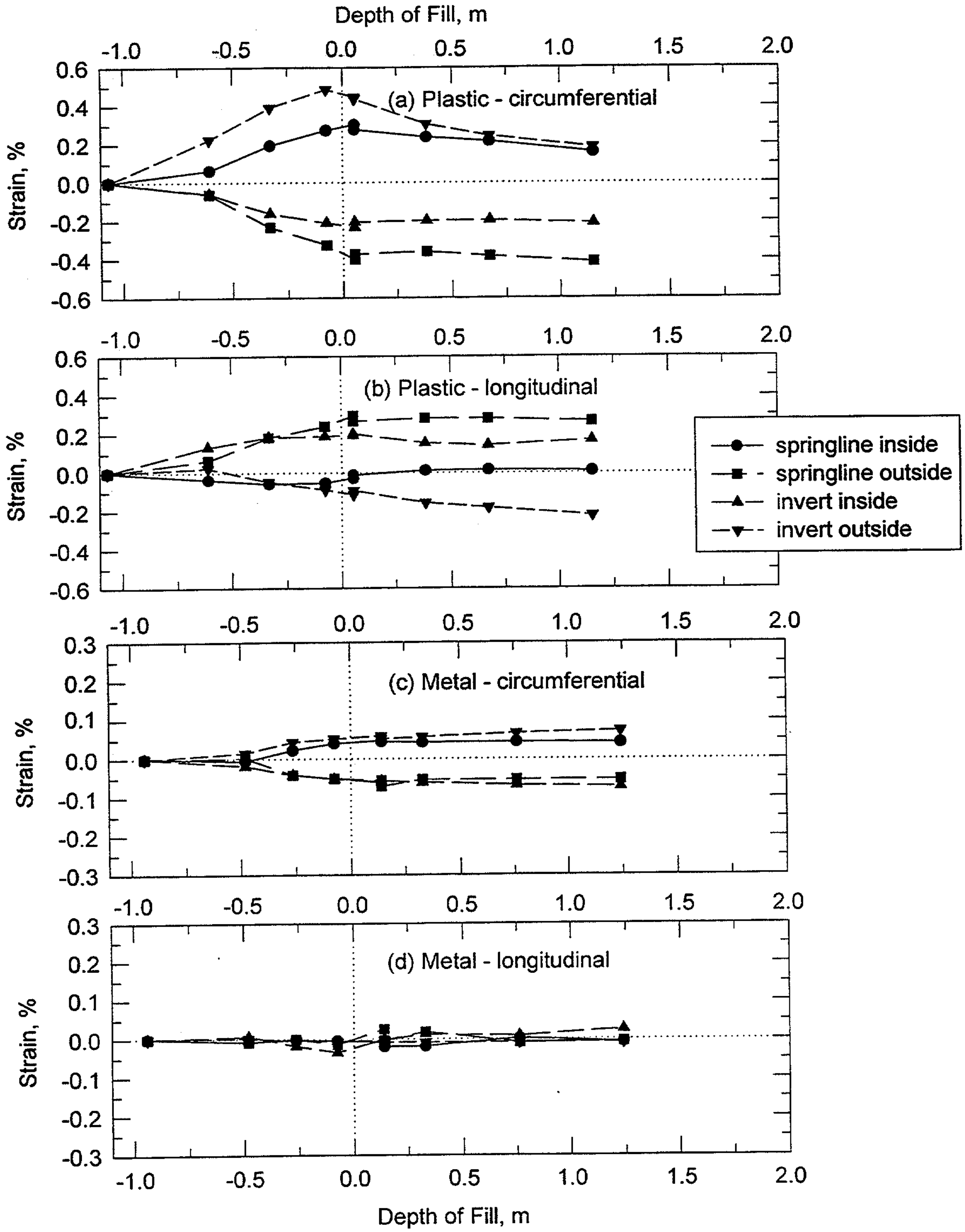
- For the plastic and metal pipes, the vertical soil stress over the sidefill is always greater than over the crown. This is also true for the concrete pipe with compaction. However, for the concrete pipe with no backfill compaction, the crown stress is greater than the sidefill soil stress.

#### 4.2.6.5 Pipe Wall Strain

The development of strains in the pipe wall during backfilling paralleled the development of deflections. As an example, figs. 4.45 to 4.47 present the invert and right springline strain versus depth of fill for tests 8, 12, and 2, respectively. These tests represent the three types of compaction, two pipe sizes, and two backfill types used in the tests. Peaking develops in test 8 during placing and compaction of the sidefill and stabilizes or partially reverses as fill is placed over the pipe. In test 2, with no compaction, there is very little peaking strain, but notable strain as backfill is placed over the crown. The plastic pipe strains in test 12, with the 1,500 mm (60 in.) diameter pipe, are quite small because the profile depth of the 1,500 mm (60 in.) plastic pipe is less than that of the 900 mm (36 in.) diameter pipe, thus there is far less bending response. Strains in the metal pipe follow the same trend as the plastic pipe but are much smaller, which is consistent with the relative depth of the pipe walls. Longitudinal strains in the plastic pipe are significant relative to the circumferential strains, while longitudinal strains in the metal pipe are small at all locations.

