CHAPTER 5
ANALYSIS OF TEST RESULTS

Analytical models of buried pipes were evaluated against the field data to investigate the accuracy of the models and then to improve understanding of the physical processes that take place during installation.

5.1 Elasticity Model

The Burns and Richard (1964) elasticity solution was discussed in chapter 2. As noted it is idealized in that it models an elastic ring embedded in an isotropic elastic medium. In some respects this makes it particularly ill-suited to model the field tests because of the use of a trench installation, the shallow cover, and the variable haunch control; however, the model still shows trends that match the data, and are informative to examine.

Analyses were conducted for the field tests using the three 900 mm (36 in.) diameter pipes and the three 1500 mm (60 in.) diameter pipes, with soil properties representing the stone backfill with densities of 95 percent of maximum standard Proctor density (rammer compaction) and 85 percent (no compaction) of maximum standard Proctor density. Based on table 3.6, for an SW material with a vertical soil stress at the springline of about 4 psi, one-dimensional soil moduli, M_s, of 16 MPa (2300 psi) and 3.5 MPa (500 psi) were selected for the compacted and uncompacted conditions respectively. The Burns and Richard model is not capable of evaluating the stresses and deformations that occur while placing backfill at the sides of the pipe, thus the results of the analysis are compared to the changes in deflection, stress and strain that occurred while placing backfill over the top of the pipe. The applied vertical soil stress was 23 kPa (3.3 psi), representing the free field stress at the crown of the pipe at the end of backfilling. Considering the generally warm weather and test durations of several days, the plastic pipe data was converted to thrusts and moments using a modulus of elasticity of 500 MPa (72,500 psi).

Table 5.1 compares the results of the analysis with the Burns and Richard method using the equations for a full-slip pipe-soil interface with field data from Test Nos. 1, 2, 3, and 9.

Table 5.1
Comparison of Burns and Richard Full-Slip Predictions with Field Data for 900 mm (36 in.) Diameter Pipe

a. Deflections and Interface Pressures

Pipe Type	M_s	S_B	S_H	Burns and Richard				Field Data			
				ΔV	ΔH	p-cr	p-sp	ΔV	ΔH	p-cr	p-sp
	(MPa)	Eq. 2.14	Eq. 2.13	(%)	(%)	(kPa)	(kPa)	(%)	(%)	(kPa)	(kPa)
Concrete	3.5	0.1	0.001	-0.008	0.007	40	5.9	--	--	30	0.5
	16	0.6	0.003	-0.008	0.007	40	6.3	--	--	25	0.5
Plastic	3.5	98	0.34	-0.97	0.60	22	17	-1.3	1.3	21	21
	16	450	1.54	-0.31	0.07	13	12	-0.2	0.1	18	12
Metal	3.5	57	0.005	-0.71	0.71	27	19	-1.5	1.3	24	18
	16	260	0.022	-0.19	0.19	24	22	-0.2	0.1	23	15

Notes
1. Field data is change caused by backfilling over top of the pipe for tests backfilled with stone. Field data for M_s = 3.5 MPa is taken from test 2. Field data for M_s = 16 MPa is taken from test 1 (Narrow trench, haunched, sand site), test 3 (Wide trench, haunched, sand site), and test 9 (Narrow trench haunched, clay site).
2. All plastic pipe calculations assume a modulus of elasticity of 500 MPa to account for the temperature and test duration.
3. Plastic and metal pipe interface pressure data taken from soil pressure cells 150 mm over crown and at trench wall.
4. ΔV = change in vertical diameter, ΔH = change in horizontal diameter, p-cr = interface pressure at crown and p-sp = interface pressure at springline.
5. 1 kPa = 6.89 psi, 1 MPa = 145 psi

Table 5.1 (Cont.)
Comparison of Burns and Richard Full-Slip Predictions with Field Data for 900 mm (36 in.) Diameter Pipe

b. Moments, Thrusts, and VAF

Pipe Type	M_s	S_B	S_H	Burns and Richard				Field Data			
		Eq. 2.14	Eq. 2.13	N-sp	M-cr	M-sp	VAF	N-sp	M-cr M-inv	M-sp	VAF
	(MPa)			(kN/m)	(kN-m/m)	(kN-m/m)		(kN/m)	(kN-m/m)	(kN-m/m)	
Concrete	3.5	0.1	0.001	14.82	-1.55	1.49	1.25	–	–	–	–
	16	0.6	0.003	14.72	-1.51	1.45	1.24	–	–	–	–
Plastic	3.5	98	0.337	9.88	-0.219	0.187	0.87	3.5	0.112 0.257	0.154	0.25
	16	447	1.540	6.11	-0.060	0.040	0.54	3.0(T1) 5.5(T3) 7.6(T9)	0.012 0.067	0.057	0.21(T1) 0.39(T3) 0.57(T9)
Metal	3.5	57	0.005	11.39	-0.289	0.288	1.05	–	0.146 –	0.162 0.171	–
	16	261	0.022	10.84	-0.077	0.076	1.00	–	0.016 0.081	–	–

Note 1. All plastic pipe calculations assume a modulus of elasticity of 500 MPa to account for the temperature and test duration.
2. Field data for M_s = 3.5 MPa is taken from test 2. Field data for M_s = 16 MPa is taken from test 1 (Narrow trench, haunched, sand site, called T1), test 3 (Wide trench, haunched, sand site, called T3), and test 9 (Narrow trench haunched, clay site, called T9).
3. Due to symmetry in Burns and Richard solution, M-cr = M-inv
4. 1 kN/m = 5.71 lb/in., 1 kN-m/m = 225 ft-lb/ft

Results from the field tests are only differentiated when significant differences are present. The table indicates that the predictions are in general agreement with the trends shown in the field data. The main observations are:

- The Burns and Richard analysis shows almost no change of bending moment, thrust, or deflection in the concrete pipe as a result of the change in soil stiffness. This is anticipated as the concrete pipe is so stiff, both in bending and in hoop compression that the soil stiffness change from 3.5 to 16 MPa (500 to 2400 psi) is not significant.

- For the concrete pipe, the measured interface pressures are lower than the Burns and Richard predictions. This is believed to be the result of the trench installation, which would reduce the vertical load on the pipe and greatly reduce the lateral pressure.

- The measured interface pressures for the metal pipe and plastic pipe are in reasonable agreement with the predicted pressures.

- Predicted vertical soil pressure near the top of on the plastic pipe are relatively uniform for both soil conditions. The measured data is uniform for the loose soil condition but less so for the dense soil condition. The vertical pressure measurement for the plastic pipe was taken at 150 mm over the top of the pipe, which could have resulted in a more nearly geostatic stress than would exist closer to the pipe.

- The predicted deflections for the metal and plastic pipe embedded in compacted soil are in good agreement with the measured deflections.

- The predictions for deflection in loose soil underestimate the measured values for both the metal and the plastic pipe. This may represent the result of the lack of haunching, which Burns and Richard cannot model, or indicate that the dumped backfill leaves voids that allow greater deformation when the first lifts of backfill are placed. Data on deeper installations would be required to evaluate this.

- The field data for thrust in the plastic pipe, appears to be affected by several factors. Lowest thrust was measured in the dense stone in a narrow trench in the sand in situ soil (test 1). Only slightly higher thrusts were measured in the loose stone in a narrow trench in sand in situ soil (test 2). Much higher thrust was measured in the dense stone in the wide trench in sand in situ soil (test 3) and still higher values were measured for the dense stone in a narrow trench but in the clay in situ soil (test 9). In all cases, the field vertical arching factors are less than the Burns and Richard predictions. As noted in Section 4.2.6.5, the metal thrust strains were not analyzed.

- Measured bending moments are variable relative to the Burns and Richard solution. The crown moments are substantially lower than the invert moments, which is expected because of the haunching effect. Invert moments are on approximately the same order of magnitude as the Burns and Richard for the plastic and metal pipe. Measured springline moments for the metal pipe are much lower than predicted, while for the plastic pipe the measured moments at the springline are somewhat lower than predicted in the loose soil and higher than predicted in dense soil. The low springline moments may be the due the influence of the trench walls. The overall match of measured to predicted moments is actually a little surprising for the loose soil, since the deflections were under predicted.

Overall, the match between the Burns and Richard predictions and the measured data is quite good considering the idealized model and the uncertain approximations, such as the estimated modulus of elasticity; however, the predictions pertain only to the changes in behavior due to backfilling over the top of the pipe.

5.2 Computer Analysis of Field Test Results

Analysis of the field tests was undertaken with CANDE, Level 3. Complete finite element meshes were developed to represent the installation conditions of the tests.

The finite element meshes for analysis of the 900 mm diameter and 1,500 mm diameter pipe installations are shown in figures 5.1 and 5.2, respectively, which also show the boundaries of the trench and various soil zones. Descriptions of the soil zones are provided in table 5.2. The same mesh was used for both the narrow and wide trench installations by changing element assignments from in situ soil to backfill as shown in the figures. Symmetry was assumed about the vertical centerline of the pipe. The pipe was divided into 20 segments, each segment extending for an arc length of nine degrees.

Undisturbed in situ soils were modeled with estimated linear elastic properties while placed soils were modeled with non-linear behavior using the Duncan (1970) hyperbolic Young's modulus with the Selig (1985) hydrostatic hyperbolic bulk modulus. The *CANDE User Manual*, Appendix A, (CANDE, 1989) contains two sets of Selig bulk modulus properties, called the "modified," which are the defaults, and the "hydrostatic," which must be input manually. Based on the evaluation in chapter 3, the hydrostatic properties were used for the analyses reported here. Soil properties and compaction levels used to model the various soil zones are summarized in table 5.3. Although the field tests were conducted to a depth of 1.2 m (4 ft) over the test pipe, the analyses were continued to a depth of 6.1 m (20 ft) to investigate implications of the various installation conditions under more demanding loading conditions.

