Long-term monitoring of SIDD Type IV installations

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Abstract: An evaluation of the standard installation direct design (SIDD) prediction method has been undertaken by constructing and monitoring full-scale test beds installed according to SIDD Type IV specifications at four test sites across southern Ontario, Canada. Stresses around the test beds were monitored over a period of 20 months. The internal diameter of the test pipe segments varied from 600 mm to 900 mm; in situ soil conditions ranged from organic clay to sand, and burial depths varied from 1.5 to 3 times the diameter of the installed pipe. All test sections were subjected to frequent heavy traffic loads, representing a worse case loading scenario. Measurements from the 20 month monitoring period were compared with predictions from Ontario Provincial Standards and SIDD specifications. It was concluded that the SIDD method reasonably predicts the stress envelope around a buried rigid pipe installed using the cut-and-cover construction method. The indirect design method currently used by the Ontario Provincial Standards was found to provide an overly conservative prediction of soil stresses at the invert of the pipe. Field measurements also suggest that the value of the horizontal arching factor (HAF) currently recommended by SIDD for Type IV installations is overly conservative and can be increased while maintaining a conservative design approach.

Key words: soil, pipe, interaction, rigid, SIDD, monitoring.

Résumé : On a entrepris une évaluation de la méthode de prédiction de la « standard installation direct design » (SIDD) en construisant et en faisant des mesures sur des lits d'essais à pleine grandeur mis en place selon les spécifications du SIDD Type IV sur trois sites d'essais à travers le sud-ouest de l'Ontario, Canada. Les contraintes autour des lits d'essais ont été mesurées durant une période de vingt mois. Le diamètre interne des segments de conduites variait de 600 mm à 900 mm; les conditions des sols in situ variaient d'une argile organique à un sable, et les profondeurs d'enfouissement ont varié de 1,5 à 3 fois le diamètre de la conduite installée. Toutes les sections d'essai ont été assujetties à de fréquents passages de trafic lourd présentant un scénario des pires cas de chargement. Les mesures de la période d'essai de 20 mois ont été comparées avec les prédictions des spécifications de l'OPSS et SIDD. On en a conclu que la méthode SIDD prédit raisonnablement bien l'enveloppe des contraintes autour de la conduite rigide enfouie mise en place au moyen de la méthode de construction déblai-remblai. On a trouvé que la méthode de conception indirecte couramment utilisée par l'OPSS fournissait une prédiction de contraintes dans le sol trop sécuritaires sur le radier de la conduite. Les mesures sur le terrain suggéraient également que la valeur du facteur d'arc-boutant horizontal (HAF) couramment recommandé par le SIDD pour les installations de type IV est trop sécuritaire et peut être augmenté tout en maintenant une approche de conception sécuritaire.

Mots-clés : sol, tuyau, interaction, rigide, SIDD, mesures.

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Introduction

The first step in the design of a buried concrete pipe is the determination of the overburden earth and live loads that will act on the pipe installation. In this context, installation parameters such as bedding type and backfill compaction level are important factors. High-quality bedding material placed under the invert and haunches of a rigid pipe structure can greatly enhance its load bearing capacity. Such bedding provides uniform support to the pipe segment against its own weight, overburden earth load, and any applicable live loads, thereby reducing stress concentrations. However, the placement of imported bedding materials and the disposal of the excavated in situ soil are costly components of construction, accounting for as much as 15% of installation costs.

In the early 1970s, the American Concrete Pipe Association (ACPA, Irvin, Texas) developed the standard installation

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Table 1. Standard trench installation types (modified from CSA 2000).