Figs. 4.48 and 4.49 show the total strain versus deflection at the end of each test for the plastic and metal pipes, respectively. Also shown on the figures is a linear regression curve for the data. For both pipe there is a reasonable linear correlation between the two parameters, but the slopes and intercepts of the regression curves differ significantly. Observations include:

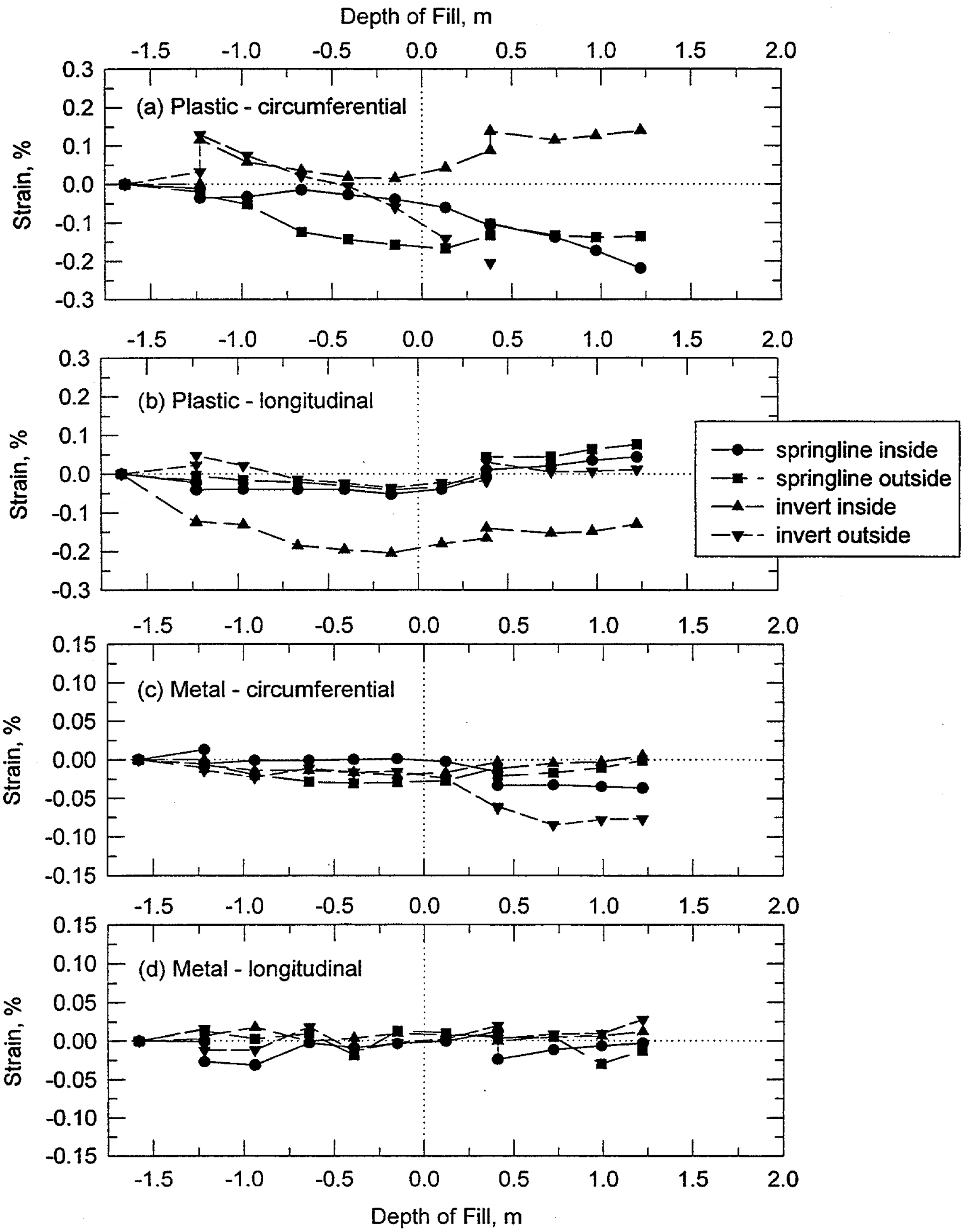
- The left and right sides of each pipe show approximately the same trend, thus reasonable symmetry was achieved in the tests;
- The reversed slopes for the regression lines of the inside and outside circumferential gages suggest that strains are dominated by bending effects. (The one exception to this is the crown gages in the metal pipe, where the outside gages show a negative slope. The relatively parallel slopes suggests that hoop forces are significant. The reason for this is not clear at this time.);
- The longitudinal strains in the metal pipe are small and do not appear to be related to deflection; and
- The longitudinal strains in the plastic pipe are significant (of equivalent magnitude to the circumferential strains) at all locations except at the inside gages at the springline.




Note: Test 8 was conducted in a wide trench with silty sand backfill and was compacted with the rammer.