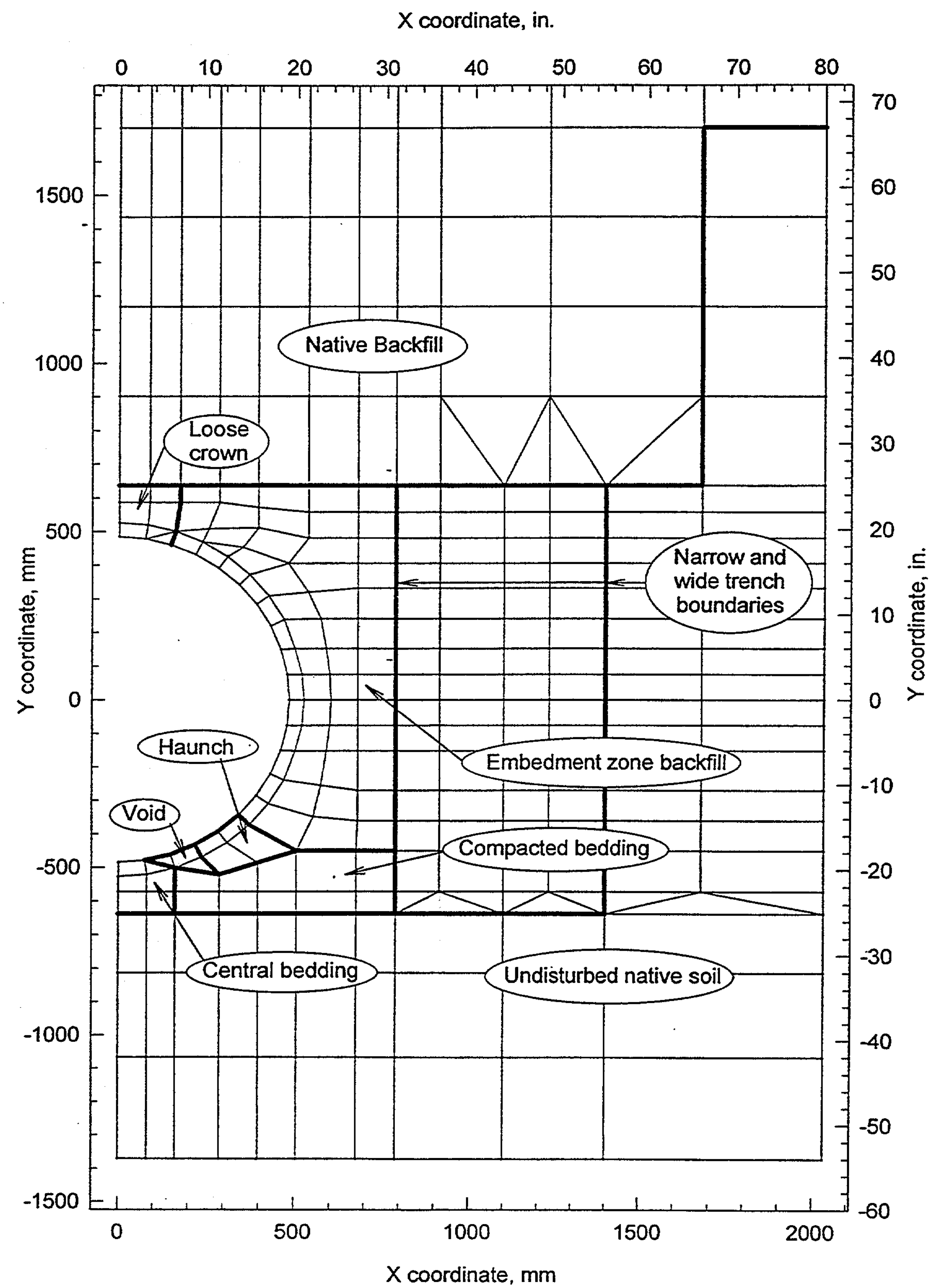


Figure 5.1 Soil Zones for 900 mm (36 in.) Diameter Plastic Pipe

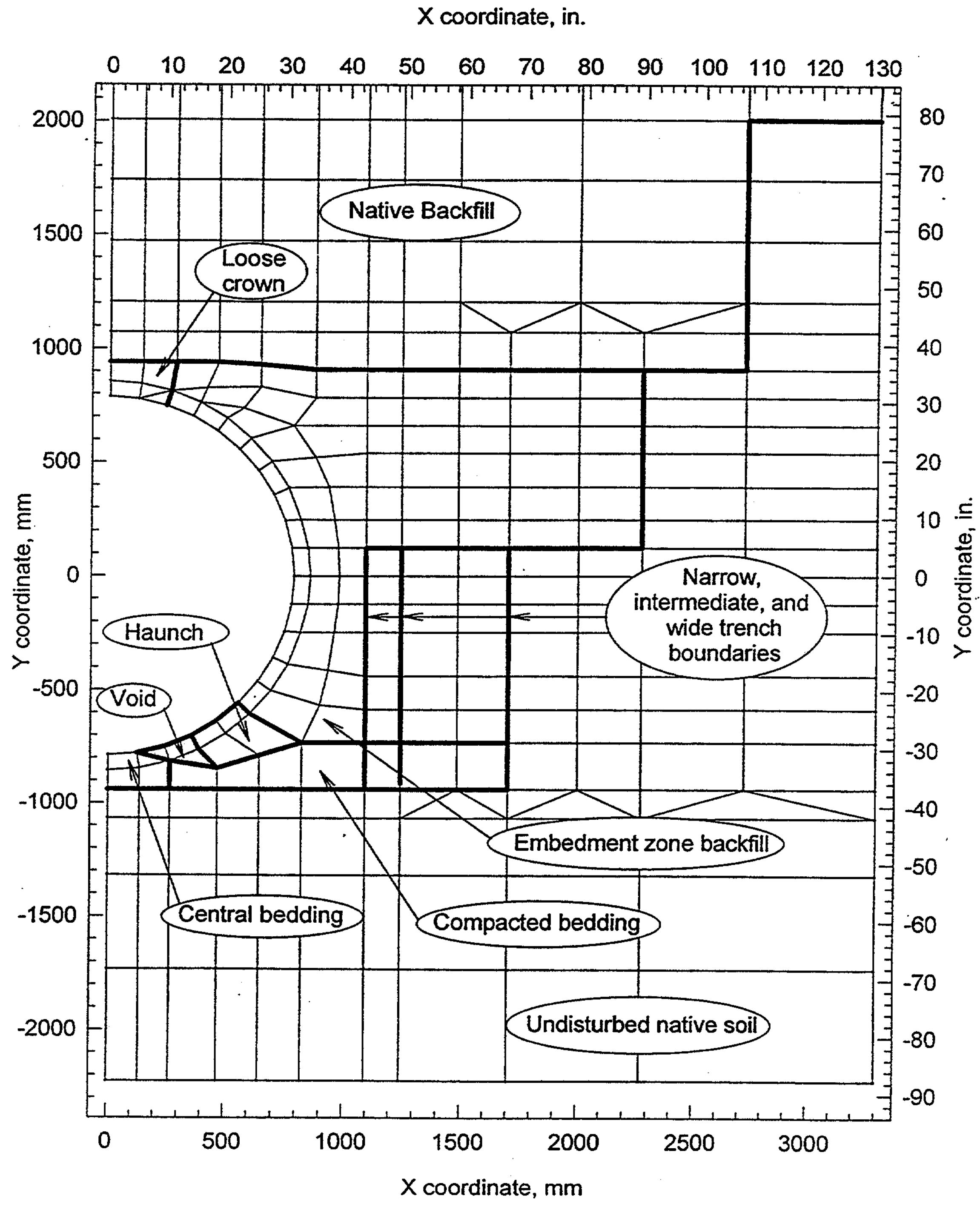


Figure 5.2 Soil Zones for 1500 mm (60 in.) Diameter Pipe

Table 5.2
Soil Zones Used in FEM Analysis of Field Installations

Soil Zone Label in Figures 5.1 and 5.2	Zone Description
Undisturbed native soil	Natural soil formation, sand or clay
Compacted bedding	150 mm deep layer of compacted backfill
Central bedding	150 mm deep, 300 mm wide layer of backfill, loose or compacted as required for specific tests
Void	Loose soil (ML49) under all conditions, even when haunching was specified
Haunch	Compacted backfill material if haunching was specified, otherwise loose backfill material
Embedment zone fill	Backfill material with properties based on achieved density
Loose crown	Backfill material with properties of loose soil
Native backfill	Compacted native backfill material

Table 5.3
Soil Properties Used in FEM Analysis

Common Name		Compacted Density (1)		Soil Model	CANDE Designation or Young's Modulus (MPa) (2)
		%	kN/m^3		
Undisturbed native sand		–	–	linear elastic	28
Undisturbed native clay		–	–	linear elastic	7
Compacted native soil	sand	96	20.1	hyperbolic	SW95
	clay	90	18.7	hyperbolic	CL90
Loose stone		79	17.9	hyperbolic	SW80
Stone compacted with vibratory plate		85	19.3	hyperbolic	SW85
Stone compacted with rammer		92	20.7	hyperbolic	SW90
Loose silty sand		82	13.0	hyperbolic	ML80
Silty sand compacted with vibratory plate		89	14.3	hyperbolic	ML90
Silty sand compacted with rammer		95	15.4	hyperbolic	ML95

Notes:
1. The compacted density is reported as the average percentage of maximum dry density, per AASHTO T 99, measured in the field, and as the wet density measured in the field.
2. Selig soil properties include the hydrostatic bulk modulus values.
3. 1 MPa = 145 psi, 1 kN/m^3 = 6.4 pcf

5.2.1 Modeling of Construction Effects During Sidefill

Modeling pipe-soil interaction while placing the sidefill requires a method to introduce compaction effects. Compaction effects are the pipe deformations and interface pressures that result from the process of bringing backfill soil from the loose state at which it is placed to its final density. The soil-culvert interaction that takes place during this stage of construction can be significant; however, the hyperbolic soil models available in CANDE were not developed to address this load condition. CANDE was tested to evaluate several methods of modeling compaction effects, without program technical changes, and to provide guidance to pipe designers who must use available software packages. Three approaches were taken in this effort:

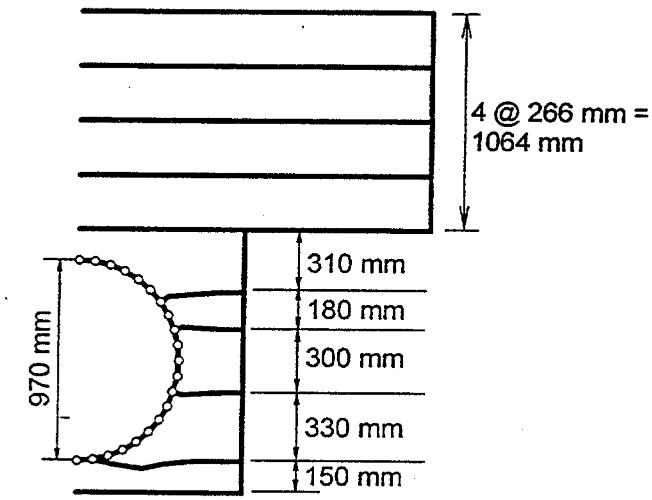
1. Applying vertical loads to the surface of the just placed layer of backfill;

2. Squeezing the most recently placed layer of backfill between vertical upward and vertical downward forces; and

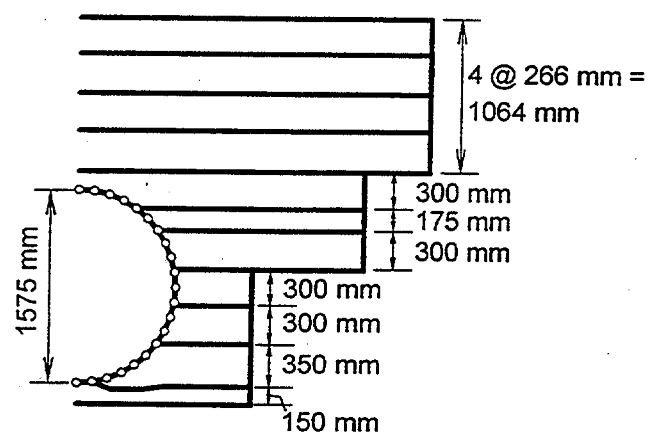
3. Applying horizontal nodal forces directly to the pipe.

Methods 1 and 2 have the advantage of creating pipe distortion and movements as a result of the pipe-soil interaction that takes place as a consequence of forces applied by a compactor. However, when using an elastic soil model, removing the compaction force results in a rebound of the pipe. Also, to correctly model the compaction problem, the model should start with the properties of a loose soil, having a low strength and stiffness, and finish with the properties of a compacted soil. Yet, again, the hyperbolic soil model was not developed to provide this transition from significantly different states of soil density, nor can it simulate the cumulative deformations that result from successive passes of the compactor. Efforts at using Methods 1 and 2 were unsuccessful in creating deformations representative of those in the field, and in general were unsuccessful in creating any significant peaking effects.

Method 3 is the least sophisticated of the three techniques in that it requires a separate algorithm or chart to provide guidance on the magnitude of the forces to be applied. Key variables in this are the soil friction angle, the size and type of compactor, and the size of the pipe. Nodal forces were applied to represent the placement of layers of backfill, as they were in the actual field tests. The distribution of the nodal forces assumed that the compaction pressures were of uniform magnitude for a depth of 300 mm (12 in.) below the soil surface. This is demonstrated in figs. 5.3 and 5.4 for both pipe sizes.



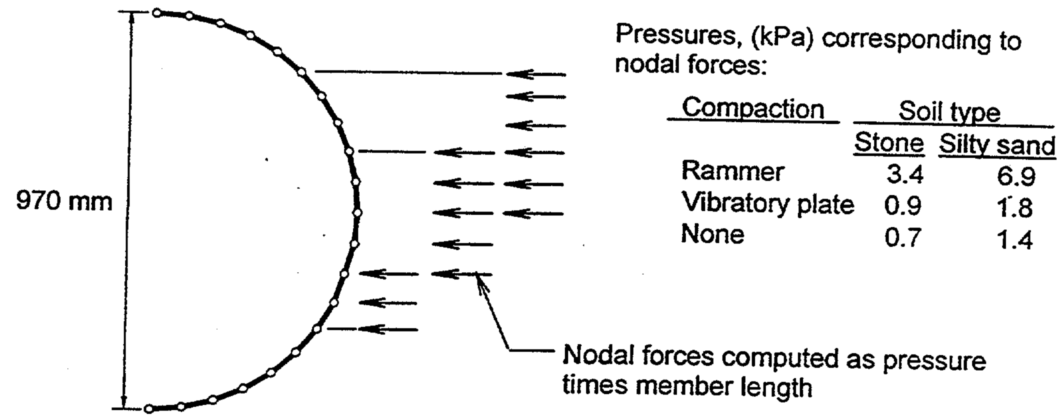
a. 900 mm Diameter Pipe in Narrow Trench
(Increments for the wide trench were at the same elevations)



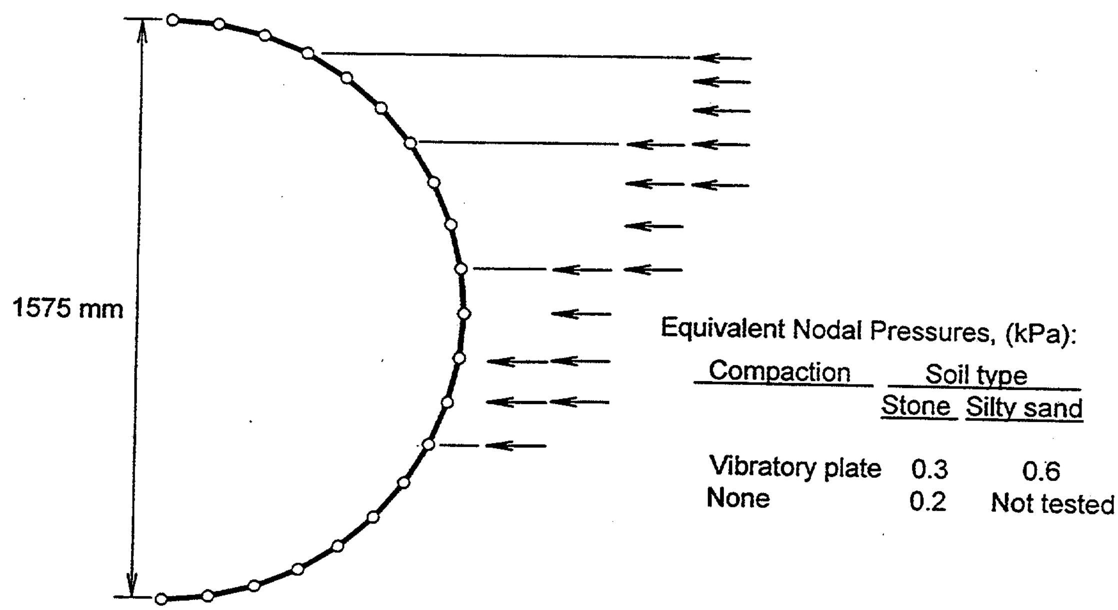
b. 1500 mm Pipe in Narrow Trench
(Increments for intermediate and wide trench were at the same elevation)

1 in. = 25.4 mm

Figure 5.3 Construction Increment Thicknesses for Field Tests



a. 900 mm Diameter Pipe



b. 1500 mm Diameter Pipe

Note: 1psi = 6.89 kPa

Figure 5.4 Application of Nodal Forces to Model Compaction Effects

For the metal and plastic pipe, the analysis showed that, for a given type of soil, compaction, and pipe size, the forces required to match the field deflections were consistent. Although the modeling was completed using concentrated nodal forces, equivalent pressures were calculated to assist in comparison of the two pipe sizes. The pressures that best matched the field deflections for each combination of parameters are presented in table 5.4.

Table 5.4
Applied Pressures (kPa) to Represent Compaction Effects

Soil Type	Compaction Type/ Pipe Diameter (mm)				
	Rammer	Vibratory Plate		None	
	900	900	1500	900	1500
Stone	3.4	0.9	0.3	0.7	0.2
Silty sand	6.9	1.8	0.6	1.4	--

1.0 psi = 6.89 kPa

Table 5.4 shows that the compaction pressures are twice as great for the silty sand as for the stone, and substantially smaller for the 1,500 mm (60 in.) pipe than for the same type of compaction for 900 mm (36 in.) pipe. Pressures that model the vibratory plate are only slightly larger than those for no compaction.

It is interesting to note the relatively small pressures required in the CANDE model to produce the observed field peaking effects. Part of this is because CANDE is a two-dimensional model, thus the model represents compaction forces applied to an infinite length of the pipe, all at the same time. In the real three-dimensional world, the compaction forces spread longitudinally away from the compactor location and a length of pipe greater than the loaded portion resists the applied load, thus, the concentrated load to cause the observed peaking would be greater than the force in the two-dimensional model.

A simple expression was developed based on the above pressures to predict the compaction pressures under other conditions. The expression assumes that the lateral pressures on the pipe are related to the at-rest lateral pressure of the soil, which is computed as the vertical stress times 1-sin ϕ, where ϕ is the friction angle of the soil in a loose condition. Values of ϕ were selected from the CANDE User Manual, Appendix A, from the Selig "hydrostatic" soil properties. The resulting expression (which is only developed in SI units) is:

$$np = 1.3\,P\,(1 - \sin\phi)^3 \left(\frac{970}{dc - 250} \right)^2 \tag{5.1}$$

where

np	=	nodal pressure used in CANDE model, kPa,
P	=	total compactor force, kN (not less than 4 kN to account for gravity effects of backfill),
ϕ	=	friction angle of soil in loose condition, degrees, and
dc	=	centroidal diameter of pipe, mm.

Table 5.5 compares the nodal pressures predicted by the Eq. 5.1 with the pressures actually used in the CANDE analyses.

The equation was developed based on limited data but suggests several items to consider when selecting compaction equipment and backfill:

- The lateral force applied to a pipe is sensitive to the friction angle as indicated by the fact that the compaction of the silty sand, with a loose friction angle 8 degrees lower than that of the stone, resulted in twice the compaction effect;
- Required compaction pressure drops significantly with increasing diameter; and
- The vibratory plate, which densifies soil by vibration, rather than by impact like the rammer, produces only slightly more compaction deflection than the gravity weight of the soil (remember, however, that the rammer produced about 5 percent greater density, per AASHTO T-99 for the same number of passes).

Table 5.5
Computed and Applied Nodal Pressures

Compactor		Diam. (mm)	Soil		Nodal pressure (kPa)	
Type	Force (kN)		Type	ϕ (degrees)	Eq. 5.1	CANDE analysis
Rammer	20.5	900	stone	36	3.4	3.4
			silty sand	28	7.2	6.9
Vibratory plate	5.2	900	stone	36	0.9	0.9
			silty sand	28	1.8	1.8
		1,500	stone	36	0.3	0.3
			silty sand	28	0.5	0.6
None	4.0	900	stone	36	0.7	0.7
			silty sand	28	1.4	1.4
		1,500	stone	28	0.2	0.2

Note: 1 lb = 0.454 kg, 1 in. = 25.4 mm, 1 psi = 6.9 kPa,

5.2.2 Results

The CANDE analyses predicted behavior during backfilling that is in substantial agreement with the results of the field tests. There are some notable exceptions that will be discussed below. The deflections, moments, thrusts, and shears in the pipe wall, and interface pressures for each analysis are presented in appendix A. Summary plots are presented here.

5.2.2.1 Deflections

The match between the field test data and the CANDE analyses can best be investigated by comparing the plots of deflection versus depth of fill. This comparison is presented in figs. 5.5, 5.6 and 5.7 for tests with (1) the 900 mm (36 in.) diameter plastic pipe with soil backfill; (2) the 900 mm (36 in.) diameter metal pipe with soil backfill; and (3) the 900 mm (36 in.) diameter pipe with CLSM backfill and all 1500 mm diameter pipe, respectively. These figures generally show that the peaking deflection during sidefilling and the deflection due to overfill are modeled quite well with the CANDE analyses.

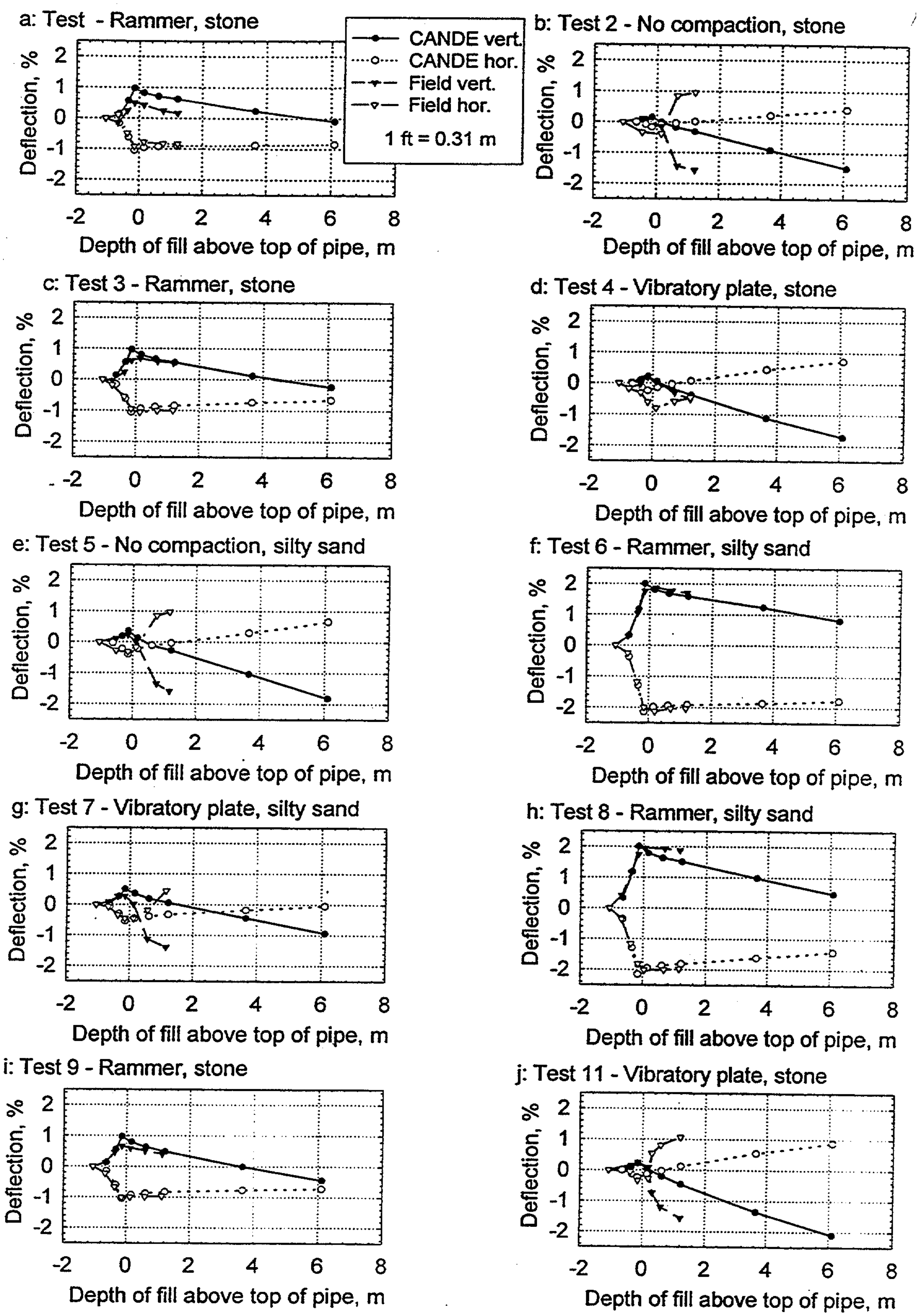


Figure 5.5 CANDE Deflection Compared to field Deflection for 900 mm (36 in.) Diameter Plastic Pipe (except CLSM)

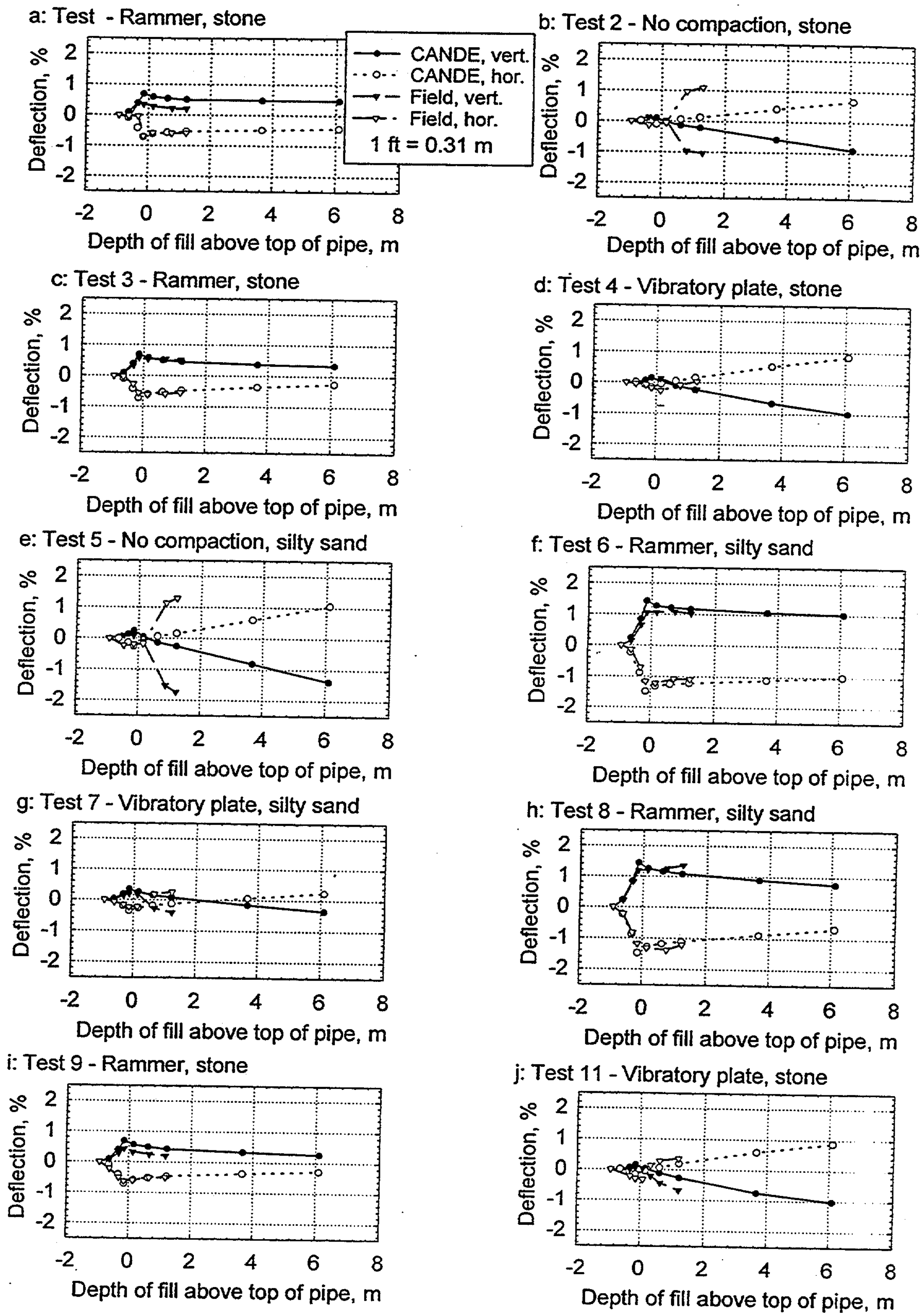


Figure 5.6 CANDE Deflection Compared to Field Deflection for 900 mm (36 in.) Diameter Metal Pipe (except CLSM)