1 ft = 0.31 m

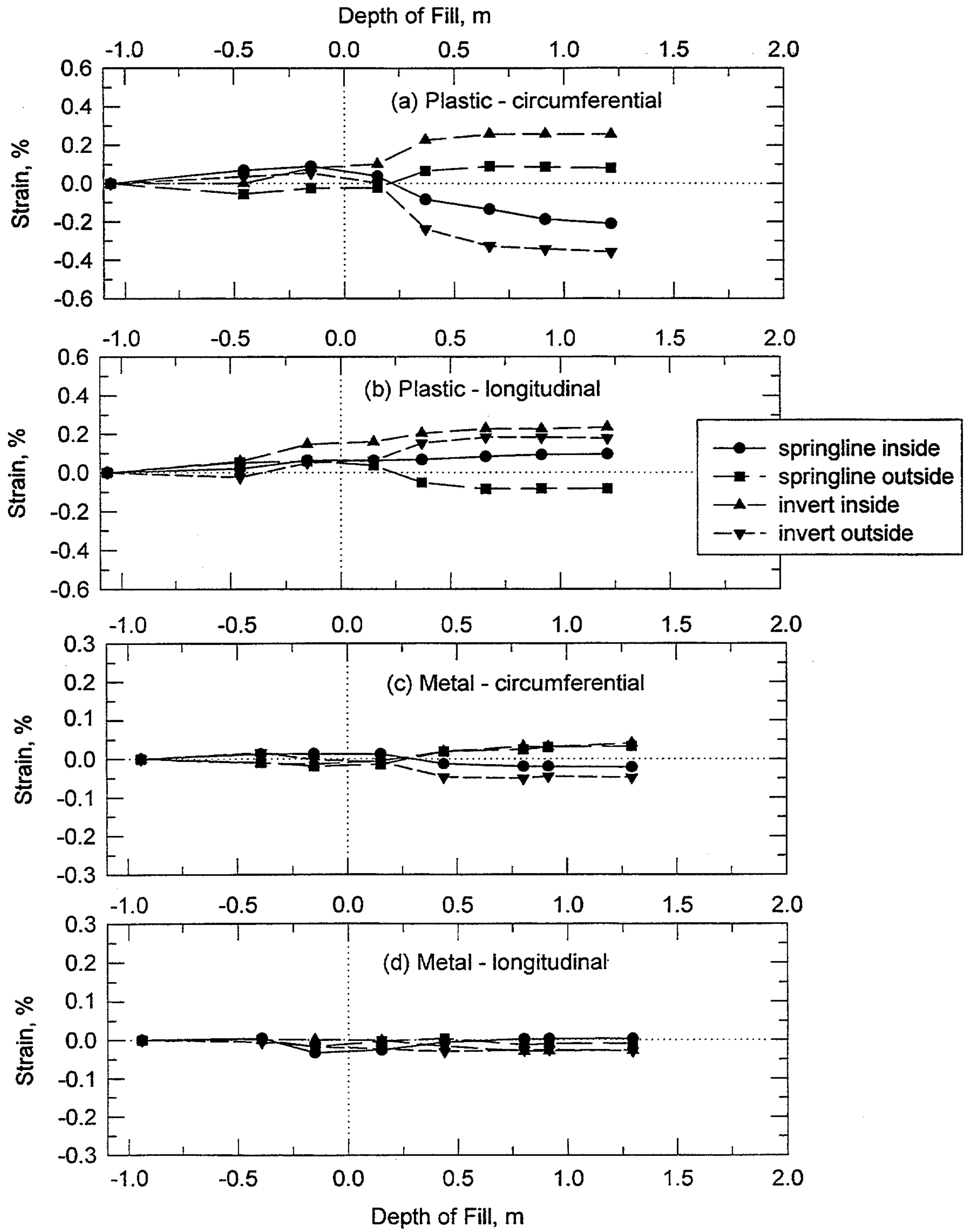
**Figure 4-45 Pipe Wall Strains From Test 8**




Note: Test 12 was conducted in a narrow trench with stone backfill and was not compacted.

1 ft = 0.31 m

**Figure 4-46 Pipe Wall Strains From Test 12**




Note: Test 2 was conducted in a narrow trench with stone backfill and was not compacted.