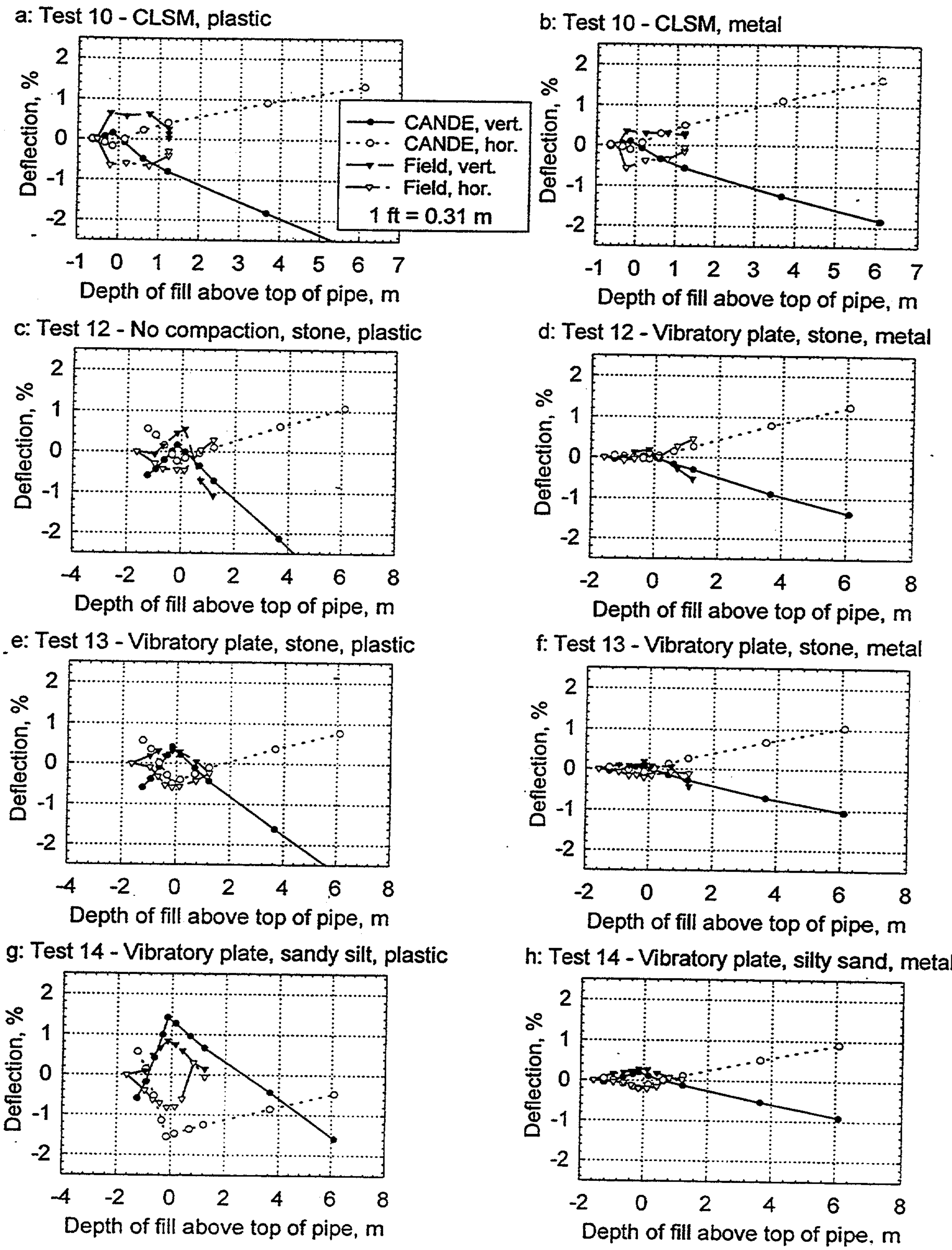


Figure 5.7 CANDE Deflections Compared to Field Deflections for CLSM Test with 900 mm (36 in.) Diameter Pipe and All Tests with 1500 mm (60 in.) Diameter Pipe

For tests 2 and 5 on 900 mm diameter pipe with uncompacted backfill, the field data show an increase in deflection of about 1 percent for the first lifts of backfill over the pipe, up to about 600 mm (24 in.) over the pipe, as shown in figs. 5.5b, 5.5e, 5.6b, and 5.6e. For the last two lifts, from 600 mm to 1,200 mm over the top of the pipe, the rate of change of deflection is closer to that predicted by the CANDE analyses. The effect, evident with both stone and silty sand backfill, is thought to be the result of the large void resulting from a lack of haunching effort and smaller voids that remain from backfill placement and do not get collapsed because no compactive effort is applied. This could be considered a seating effect. When backfill is compacted, it is pushed into intimate contact with the pipe and the trench wall, and voids in the backfill are eliminated. If the backfill is not compacted, then these voids are eliminated during overfilling and result in a significant deflection increment. This effect is apparent for the plastic pipe in test 12 (1,500 mm, fig. 5.7c) but not for the metal pipe. Test 12 was haunched, and the effect may also be less apparent because the trench is relatively narrow (pipe diameter to trench width ratio of 0.7 for test 12 versus 0.6 for tests 2 and 5) and the stiff trench walls may have a greater effect.

In test 7, the plastic pipe deflections, fig. 5.5g, also increased more during placement of fill over the top of the pipe. Test 7 was backfilled with silty sand, compacted with the vibratory plate to 90 percent of maximum standard Proctor density, but no haunching effort was applied. Test 4 (figs. 5.5d), with the same test variables except that the backfill was stone did not show this effect. The silty sand is uniform, relatively fine grained and very sensitive to moisture content, as evidenced by the saturation and loss of bedding compaction in test 5 (see section 4.2.5.3) that was remedied by introducing a bedding layer of coarser sand. The sensitivity to moisture and the presence of voids due to lack of haunching may have permitted the backfill to deform, and drop in average density as fill was placed over the top of the pipe. The stone backfill of test 4 would be more stable under moist conditions. This effect was readily evident in the plastic pipe, which has deep corrugations that do not get filled near the invert. The metal pipe, which has less prominent corrugations, shows the same effect but with a lower magnitude.

The plastic pipe in test 11, fig. 5-5j, showed a higher deflection trend than predicted by the CANDE analysis or as seen in the metal pipe, Fig. 5-6j. This test was inundated during construction when the backfill was at a level about 450 mm over the top of the pipe, and construction was halted for about 1 week. Even though the clay in situ soil was relatively stiff during excavation in the dry it became soft when wet and could and have deformed during the delay. This is the test where the most trench wall movement was recorded by the soil strain gages (see section 4.2.6.6). The same trend was not noted in the metal pipe. This may be because the metal pipe is substantially stiffer under long term loads than the plastic pipe.

The pipe in test 10, figs. 5-7a and 5-7b, showed peaking effect during the placement of the CLSM which was not modeled well by the assumptions used in the CANDE analysis. The hydrostatic nature of the loading is somewhat different from the horizontal loads applied. Undoubtedly, with additional data, a method of modeling this peaking could be developed.

Other observations related to the deflection comparison include:

- The CANDE predictions of deflection due to backfill over the top of the pipe generally match the field deflection quite well. This suggests that the Selig hydrostatic properties are an appropriate design choice.
- For the plastic pipe, the vertical deflection decreases with increasing depth of fill over the pipe at a greater rate than the horizontal diameter increases, while for the metal pipe the vertical and horizontal diameter change at approximately the same rate. This trend, apparent in both the field data and the CANDE analyses, suggests that the plastic pipe is shortening circumferentially due to the low hoop stiffness.
- The CANDE analysis indicates that the 1500 mm diameter plastic pipe deflects about 0.5 percent under its own weight. This was not evident in any of the other tests, but the 1,500 mm plastic pipe was about 10 times less stiff than the 900 mm diameter plastic pipe or either of the steel pipe. Field data were not taken to monitor this effect.
- Related to the previous observation, while the peak deflection that developed in the CANDE model for this pipe reasonably matched the measured peak deflection, the CANDE model actually produced far too much peaking effect that is partially obscured because of the initial downward deflection caused by self weight. The Spirolite type of profile wall may mobilize a greater length of pipe than the corrugated profiles.

5.2.2.2 Interface Pressures

The CANDE vertical and horizontal pressure distribution against the concrete pipe for tests 1 and 2 are shown in figs. 5.8 and 5.9, respectively. These figures show the principal characteristics of all of the figures in appendix A.

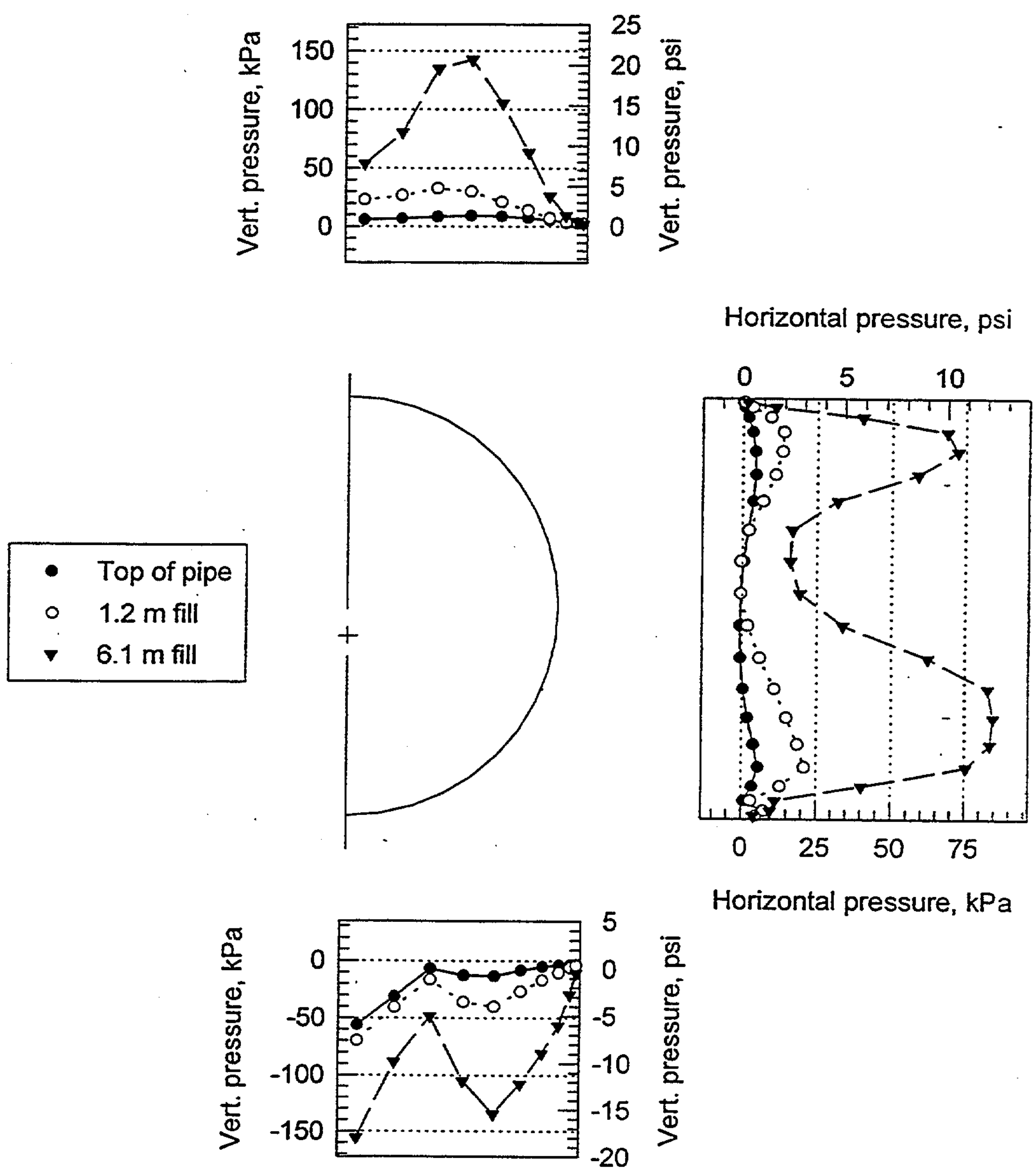


Figure 5.8 Vertical and Horizontal Pressures on Concrete Pipe, CANDE Analysis Test 1 – Rammer Compaction, Compacted Bedding, Haunching, Stone Backfill

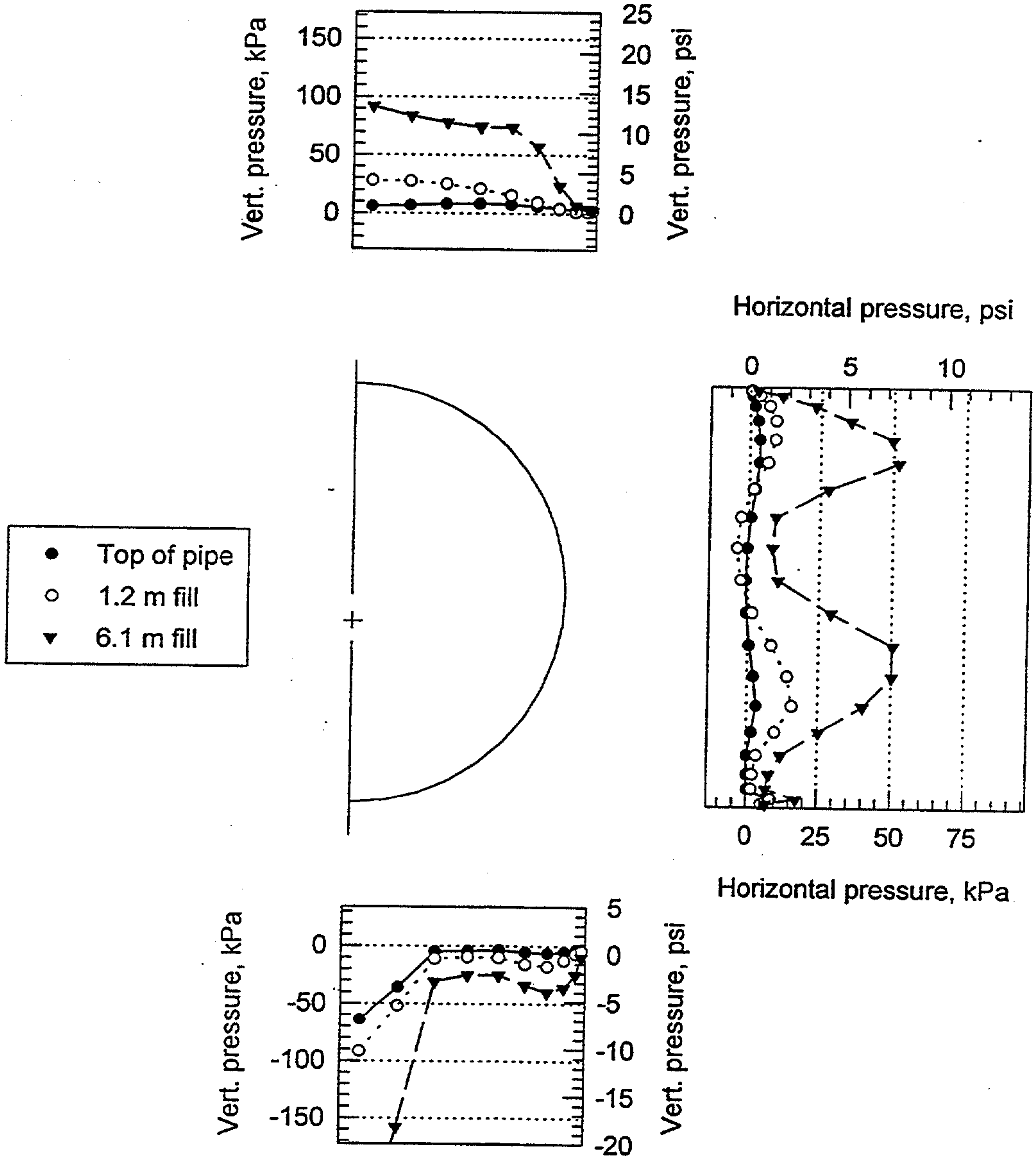


Figure 5.9 Vertical and Horizontal Pressures on Concrete Pipe, CANDE Analysis Test 2 – No Compaction, Compacted Bedding, No Haunching, Stone Backfill

Results for test 1, which was backfilled with stone, compacted with the rammer, and haunched are shown in fig. 5.8. The vertical upward pressure distribution at the bottom results from the assumption of a void, even though haunched. This was borne out in the field tests by the low interface pressures measured at thirty degrees from the invert and the low penetration resistance measured after removal of the pipe. The vertical pressure distribution at the top of the pipe is relatively uniform at 1.3 m of cover, but shows a significant drop at 6.1 m of cover. This is apparently the result of not compacting directly over the pipe. The side pressure at the invert is low at all stages of backfilling; however significant pressures develop just above and below the springline. These are only changes in pressure caused by fill over the crown, because the CANDE analysis did not model compaction pressures.

Results for test 2, which was backfilled with stone, without compaction and without haunching are presented in fig. 5.9. The upward vertical pressure distribution at the bottom of the pipe is peaked at the invert and does not develop the secondary pressure at the side of the pipe. This results from the lack of side support and haunching effort. At the top, the vertical downward pressure distribution is uniform at all depths. For test 2 without compaction, all of the backfill over the pipe is of uniform density and this is reflected in the pressure distribution. The lateral pressure distribution at the side of the pipe is similar to that in test 1, but lower in magnitude.

Measured interface pressures and soil stresses at the trench wall and 150 mm over the crown for the concrete pipe are compared to the CANDE predictions in fig. 5.10. The data presented are the changes in interface pressure as the backfill was placed and compacted from an elevation 150 mm (6 in.) above the pipe, called the top of the pipe, to 1.2 m (4 ft) above the pipe, called the end of test.

The CANDE predictions for invert interface pressure against the concrete pipe are consistently low relative to the field measured values, and the disparity increased as the compactive effort decreased (rammer, vibratory plate, none). The highest field change in invert pressure occurred in tests 2 and 12 which had compacted stone bedding, no haunching, and no compaction. Pressures were closer to the field values as the installation quality improved.

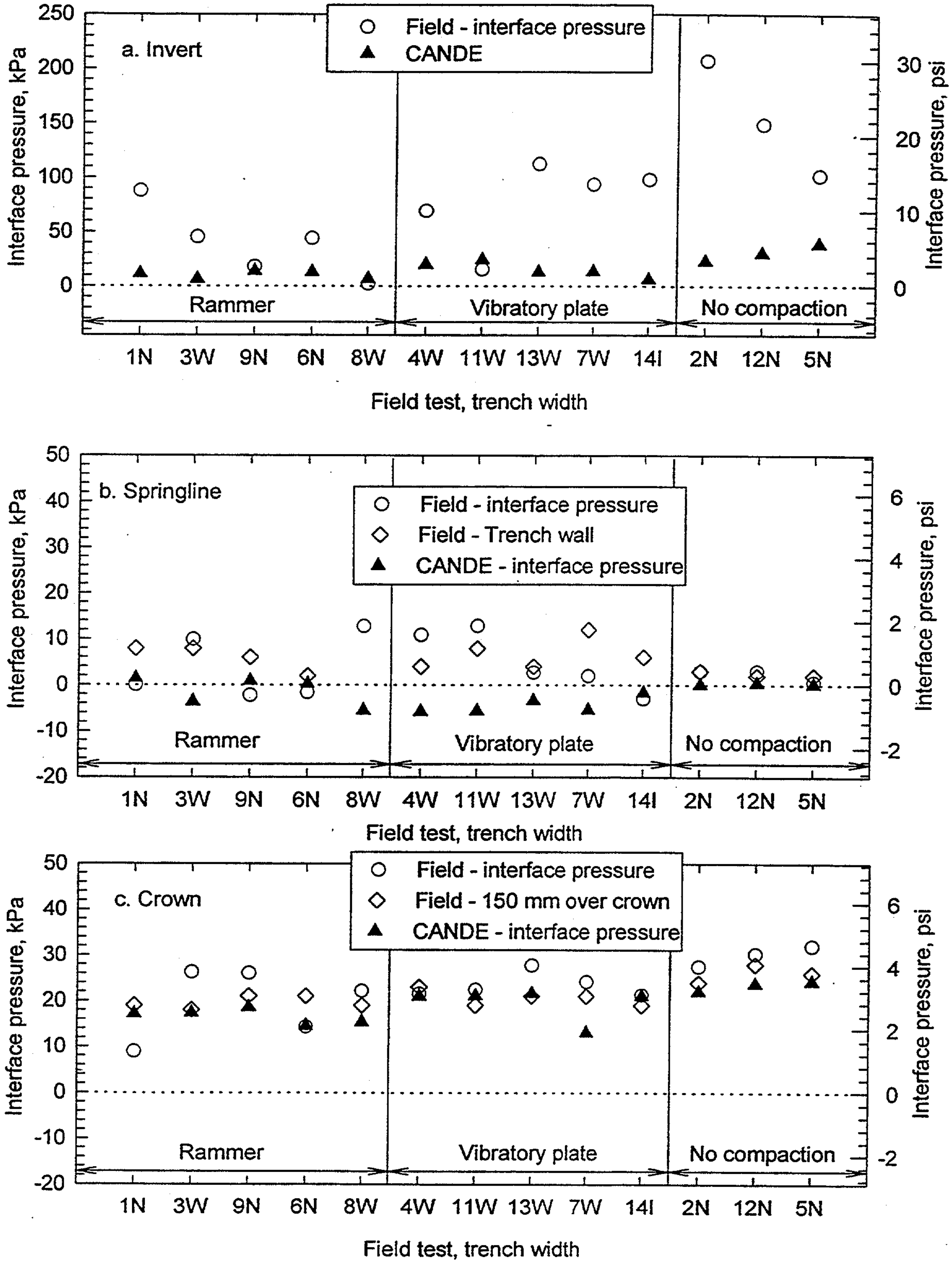


Figure 5.10 CANDE Interface Pressures Compared to Field Pressures for Concrete Pipe

Interface pressures at the springline were quite low in both the CANDE analyses and the field data. The larger pressures developing above and below the springline, as shown in figs. 5.8 and 5.9 indicate that the backfill is arching between the pipe and the trench wall, and little load travels directly through the backfill at the springline.

Measured interface pressures at the crown of the concrete pipe were similar to those predicted by CANDE.

The interface pressures calculated with CANDE for the plastic and metal pipe for test 5 with sandy silt backfill, no compaction, no haunching and compacted bedding (the saturation of the silty sand bedding may have resulted in a softening of the bedding) are presented in fig. 5.11. The pressures for the metal pipe were similar and, for clarity, are only shown at a depth of 6.1 m (20 ft). The trends are similar to the those for the concrete pipe for the vertical pressures at the top and bottom; however, at the side, substantially more pressure develops for the flexible plastic and metal pipe than did for the rigid concrete pipe. The pressure is greatest below the springline. The same information for test 8, with sandy silt backfill, rammer compaction, haunching and soft bedding, is presented in fig. 5.12. The effect of the soft bedding in reducing the invert pressure and increasing the vertical pressure at the side of the pipe is significant. Also of note is that the lateral pressure for test 8 is of a higher magnitude and more centered on the springline than was the case for test 5. Similar plots for all the metal and plastic pipe tests are included in appendix A. The appendix figures plot actual data against the CANDE predictions.

Interface pressure predictions for all flexible pipe tests are compared with CANDE predictions in fig. 5.13. The field data are slightly higher than the predicted data, but the trends with test variables are quite consistent. In fig. 5.13 the field test data are actually taken from the gages installed 150 mm (6 in.) over the crown and at the backfill-trench wall interface. This difference in location from the predictions of pressure at the actual interface by CANDE could account for some of the mismatch between the data and the predictions. In general the lateral pressures are of relatively constant magnitude, even though the deflection varied considerably, upward in some cases and downward in others. This shows that the lateral pressures required to carry a given load is about constant and the pipe will deflect until that pressure develops. This emphasizes the importance of compaction to provide stiff soil and control deflection levels.

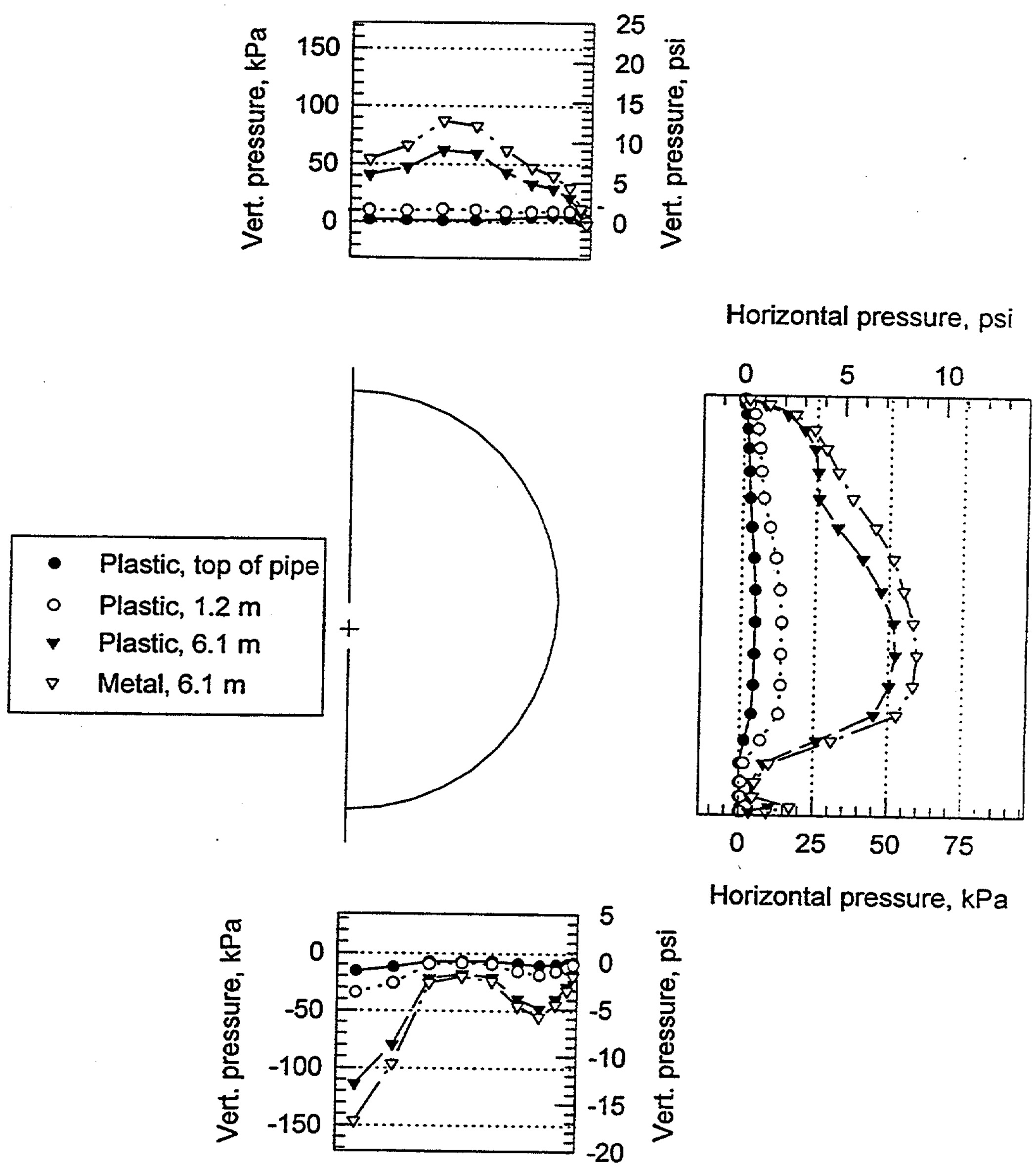


Figure 5.11 Vertical and Horizontal Pressures on Plastic and Metal Pipe, CANDE Analysis, Test 5 – Rammer Compaction, Soft Bedding, No Haunching, Sandy Silt Backfill