1 ft = 0.31 m

**Figure 4-47 Pipe Wall Strains From Test 2**

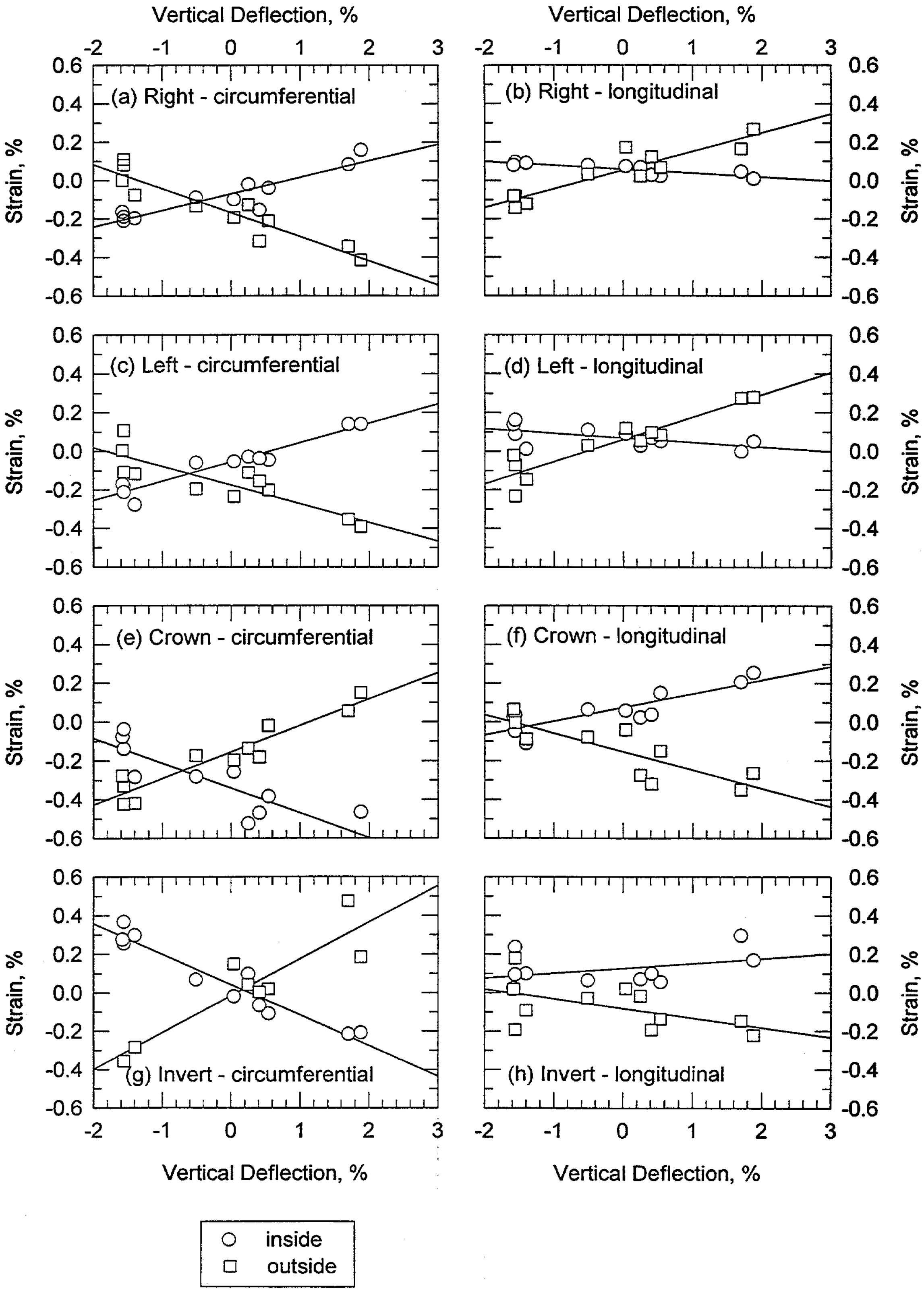



**Figure 4-48 Strain and Deflection at End of Backfilling for 900 mm (36 in.) Plastic Pipe**

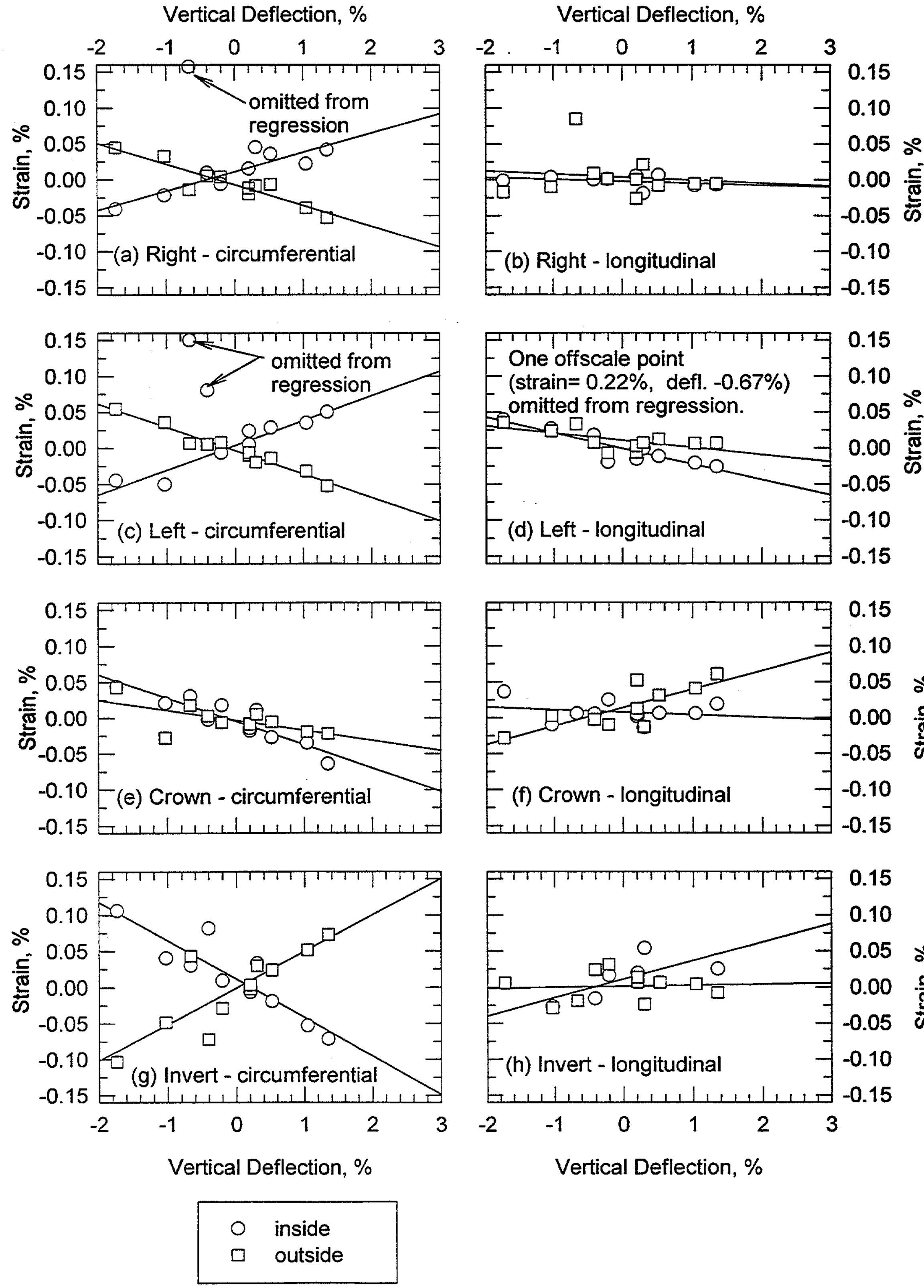



**Figure 4-49 Strain and Deflection at End of Backfilling 900 mm (36 in.) Metal Pipe**

The total strains can be separated into bending and hoop components. The Poisson effect circumferential strains are removed by using the measured longitudinal ($\epsilon_{l\text{-}m}$) and circumferential ($\epsilon_{c\text{-}m}$) strains at the same location and the relationships:

$$\epsilon_{c\text{-}d} = \frac{\epsilon_{c\text{-}m} + \epsilon_{l\text{-}m}\,\nu}{1 - \nu^2}\,, \tag{4.1}$$

and

$$\epsilon_{l\text{-}d} = \frac{\epsilon_{l\text{-}m} + \epsilon_{c\text{-}m}\,\nu}{1 - \nu^2}\,. \tag{4.2}$$

where

| | | |
|---|---|---|
| $\epsilon_{c\text{-}d}$ | = | circumferential strain due to direct stress, |
| $\epsilon_{c\text{-}m}$ | = | measured circumferential strain, |
| $\nu$ | = | Poisson's ratio, |
| $\epsilon_{l\text{-}m}$ | = | measured longitudinal strain, and |
| $\epsilon_{l\text{-}d}$ | = | longitudinal strain due to direct stress. |