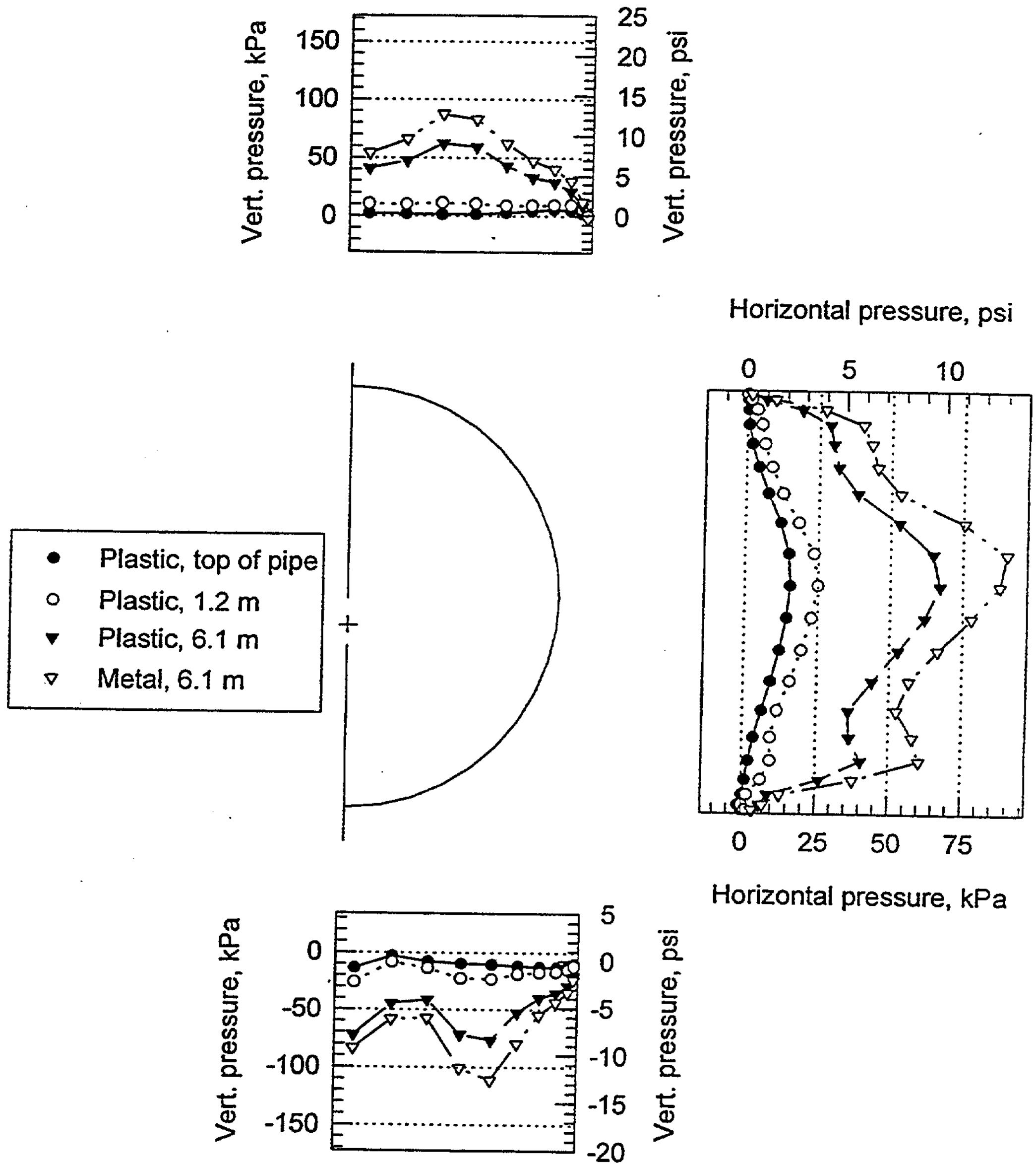


Figure 5.12 Vertical and Horizontal Pressures on Plastic and Metal Pipe, CANDE Analysis, Test 8 – Rammer Compaction, Soft Bedding, Haunching, Sandy Silt Backfill

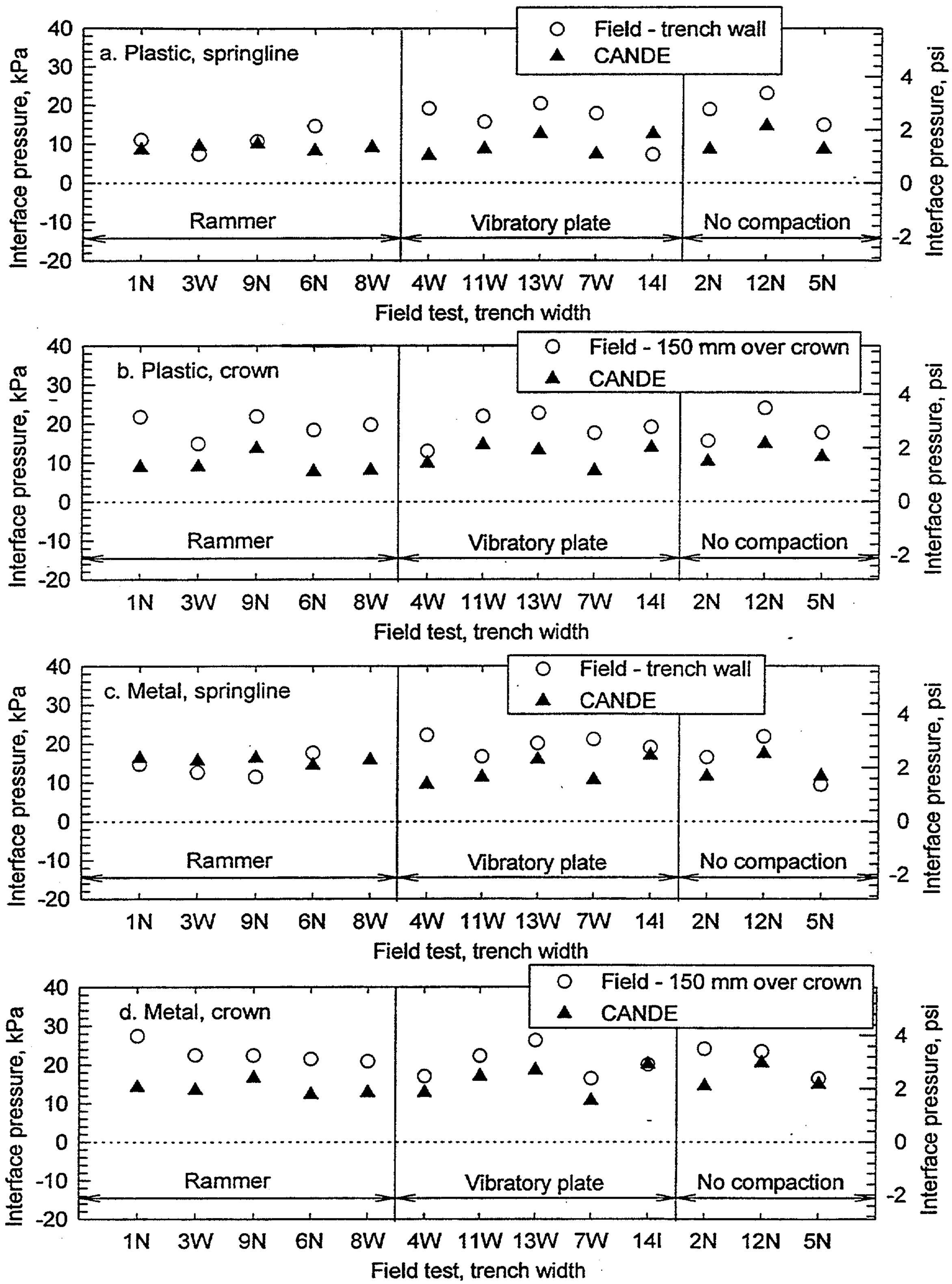


Figure 5.13 CANDE Interface Pressures Compared to Field Pressures on Plastic and Metal

5.2.2.3 Strains

The thrust and bending moment predictions from CANDE were converted to strains by dividing by the modulus of elasticity of 205 GPa (29,000,000 psi) for steel and 500 MPa (72,500 psi) for plastic and comparing to the field data in figs. 5.14 and 5.15 for the plastic and metal pipe respectively. The modulus of plastic is an estimated value, as noted earlier in this chapter. As noted in section 4.2.6.5, the strain levels for the metal pipe were small and are not reported. The match between analysis and data is generally good, which is expected since the deflection predictions matched well.

The comparison of thrust strains in fig. 5.14a suggest that CANDE predicts the thrust reasonably well for the 900 mm (36 in.) diameter pipe and modestly overestimates the thrust for the tests with 1,500 mm (60 in.) pipe. The strain predictions at the invert, springline, and crown of the plastic pipe are also in general agreement with the field data.

The same comparison for the metal pipe in fig. 5.15 also shows that the data are in general agreement with the CANDE predictions.

5.3 Summary

In general, both the Burns and Richard elasticity solution and the CANDE finite element program provide reasonable estimates of pipe response to earth load. The Burns and Richard solution is somewhat idealized and does not have the ability to treat special design conditions such as soft haunching, trench installations, or differing embedment material; however with some empirical adjustments, it is likely that this method could be developed into a simplified design method. The CANDE finite element program provided quite good estimates of behavior and is quite powerful in its ability to address special design situations; however, the complexity of the program and the uncertainty of actual installation conditions for most pipes, will probably result in CANDE being used only for special design situations.

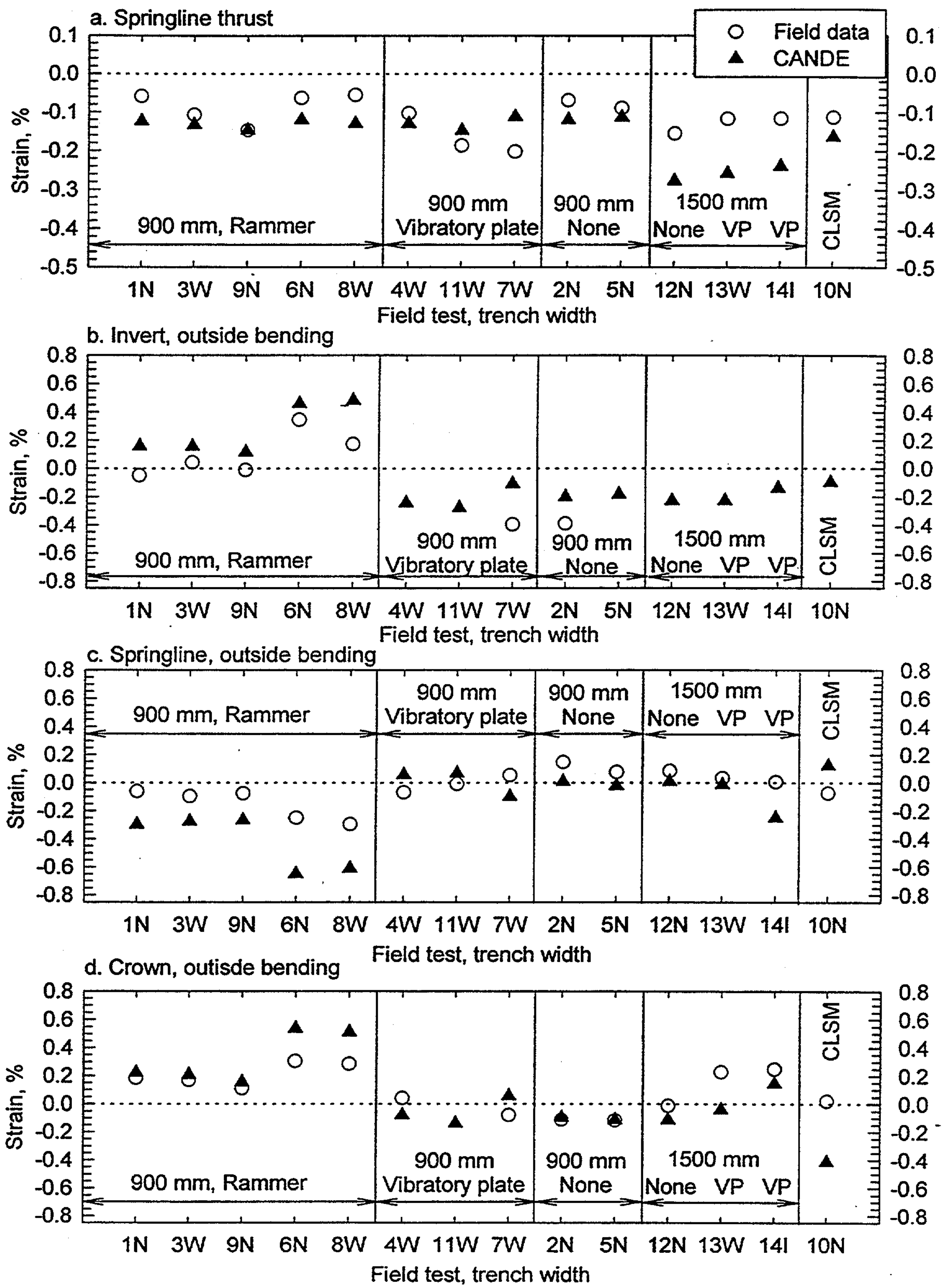


Figure 5.14 CANDE Strains Compared to Field Strains for Plastic Pipe

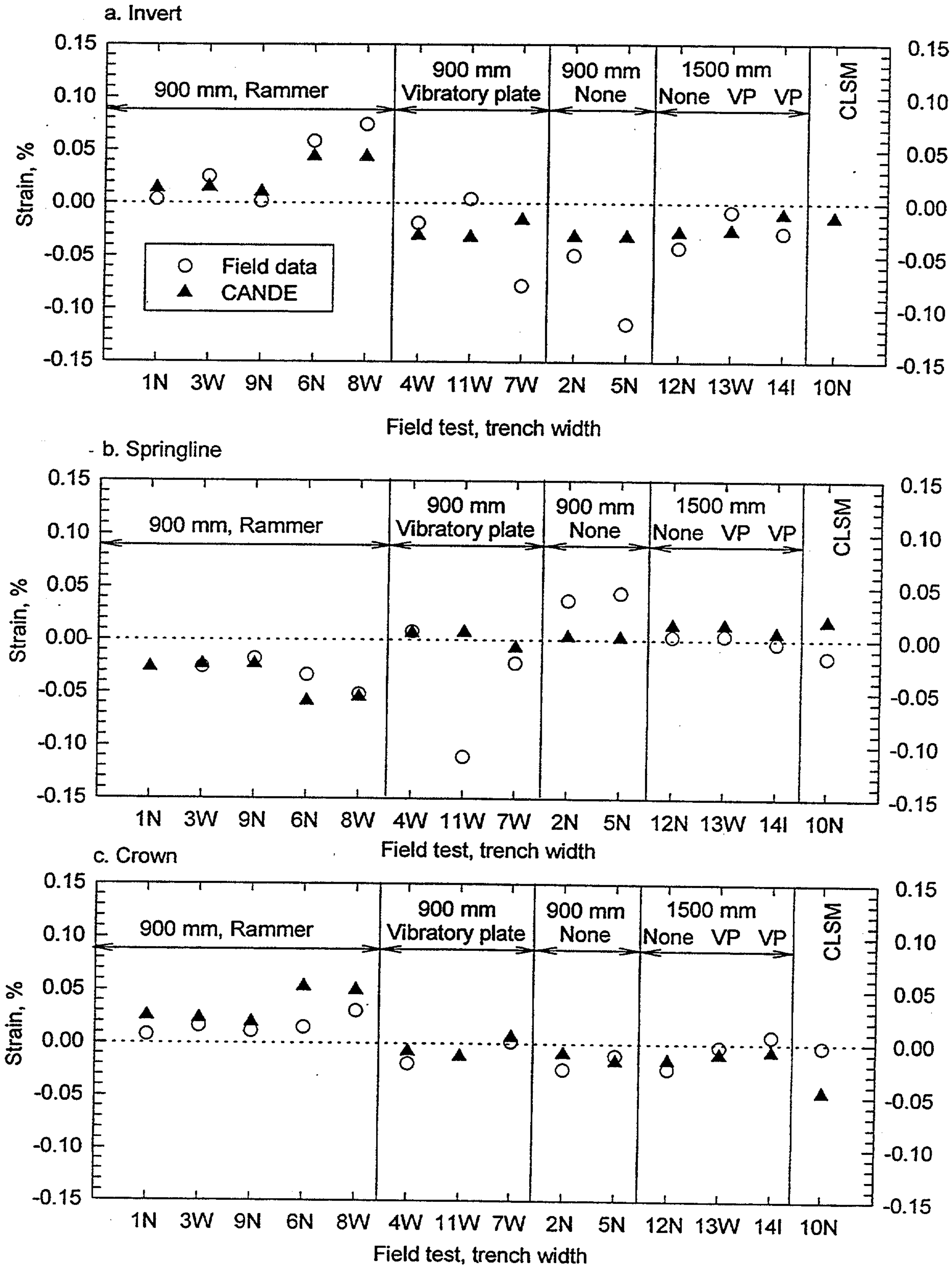


Figure 5.15 CANDE Strains Compared to Field Strains for Metal Pipe

CHAPTER 6
CONSIDERATIONS FOR INSTALLATION PRACTICE

Prior chapters have presented information on the following important issues related to installation practice for buried pipe:

- Characterization of in situ soils.
- Classification and characterization of backfill materials.
- Guidelines for installation practice.
- Computer modeling of buried pipe behavior.
- Use of CLSM as backfill for buried pipe installations.
- General behavior of buried pipe.

The nature of the pipe soil system makes it difficult to separate installation practice from design practice and almost any decision regarding one will affect the other. While the focus of this project is to understand the process of pipe installation, i.e., what happens as backfill is placed at the side of the pipe, some of the findings are applicable to the design process. In the following sections, each of the above items is discussed with a primary focus on installation practice. Design practice is discussed where appropriate.

6.1 In Situ Soils

Installation of a pipe requires stable in situ soil. This includes vertical support of the bedding and, for trench installations, lateral support by the trench walls. Provisions for achieving a stable foundation beneath a buried pipe are well defined in installation standards such as ASTM D 2321 and were not a subject of this study. Characterization of trench walls for lateral support provided to pipe, especially flexible pipe, is not as well defined. To address this issue, the designer needs to characterize the soil properties in terms of stiffness and strength and then assess the affect on the pipe's performance. The latter issue will be affected by the trench width relative to the pipe diameter and by the stiffness of the in situ soil relative to the backfill soil. These are largely matters considered in flexible pipe design, where a soil stiffness is required to evaluate lateral soil support to the pipe. In designing rigid pipes for trench installations, it is often assumed that the pipe receives no lateral soil support.

In Situ Soil Stiffness – The stiffness of in situ soils is vastly more variable than that of placed backfill materials. Placed materials must have a range of particle sizes that is suitable for handling and placing next to a pipe, and the potential for developing adequate support to the pipe when placed and compacted. Thus, formations with boulders and solid rock, aged deposits, such as some glacial tills that can be extremely hard when undisturbed, or excessively compressible materials, such as peats and soft clays, need not receive consideration as backfill materials. However, as in situ materials, all of these types of soils must be considered and evaluated.

A second issue in evaluating in situ materials is that pipelines are linear structures extending over great distances, and often through several soil formations. While complete evaluation of in situ properties could require many soil borings, few are generally taken because of the expense.

It is desirable therefore to provide simplified methods for evaluating in situ soils, such that the results of standard exploration techniques may be used. Perhaps the most common test conducted as part of soil exploration is the standard penetration test (ASTM D 1586), which evaluates soil by driving a sampler with a known effort. The result of this test is reported as the blows required to advance the sampler 300 mm (12 in.). Alternatively, either by the use of unconfined compression test (ASTM D 2166) or penetrometers, the strength of a fine-grained soil may be estimated relatively quickly. AWWA Manual M 45 (AWWA, 1996), *Fiberglass Pipe Design*, has published a table of E' values that are based on the results of the standard penetration test (SPT) or the unconfined compression strength of the soil (table 2.14). Given the work of chapter 3 (See section 3.4 and fig. 3.13, and section 6.2), which provides support for the use of the equality $E' = M_s$, this table can be used in empirical- or elasticity-based design methods, and should be a substantial aid to designers who have SPT or unconfined compression data available. The one-dimensional modulus may also be related to Young's modulus through Eq. 2.5, allowing the use of correlations between modulus and other soil properties.

A key consideration when evaluating in situ soil stiffness is that the condition of the soil at the time of testing may not be representative of the conditions at all times. Field tests 9 to 14, conducted at the clay site provide a good example of this. The undisturbed clay was relatively stiff, and for most of these tests, the soil strain gages indicate that lateral movement of the trench wall was inconsequential; however, during field test 11, there were heavy rains and the site became inundated. At the end of the test the trench, walls had moved outward 4 to 6 mm (0.15 to 0.25 in.). This is a relatively small movement, but it occurred over a period of a few days, and is indicative of ongoing movements that would continue in a permanent installation. Thus, the designer must consider potential changes in natural conditions.

Combined Pipe Support from Backfill and In Situ Soil – Also required for flexible pipe design is an evaluation of the affect of the in situ soils in providing support to a pipe. In a very narrow trench with little clearance between the trench walls and the pipe, the pipe deflection may be controlled mostly by the stiffness of the in situ soil; while in a very wide trench, the stiffness of the in situ soil will be inconsequential. Leonhardt (1979) developed Eq. 2.10 to address this issue and AWWA Manual M 45 (AWWA 1996) adopted a similar approach in the form of a table of influence factors for the in situ soil. The basis of both of these approaches is that the in situ soil is inconsequential for trench widths wider than about five pipe diameters. The field tests were consistent with this previous approach. In tests with wide trenches, with a width of about three pipe diameters for the 900 mm (36 in.) pipe, there was still an influence of the trench wall on the pipe behavior. The lateral soil stresses at the trench wall were of similar magnitude for the tests with this condition as for the narrow trench tests, with a width of about 1.6 pipe diameters (See fig. 4.44). While the assumption of needing a trench width of five pipe diameters would appear to be conservative, the cost of excavating wide trenches is expensive, especially with large diameter pipe. The method of Eq. 2.10, or AWWA Manual M 45 may be used in design for the time being, but better solutions are desired.

6.2 Backfill

Soil Groupings for Design – Many installation standards for buried pipe (ASTM D 2321, ASTM D 3839, AWWA Manual M 45, and AASHTO SIDD standard concrete pipe installations) identify three or four general soil groups within which the soils have similar characteristics as pipe backfill materials. This approach was also adopted by Howard in developing his table of values for the modulus of soil reaction. The typical groups, as discussed in section 2.2.1, generally include:

- Angular processed material, such as crushed stone (except for the SIDD soil groups),
- Gravels and sands with minimal fines content,
- Soils with fines but with a limit on total fines content and/or low plasticity, and
- Soils with unlimited fines content, but low plasticity.

Soils with high plasticity such as CH, and in some systems MH, while included in some soil design groupings, are generally considered unsuitable for pipe backfill material.

Overall, the approach of grouping soils into three or four broad categories has worked well, but it is desirable to adopt a single system of soil groups for pipe backfill that will apply to all types of pipe. The two soil groupings of most interest, since they are associated with stiffness properties that can be used in design, are the SIDD soil groups adopted by AASHTO for concrete pipe design and the Howard soil groups. The differences between these two groups in terms of gradation and plasticity were discussed in section 2.2.1, where it was shown that the SIDD soil groups tend to differentiate on the basis of clay versus silt (above or below the A-line, fig. 2.8), while the Howard soil groups tend to differentiate on the basis of fines content (more or less than 30 percent coarse grained material). There is not a clear choice for one group over the other; however, since the soil properties in the SIDD groups were developed for finite element analysis, and are the basis for the stiffness recommendations in this report (table 3.6), it is proposed that these groups be adopted for all types of pipes. The one shortcoming of this is that no hyperbolic properties have been developed for angular crushed stone materials. The properties of the SW soils could be used until more appropriate values become available. Although empirical in nature, the Howard recommendations of E' could also be used as a basis for extrapolating the SW values to values for crushed stone.

Also of interest is the approach of the Water Research Centre in the United Kingdom (table 2.9) which distinguishes between single size gravel and graded gravel. The single size gravel has the benefit of having a relatively high stiffness when placed loosely (note the relatively high values of loose density for soils 1 and 4 in fig. 3.3). The results of the laboratory soil box tests confirm this (see fig. 4.4 and individual test results). This high stiffness with minimal effort can be a significant aid when installing backfill in difficult situations or without inspection. The one concern with single size materials is that they have significant void space and thus are susceptible to migration of fine-grained soil from the adjacent in situ soils. Action must be taken to assess the likelihood of migration and, if necessary, take action to prevent it by using a geosynthetic filter fabric or control of the relative gradations of adjacent soils. ASTM D 2321 provides guidance on the latter subject.

Empirical and True Soil Properties: E' versus M_s – Preceding discussions have recommended the adoption of the constrained, or one-dimensional modulus, M_s, as a design soil modulus in lieu of the historically used modulus of soil reaction, E'. This is highly desirable as it allows testing for soil properties rather than back calculation from buried pipe tests to evaluate different types of soil. However, a large body of literature exists based on the modulus of soil reaction and some of this information is useful in characterizing soil stiffness for design even when using the constrained modulus. A comparison of the Howard values of E' with the Selig/SIDD hyperbolic soil properties was presented in fig. 3.13. This suggests that at low levels of applied stress the two sets of properties match reasonably well, and indeed, the data base from which Howard developed his recommended values of E' was based on pipe buried at modest depths of fill. While it is desirable to move away from E' as a design parameter and to take advantage of the available work related to it, the relationship $E' = M_s$ is recommended for use until more work is completed for values of M_s.

Reliability – The reliability of buried pipe installations is a significant issue. This requires an honest assessment by a designer about the quality of installation practice that will be exercised in the field. Examination of table 3.6 shows that the modulus of a soil at a density of 90 percent of maximum standard Proctor is about one half the modulus of a soil at 95 percent of maximum density, and the modulus of a soil at 85 percent of maximum density is one half or less that of a soil at 90 percent of maximum density. These significant changes suggest that the designer must evaluate the sensitivity of the installation to achieving the design soil stiffness, and must consider the likelihood of actually achieving the design soil stiffness during construction. In future development of design procedures for flexible or rigid pipes, introduction of a strength reduction factor on the soil stiffness term to account for sensitivity should be considered.

The selection of the most economical backfill and treatment in design is related to reliability as well as cost and deserves considerable attention. Crushed rock and SW materials provide good support to a pipe, and at high percent compaction will allow the use of the least expensive pipe. In addition, these materials have good stiffness properties even at low percent compaction. However, coarse grained backfills are often processed materials and are extremely expensive in some locations (Louisiana and Florida for example). Thus it is often economically desirable to use finer grained processed backfills or in situ soils as pipe embedment. Finer grained materials, such as the silty sand used in the field tests, are sensitive to moisture, are inherently less stiff at the same percent compaction as a coarser grained soil, and produce more deformation in flexible pipe during backfill compaction. The field tests clearly demonstrate that these materials may be successfully used as pipe backfill; however, they also demonstrate some of the problems that are likely. The saturation of the silty sand bedding in test 5, and the increased deflection in test 7, in which the pipe was installed without haunching are indications of the types of problems that can occur. Field tests with the stone backfill was subjected to the same conditions without problems.

The above discussion raises the question: What is the most economical pipe installation? It is easy to think that a less expensive pipe will be more economical; however, the total installation cost, which includes the cost of purchasing, placing, and compacting backfill and the cost of inspection, should be considered. High-quality installations should always be inspected. As noted above, the design soil stiffness is very sensitive to just a 5 percent variation in level of compaction. The cost of this inspection should be balanced against the cost of a more expensive pipe with backfill compacted to a less stringent requirement, and perhaps with reduced inspection. It may be more economical to purchase a more expensive pipe and reduce the sensitivity of the installation to variations in construction practice.

6.3 Guidelines for Installation Practice

There are many important steps that must be taken to achieve a quality buried pipe installation. A few of these steps and the related findings of the study are discussed here.