Assuming a linear distribution of strain across the wall, these direct strains can then be separated into the components due to hoop thrust and bending moment using the expressions:

$$\epsilon_h = \epsilon_{c\text{-}d\text{-}out} - \left( \frac{\epsilon_{c\text{-}d\text{-}out} - \epsilon_{c\text{-}d\text{-}in}}{c_{in} - c_{out}} \right) c_{out}\,, \tag{4.3}$$

$$\epsilon_{b\text{-}in} = \epsilon_{c\text{-}d\text{-}in} - \epsilon_h\,, \tag{4.4}$$

and

$$\epsilon_{b\text{-}out} = \epsilon_{c\text{-}d\text{-}out} - \epsilon_h\,, \tag{4.5}$$

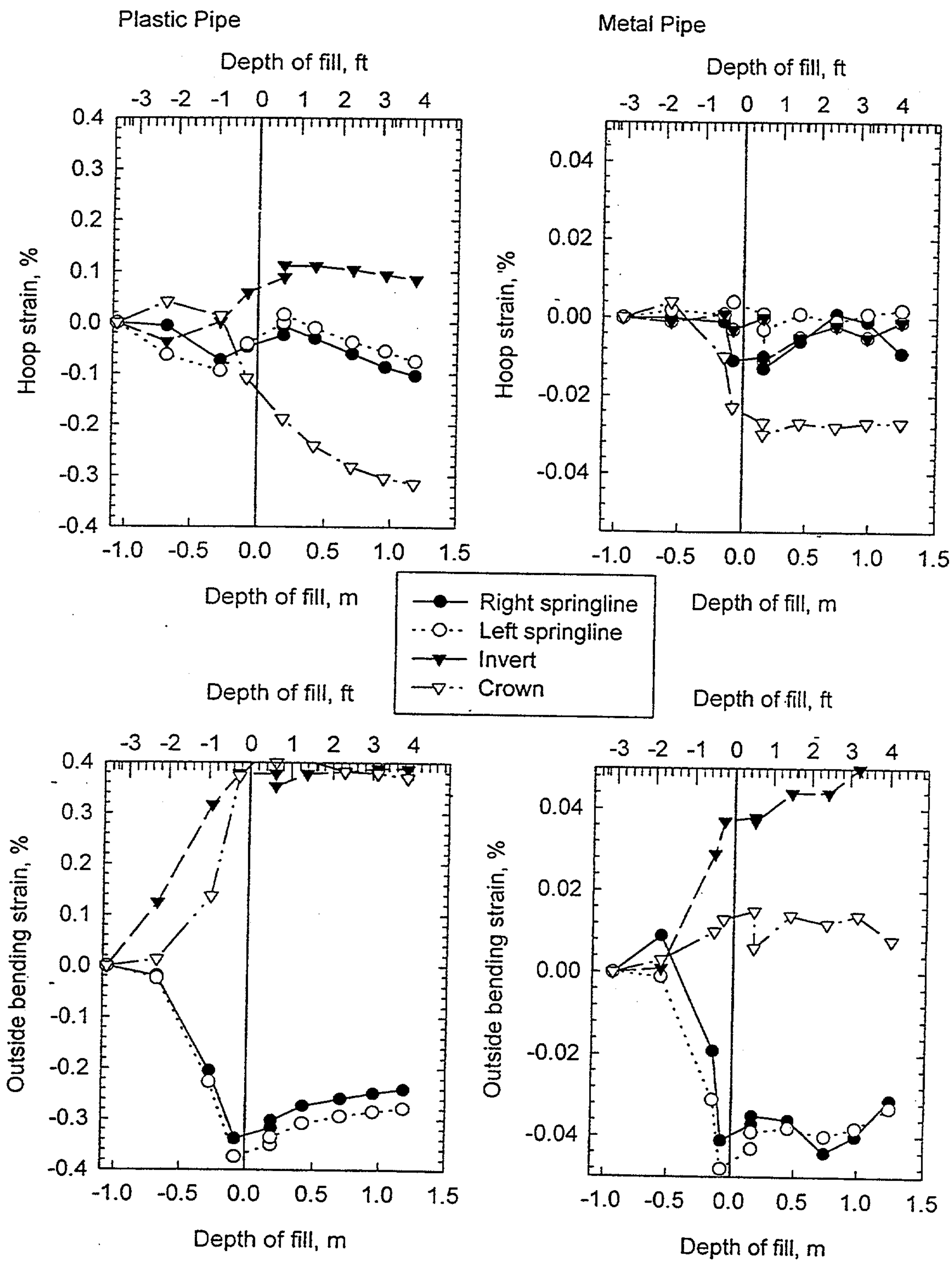
where

| | | |
|---|---|---|
| $\epsilon_h$ | = | strain due to hoop compression forces, |

$\epsilon_{c\text{-}d\text{-}out}$ = outside strain caused by direct stress,
$\epsilon_{c\text{-}d\text{-}in}$ = inside strain caused by direct stress, and
$c_{in}$ = distance from centroidal axis to inside surface, mm, in.,
$c_{out}$ = distance from centroidal axis to outside surface, mm, in.,
$\epsilon_{b\text{-}out}$ = strain on outside surface caused by bending forces, and
$\epsilon_{b\text{-}in}$ = strain on inside surface caused by bending forces.