Trench Width – The previous section discussed the effects of trench width in terms of soil support to the pipe. There are many other considerations that affect the design decision of trench width as well. Traditionally, designers specify that trench widths be kept as narrow as possible to minimize excavation cost and the load predicted by the Marston trench load theory. Specifications sometimes allow trench widths as narrow as the pipe outside diameter plus 300 mm (12 in.). The actual criteria for trench width should be based on constructability. Working material into the haunch and compacting fill at the sides of the pipe are far more critical than minimizing the trench load. While wider trenches cost more to excavate and backfill, they must be used if required to properly construct the embedment zone. The findings of the project regarding trench width were:

1. For the 900 mm pipe the working space in the narrow trench (pipe outside diameter plus 600 mm, 24 in.), the working space was the minimum acceptable but adequate only because the trench was benched near the top of the pipe (See figs. 4.37 and 4.38).
2. For the 1,500 mm pipe, the narrow trench condition (pipe outside diameter plus 600 mm, 24 in.) was clearly inadequate to allow room for joining the pipe, haunching, and compacting the backfill, the intermediate trench (pipe outside diameter plus 900 mm, 36 in.) was marginally acceptable.
3. For both sizes of the pipe, the wide trench (pipe outside diameter plus 1800 mm, 72 in.) provided good working space.

In addition to the findings of the field tests, the conditions of a particular installation need to be considered. If CLSM is used as backfill then the trench need only be wide enough to allow placing and joining the pipe, because haunching and compaction are not required. If rounded pea gravel, or similar single sized material that is relatively free flowing is used then trenches could also be narrowed. The space between the trench wall and the springline should be wider than the compaction equipment. The rammer used in the field tests could be used for compaction in spaces as narrow as 300 mm, while compaction with the vibratory plate required a space at least 450 mm.

Bedding – Traditionally bedding under a pipe has been compacted, primarily as a method of controlling the pipe grade by minimizing settlement after construction (and perhaps also because it is easy to compact the bedding since the pipe does not get in the way). The SIDD installations adopted by AASHTO have incorporated a recommendation to leave the middle bedding, directly under the bottom of the pipe (fig. 2.4) and uncompacted. The computer modeling indicates that this reduces the load on the pipe and the invert bending moments. It is important that the outer bedding still be compacted to provide support to the haunch area of the pipe and to provide an alternate vertical load path around the pipe bottom. The field tests suggest that leaving the bedding soft does reduce the interface pressures at the pipe bottom. The computer modeling (chapter 5) confirms this benefit. Even though the invert interface pressures that were measured in the field were consistently higher than predicted by the model, both field and computer model demonstrate lower invert pressures with uncompacted bedding.

Haunching – Some effort at haunching should always be specified. The bending moments in the field tests and the computer models are significantly greater in the unhaunched installations. In addition, the failure to provide haunching incorporates a significant void in the backfill that can lead to longer term soil movements and corresponding reduced support to the pipe. In the field and laboratory tests, slicing backfill into the haunch area with shovels was shown to be an effective method of providing haunch support. Tampers, such as used on field tests 12 to 14 were also very effective. A large-faced tamper, 75 by 150 mm (3 by 6 in.), was effective for the silty sand and a small-faced tamper, 25 by 75 mm (1 by 3 in.) was effective for the stone. A small faced tamper is imperative for angular materials to generate sufficient force to overcome the particle interlocking. A tamper attached to a long rod can allow a laborer to be out of the trench while tamping the haunch.

Haunching is best accomplished after the pipe is set in position, by placing part of the first lift of backfill, working it into the haunches and then placing the remainder of the lift. Haunching effort cannot be effectively applied if backfill is placed so high on the pipe that it blocks access to the haunch zone.

Compaction of Backfill – Some compactive effort is always desirable. Even though some coarse-grained backfill materials may achieve 85 percent to 90 percent of maximum Proctor density when placed with little effort, there are undoubtedly voids that develop around pipes and against trench walls when the material is first placed. This appears to be particularly true with the deep corrugations of the plastic pipe. A modest effort at compaction (perhaps a simple effort at shovel slicing, although this was not evaluated during the tests) would likely eliminate the 1 percent jump in deflections observed in tests 2 and 5.

Compaction induced deflections (peaking) clearly increase as the backfill materials become finer grained. In the field tests the peaking deflection with silty sand backfill was about three times the peaking deflection with the stone for the same number of coverages of the compactor. While the magnitudes of the peaking deflections (up to 2 percent change in diameter, see fig. 4.40) were not excessive, they were significant, and designers should be aware of this issue. Larger compaction equipment, such as walk behind or ride on rollers, or the use of lower stiffness pipe, could easily result in excessive peaking, or distortion of the pipe shape during compaction. Limits on upward peaking because of compaction effects should be lower than limits on downward deflection caused by earth load. This recommendation is made because peaking deflection is essentially the result of a point load and can result in higher local deflections and stresses than deflection caused by earth load.

Similar to leaving the bedding uncompacted under the pipe, there is merit in leaving the portion of the first backfill lift that covers the pipe uncompacted directly over the pipe as well. The computer model suggests that this drops the interface pressure on the top of the pipe, meaning that load is transferred into the pipe further out toward the sides of the pipe which should reduce the bending moments in the pipe.

6.4 Computer Modeling

The field tests were successfully modeled using the finite element computer program CANDE. A consistent approach was taken for all of the tests, and the field data matched the computer predictions quite well. A number of recommendations are made here:

1. Interface pressure readings and penetrometer testing indicate that with soil backfill, even with significant haunching effort, there is always a soft spot about 30 degrees

from the invert. This was modeled with the "void" zone shown in figs. 5.1 and 5.2. It is recommended that this zone be incorporated in all models of buried pipe installations unless the backfill is CLSM.

2. The use of concentrated forces has been shown to be an effective method to model compaction effects, and a simplified expression for computing these forces was developed; however, a soil model should be developed that would allow application of compaction forces directly to the soil. No practical method of accomplishing this has yet been incorporated into a generally available computer program such as CANDE.

3. When a soil layer is placed in the CANDE program, it is assigned the properties of the final compacted material. In actual construction, it is placed loosely and then compacted. This means that the weight of the soil is imposed on the pipe when the soil strength and stiffness are low, and it is then compacted to improve the properties. This type of modeling can have a significant effect on the loads imposed on a pipe, particularly in a trench installation. The apparent "arching" of load between the trench wall and the pipe noted for concrete pipes in section 5.2.2 (figs. 5.8 and 5.9) could be significantly reduced if the soil properties are those of loose soil when the weight of the soil is applied, and then increased to dense properties.

4. The behavior of the plastic pipe was best modeled using a lower modulus of elasticity than the specified short term value in AASHTO. This suggests that the viscoelastic nature of thermoplastics has an effect on pipe response during backfilling.

6.5 CLSM

The field tests show that CLSM can be an excellent backfill material. It placed easily and formed a stiff, uniform pipe support. Study of CLSM was not a key goal of this project; however, several recommendations and suggestions for further research can be made.

Mix Design – The ASTM flow test, Provisional Standard PS-28, is an excellent measure of the flowability of the mix. The study showed that flowability is derived largely from fly ash, not water. Mixes with high water contents tend to have the water segregate and do not flow well. The drawback to high fly ash content is that the pozzolanic nature of fly ash contributes to the long term strength gain and inhibits excavatability of the material. The mix design used in this study, which included 45 kg/m^3 (76 lb/yd^3) cement and 244 kg/m^3 (412 lb/yd^3) of fly ash had excellent flowability characteristics but its strength made it difficult to excavate. It may be appropriate to reduce the cement content.

Placing CLSM – Placing pipe up on blockings or bags as was done for the field tests in this study assures that the CLSM gets under the pipe and provides uniform support. The blocking should not be overly stiff, i.e., polystyrene foam would be desirable, wood would probably be acceptable, and concrete blocks would be unacceptable. If blocking the pipe is found too time consuming, it should be acceptable to place the pipe directly on the bedding as shown in fig. 2.5 taken from the clay pipe installation standard ASTM C 12; however, the CLSM will have to be delivered to both sides of the pipe. Installation with CLSM requires some control over when the pipe is backfilled. The pipe should not be further backfilled until the CLSM embedment has a greater stiffness than the bedding. Adding backfill when the CLSM is still soft, may actually produce a hard bedding situation and a line load at the invert of the pipe, since the CLSM in the haunch zone could be quite soft and not capable of providing good support. This should be an area of future study.

Controlling flotation is a key issue in the use of CLSM. In the field test, the pipe were weighted with gravel bags; however, this is not appropriate for an actual construction project. A quickly installed bracket that holds down the top of the pipe by bracing against the trench wall could be developed or, short sections culverts could be (carefully) held down with construction equipment. Because of the large magnitude of the flotation forces, placing the CLSM in multiple lifts will almost always be required. In the field tests, the plastic pipe, with deep corrugations developed a mechanical interlock with the first lift of CLSM that kept it from floating while placing the second lift. This suggests that studs could be welded to steel pipes, or could be strapped to plastic pipes to similarly form a mechanical bond to a first lift. This type of system could be developed to serve both the function of supporting the pipe off the bedding and providing anchorage from flotation.

The two deliveries of CLSM to the field tests for this project were quite different in strength and flowability and hence required mix adjustment in the field. Thus, checking the flow characteristics at the time of placement should be standard practice.

Quality Control – The use of test cylinders for strength testing may not be suitable as a quality control procedure. The low strength mixes, which are desirable for excavatability, were fragile and very difficult to test at an age of 7 days, and could not have been tested at earlier ages. At an age of 7 days, it is likely that a pipe or culvert has already been backfilled and the test results would serve as documentation of the material rather than a true quality control test. During the conduct of the field tests in this study, the density of the CLSM was checked with a nuclear density gage. This has merit as a field control procedure since the result of the test is known immediately.

It is necessary to decide what CLSM characteristics are important and require quality control. In structural design of buried pipe and culverts, a dense soil backfill is considered to be a high quality pipe support. In the field tests, the in place density of the CLSM was 2,130 kg/m^3 (133 pcf) which is representative of a broadly graded dense sand. This suggests that the flowable nature of the CLSM is actually a delivery system to place soil, rather than a cementitious material dependent on strength gain. This philosophy allows field testing to use geotechnical type tests that can be conducted quickly with results available right away.

During the field tests, the excess water hydrated out of the CLSM quickly and the material could be walked on within two hours. There were no problems in placing the second lift after 2 hours, and, had it not been the end of the work day, it is expected that there would have been no problems continuing normal backfilling after the second pour had set for 2 hours.

Air-Modified CLSM – Although not tested in this study, McGrath and Hoopes (1997) reported on the use of air-modified CLSM. This is CLSM with high air content, about 30 percent by volume, to produce flowable mixes without depending on fly ash. This has the benefit of reducing the long-term strength gain that results because of the pozzolanic reaction of the fly ash. The draw back to air-modified CLSM is that it depends on the strength gain caused by the curing of the cement to develop strength and stiffness. This material could not be backfilled after 2 hours.

6.6 General Behavior of Buried Pipe

The relatively high compaction deflections generated in the computer model of the 1,500 mm (60 in.) plastic pipe relative to the 900 mm (36 in.) plastic and metal pipe and the 1,500 mm (60 in.) diameter steel pipe, that were not observed in the field data, suggest that this profile design (a solid wall with a bonded tube as a rib) mobilizes a greater longitudinal length of pipes to resist compaction forces than does the corrugated pipe wall.

It may be appropriate to introduce design conditions based on how great a length of pipe is developed in resisting concentrated (i.e., compaction) loads.

The longitudinal strains in the 900 mm diameter plastic pipe were about 50 percent of the circumferential strains. This is a significant level which means that consideration of longitudinal stresses may be necessary for buried pipe.

CHAPTER 7
CONCLUSIONS

This report presents the results of an in depth evaluation of installation practice for buried pipe. The current practice of AASHTO member States was surveyed, as well as the current practice of pipe suppliers and standards organizations such as ASTM and AASHTO. Additional insight into backfill materials, and pipe behavior during installation was developed through laboratory backfill characterization tests, laboratory soil box tests, full-scale field tests, and computer modeling of test results. The main conclusions of the study are:

1. The soil properties used for the development of the SIDD concrete pipe installations are recommended as design properties for all types of pipes. These properties were developed for the hyperbolic model of soil behavior that is widely used for culvert analysis.

2. For simplified design use of the constrained modulus, M_s, is recommended, in lieu of the historical, but empirical modulus of soil reaction, E'. Design values for the constrained soil modulus are presented. The introduction of the table of soil values for M_s allows designers to assess the impact of using lower quality backfill materials than currently allowed by AASHTO specifications and to consider the effect of change in soil modulus with increasing confinement. Although it has been clearly demonstrated that fine grained soils have inherently lower stiffness, are sensitive to moisture, and require greater compactive effort to install, there are installation conditions where use of such materials may be economical provided proper installation controls are in place.

3. Pipe bedding should be left uncompacted under the middle third of the pipe diameter. This has been shown to be an effective method of reducing invert bending moments, particularly for rigid pipes.

4. Finite element modeling with the computer program CANDE has been shown to be an effective tool to understand pipe behavior during installation. It is important to model the actual installation conditions, such as the soft area in the lower haunch and compaction effects.

5. CANDE is the only generally available finite element computer program for culvert design at the present time. Technical improvements, such as the introduction of soil with loose soil properties and a later conversion to compacted properties, have been proposed and a better user interface would greatly increase the utility of the program. Of particular importance is access to the SIDD soil properties. Currently, use of these properties in CANDE requires manual input by the user. CANDE should be modified to make these properties available as defaults.

6. Longitudinal effects should be considered in design. Longitudinal strains in some pipe are significant, and the response to compaction effects involves the longitudinal stiffness of the pipe.

7. Controlled low strength material, CLSM, has been shown to be an effective backfill material. Recommendations for design and implementation of CLSM installations have been developed.

Overall, pipe performance is significantly controlled by installation practice and soil properties. This project has demonstrated that soil properties vary widely with relatively small changes in density. Quality control of construction practices is a key issue to be assessed by the designer when selecting pipe and backfill materials. It is believed that significant improvements in long term performance and reductions in installed cost will be achieved by improved contractor education and greater enforcement of specifications. The effort to accomplish this may appear expensive; however, the cost of investigating and repairing just an occasional failure, and the cost of maintaining pipes that were installed poorly, are so significant that any expense caused by increased construction costs should be saved quickly through elimination of lifetime costs.