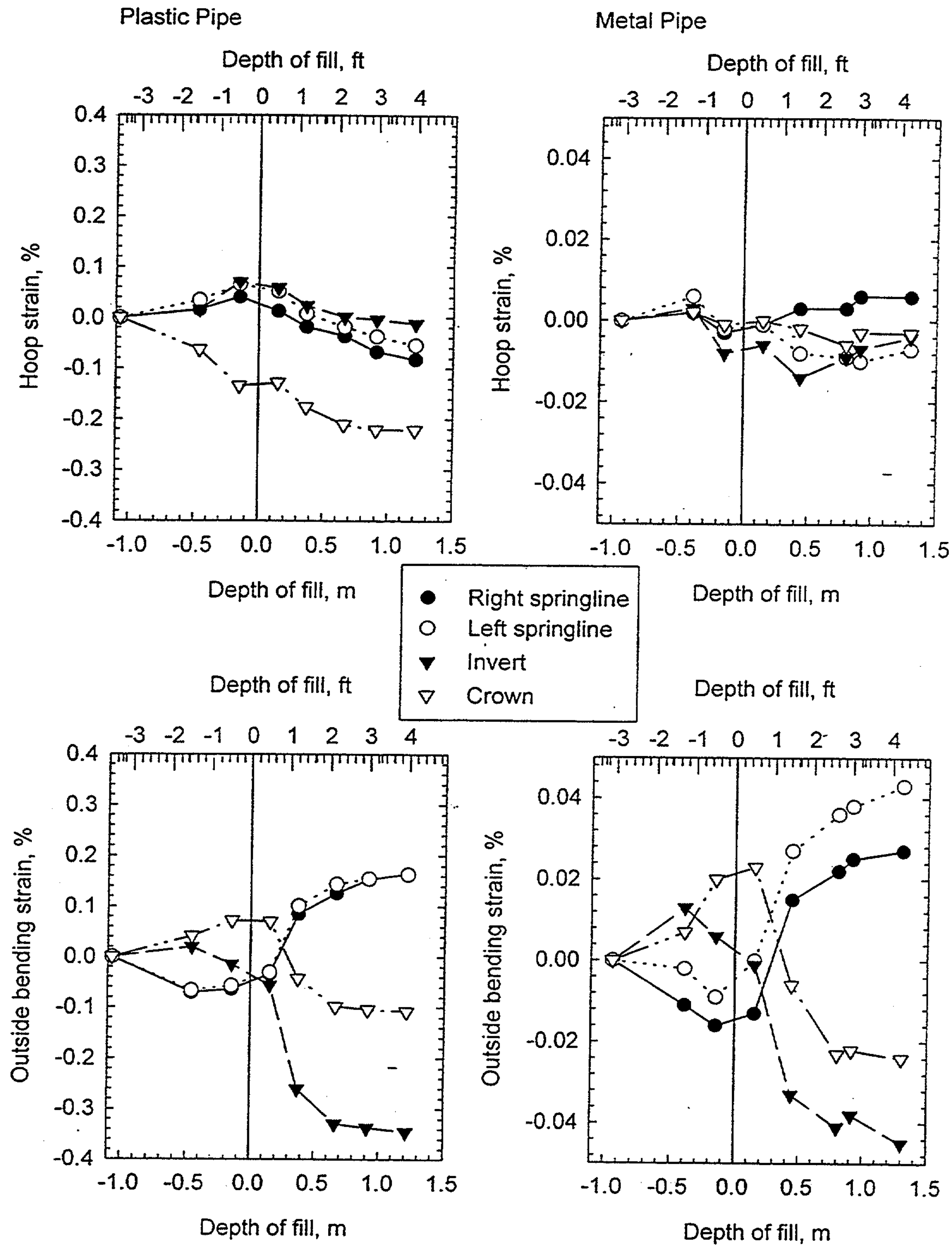
Figs. 4.50 and 4.51 show the hoop and bending strains for the plastic and metal pipe versus depth for tests 6 and 2, respectively. The bending strains, as expected, parallel the deflection plots. The magnitude of the hoop strain in the metal pipe is very small and the data does not appear to be meaningful. The hoop strains in the plastic pipe show a trend of increasing with the depth of fill, at approximately the same rate at the invert, crown and springlines, however the peak occurs at the crown. This higher value at the crown is mostly caused by thrust developed during placement of the sidefill, and thus is not indicative that the crown develops thrust at a higher rate than the springlines because of soil placed over the top of the pipe.

Springline hoop strain, and crown, invert, and springline bending strains for the plastic pipe are presented in table 4.15. Table 4.16 presents similar data for the metal pipe, except that, as noted, the hoop strains are not presented because the data did not appear meaningful. This data will be discussed in more detail in chapter 5.




Note: Test 6 was installed with silty sand backfill in a narrow trench and compacted with the rammer.

**Figure 4.50 Hoop and Bending Strains for Field Test 6**




Note: Test 2 was installed with silty sand backfill in a narrow trench with no compaction.

**Figure 4.51 Hoop and Bending Strains for Field Test 2**

**Table 4.15**
**End of Test Strains – Plastic Pipe**

| Test No. | Compaction and Backfill | Pipe strains, % | | | |
|---|---|---|---|---|---|
| | | Springline Hoop compression | Bending, outside surface (2) | | |
| | | | Springline | Invert | Crown |
| **a. 900 mm (36 in.) Diameter Pipe** | | | | | |
| 1 | Rammer/Stone | -0.058 | -0.060 | -0.050 | 0.184 |
| 3 | Rammer/stone | -0.107 | -0.095 | 0.042 | 0.170 |
| 9 | Rammer/stone | -0.147 | -0.075 | -0.012 | 0.112 |
| 6 | Rammer/silty sand | -0.062 | -0.248 | 0.345 | 0.305 |
| 8 | Rammer/silty sand | -0.055 | -0.296 | 0.172 | 0.285 |
| 4 | V. plate/stone | -0.102 | -0.067 | ND | 0.041 |
| 11 | V. plate/stone | -0.186 | -0.009 | ND | ND |
| 7 | V. plate/silty sand | -0.202 | 0.053 | -0.396 | -0.080 |
| 2 | None/stone | -0.069 | 0.148 | -0.390 | -0.111 |
| 5 | None/silty sand | -0.089 | 0.076 | ND | -0.117 |
| 10 | CLSM | -0.113 | -0.073 | ND | 0.020 |
| **b. 1,500 mm (60 in.) Diameter Pipe** | | | | | |
| 12 | None/stone | -0.155 | 0.084 | ND | -0.013 |
| 13 | V. plate/stone | -0.117 | 0.033 | ND | 0.228 |
| 14 | V.plate/silty sand | -0.116 | 0.006 | ND | 0.248 |

Notes:
1. ND indicates no data, one of the gages did not function properly.
2. Inside bending strain is directly proportional to the outside bending strain, based on the distance from the centroidal axis and is not shown.

**Table 4.16**
**End of Test Strains – Metal Pipe**

| Test No. | Compaction and Backfill | Circumferential bending strain, % | | |
|---|---|---|---|---|
| | | Springline | Invert | Crown |
| **a. 900 mm (36 in.) Diameter Pipe** | | | | |
| 1 | Rammer/Stone | ND | 0.0034 | 0.0075 |
| 3 | Rammer/stone | -0.0258 | 0.0249 | 0.0161 |
| 9 | Rammer/stone | -0.0179 | 0.0016 | 0.0110 |
| 6 | Rammer/silty sand | -0.0333 | 0.0582 | 0.0144 |
| 8 | Rammer/silty sand | -0.0515 | 0.0740 | 0.0302 |
| 4 | V. plate/stone | 0.0078 | -0.0186 | -0.0192 |
| 11 | V. plate/stone | -0.1107 | 0.0041 | ND |
| 7 | V. plate/silty sand | -0.0220 | -0.0780 | 0.0015 |
| 2 | None/stone | 0.0373 | -0.0492 | -0.0246 |
| 5 | None/silty sand | 0.0444 | -0.1143 | -0.0113 |
| 10 | CLSM | -0.0161 | ND | -0.0029 |
| **b. 1,500 mm (60 in.) Diameter Pipe** | | | | |
| 12 | None/stone | 0.003 | -0.042 | -0.024 |
| 13 | V. plate/stone | 0.004 | -0.008 | -0.003 |
| 14 | V.plate/silty sand | -0.003 | -0.028 | 0.007 |

Notes:
1. ND indicates no data, one of the four gages did not function properly.

#### 4.2.6.6 Sidefill Soil Strain

Soil strain gages were installed to measure the change in distance between the springline of the test pipe and the trench wall. Data from these gages for test 3, with rammer compacted stone backfill, and test 5, with uncompacted silty sand backfill, is shown in fig. 4.52, which presents the average displacement from both sides of the pipe. These figures show the following characteristic trends:

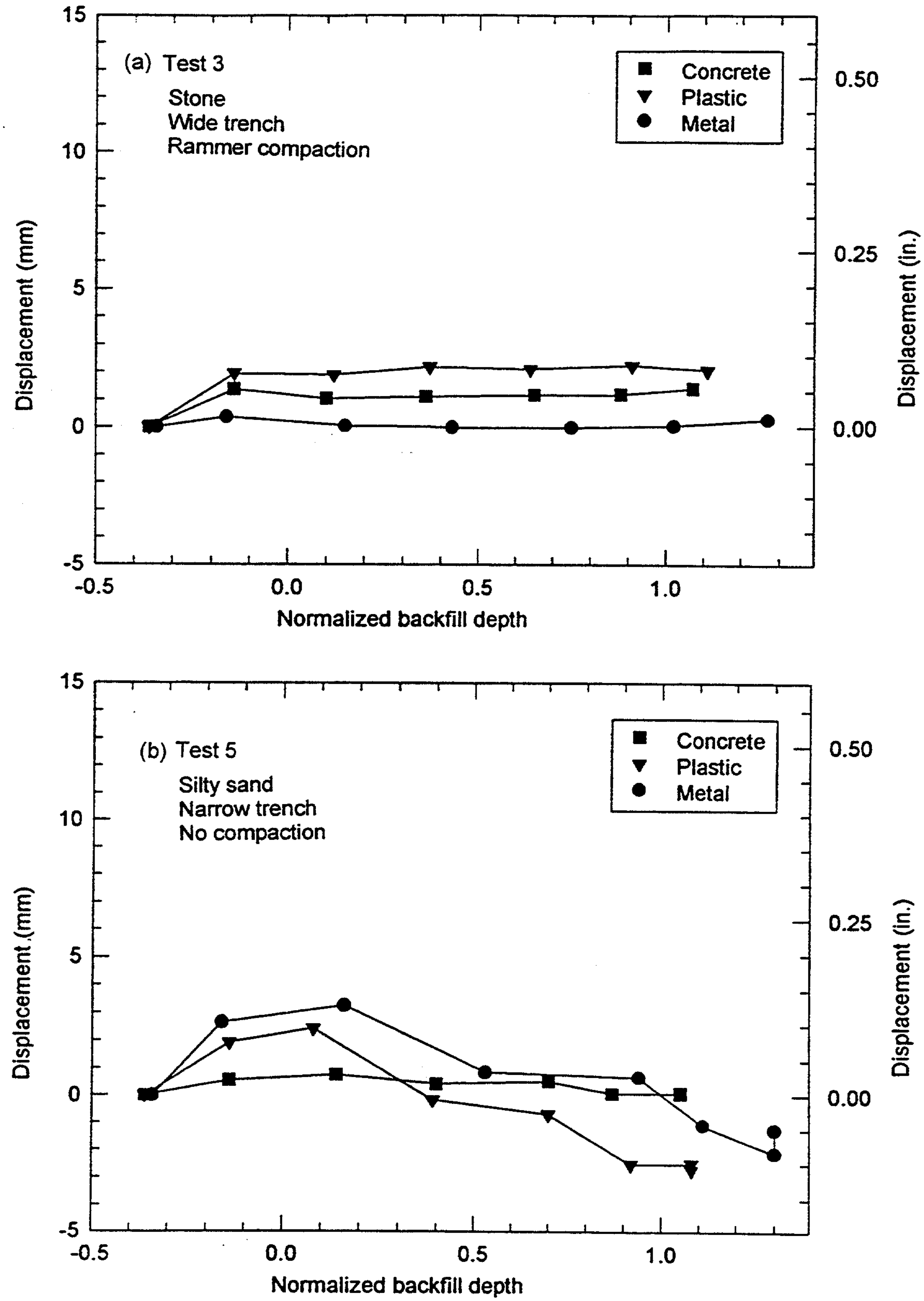
- A substantial part of the extension of the gages occurs during compaction of the first backfill layer after the gages are installed (some of which may be a seating effect as the fill around the gages is compacted);
- For tests with compacted backfill very little displacement occurred thereafter (fig. 4.51(a)); and
- For tests with uncompacted backfill a notable compression occurred as backfill was placed over the crown (fig. 4.51(b)).

Data for the change in width of the soil sidefill during backfilling over the top of the pipe are presented in table 4.17.

**Table 4.17**
**Change in Soil Sidefill Width During Backfilling Over Top of the Pipes**

| Test | In situ soil | Concrete | Plastic | Metal |
|---|---|---|---|---|
| | | mm | mm | mm |
| 1 | sand | 0.1 | 0.2 | 0.0 |
| 3 | sand | 0.4 | 0.2 | 0.2 |
| 9 | clay | 0.5 | 0.5 | 0.5 |
| 6 | sand | -0.5 | -1.4 | -1.0 |
| 8 | sand | gages not installed | | |
| 4 | sand | 2.0 | 1.1 | 0.1 |
| 11 | clay | 1.7 | -0.5 | 0.9 |
| 13 | clay | 0.5 | -0.4 | -0.3 |
| 7 | sand | -1.1 | -2.2 | -1.3 |
| 14 | clay | data erratic | | |
| 2 | sand | data erratic | | |
| 12 | clay | 1.1 | -2.9 | -3.0 |
| 5 | sand | -0.8 | -5.1 | -4.5 |
| 10 | clay | gages not installed | | |

1 mm = 0.04 in.




Note: Positive displacement represents gage extension

**Figure 4.52 Sidefill Soil Displacement During Backfilling**

In general the data from these gages were variable; but when several like conditions were averaged together, trends emerge. Several variables are evaluated in table 4.18.

**Table 4.18**
**Change in Soil Sidefill Width – Grouped by Test Variable**

| Variable | | Concrete | Plastic | Metal | Tests included |
|---|---|---|---|---|---|
| Type | Condition | mm | mm | mm | |
| In situ soil | sand | 0.0 | -1.2 | -1.1 | 1,3,4,5,6,7 |
| | clay | 0.9 | -0.8 | -0.5 | 9,11,12,13 |
| Backfill | stone | 0.9 | -0.3 | -0.2 | 1,3,4,9,11,12,13 |
| | silt | -0.8 | -2.9 | -2.3 | 5,6,7 |
| Compaction | R | 0.1 | -0.1 | -0.1 | 1,3,9,6 |
| | VP | 0.8 | -0.5 | -0.1 | 4,7,11,13 |
| | N | 0.2 | -4.0 | -3.8 | 12,5 |
| Pipe diameter | 900 mm | 0.3 | -0.9 | -0.6 | 1,3,4,5,6,7,9,11 |
| | 1,500 mm | 0.8 | -1.6 | -1.7 | 12,13 |
| Trench width | Narrow | 0.1 | -1.7 | -1.6 | 1,5,6,9,12 |
| | Wd & Int. | 0.7 | -0.4 | -0.1 | 3,4,7,11,13 |
| All data | | 0.4 | -1.0 | -0.8 | |

1 mm = 0.04 in.

The data in table 4.17 can also be combined with the deflection data to evaluate movement of the trench wall. This evaluation was made and indicates that test 11, which was inundated with rain, showed outward trench wall movement of 4 to 6 mm (0.15 to 0.25 in.). This movement undoubtedly resulted from the inundation and explains the higher deflections in test 11 relative to other tests with similar variables. In general, tests where the native soil was sand showed less than 2 mm (0.08 in.) of outward trench wall movement and tests where clay was the native soil showed 1 to 3 mm (0.04 to 0.12 in.) of outward movement. These small movements are unimportant.