Installation type	Minimum bedding thickness: Soil foundations	Minimum bedding thickness: Rock foundations	Soil group[a]	Minimum standard Proctor compaction: Haunch and outer bedding (%)	Minimum standard Proctor compaction: Lower sidefill (%)
I	D_o/24, not less than 75 mm	D_o/12, not less than 150 mm	I	95	90
I	D_o/24, not less than 75 mm	D_o/12, not less than 150 mm	II	Not permitted	95
I	D_o/24, not less than 75 mm	D_o/12, not less than 150 mm	III	Not permitted	100
II	D_o/24, not less than 75 mm	D_o/12, not less than 150 mm	I	90	85
II	D_o/24, not less than 75 mm	D_o/12, not less than 150 mm	II	95	90
II	D_o/24, not less than 75 mm	D_o/12, not less than 150 mm	III	Not permitted	95
III	D_o/24, not less than 75 mm	D_o/12, not less than 150 mm	I	85	85
III	D_o/24, not less than 75 mm	D_o/12, not less than 150 mm	II	90	90
III	D_o/24, not less than 75 mm	D_o/12, not less than 150 mm	III	95	95
IV	No bedding needed	D_o/12, not less than 150 mm	I	Not needed	Not needed
IV	No bedding needed	D_o/12, not less than 150 mm	II	Not needed	Not needed
IV	No bedding needed	D_o/12, not less than 150 mm	III	85	85

Note: D_0, outside diameter.
[a]I, sand and gravel; II, sandy-silt; III, silty-clay.

direct design (SIDD) prediction method to establish guidelines for the maximum utilization of native soils using four different types of standard installations (Types I–IV). The use of native material reduced the need for imported pipe bedding materials. Aside from significant cost savings associated with using native materials, particularly where favourable in situ conditions exist, additional benefits of SIDD include the conservation of nonrenewable granular material deposits as well as landfill space. It is estimated that up to 40% of the concrete pipe installations performed annually across the province of Ontario can utilize native material to some degree under the SIDD guidelines (P. Smeltzer, personal communication (2001)). Considering that pre-cast concrete is the dominant material type in sewer pipes of 600 mm diameter and larger installed in Canadian municipalities, the potential annual savings are substantial.

With SIDD, the designer can approximate the in situ earth pressure around the pipe and account for its contribution to the stresses developed within the pipe wall. Lateral soil pressures provide additional confining stresses that support the pipe at the springline and reduce bending moment at the invert. Thus, the requirements of steel reinforcement in the pipe wall can be reduced, potentially introducing additional savings during the manufacturing process.

Several prior researchers evaluated selected aspects of SIDD using full-scale test beds. The Minnesota Department of Transportation (Roseville, Minnesota) performed full-scale tests aimed at evaluating SIDD Type II and III installations (Hill et al. 1999). Simpson Gumpertz & Heger Inc. (1999) evaluated a full-scale SIDD Type I installation in Calgary, Alberta. Zhao and Daigle (2001) reviewed the performance of SIDD Type II and III installations with respect to field measurements collected from a test bed in Ottawa, Ontario. However, to the best of the authors' knowledge, no field tests were undertaken to evaluate installations that utilized SIDD Type IV bedding. To date, SIDD has been adopted in the United States and in parts of western Canada (e.g., the city of Calgary, Alberta and the province of Manitoba). In addition, it has been incorporated into the Canadian Highway Bridge Design Code (CSA 2000).

In 2000, the Ontario Concrete Pipe Association (OCPA) joined with the National Research Council Canada (NRCC), the Ministry of Transportation of Ontario (MTO), the Regional Municipality of Ottawa-Carleton, and The University of Western Ontario (UWO) to study the possibility of using SIDD as an alternative design approach to the current Ontario Provincial Standard OPSS 421 (OPSS 1995). As part of this project, a field study was undertaken using full-scale SIDD Type IV test beds for monitoring short- and long-term in situ stresses around buried concrete pipes under earth and traffic loads. Full-scale test beds were installed at four sites across Ontario in conformance with SIDD Type IV specifications. This paper presents the results of short- and long-term monitoring of stresses around the test beds as well as a comparison of field measurements with predictions obtained from two design methods, OPSS 421 (OPSS 1995) and ASCE 93–15 (ASCE 1993).

SIDD: An overview

Direct design considers the contribution of the earth pressure envelope developed around the pipe circumference due to the applied vertical loads. Early research efforts led to the development of two semi-analytical earth pressure distributions (Paris 1921; Olander 1950), however, both of these methods require the use of Marston's method (Marston and Anderson 1913) to determine the total load acting on the pipe. Heger et al. (1985) developed a pressure distribution based on analysis of soil–structure interaction using a finite element (FE) program named SPIDA (soil–pipe interaction design and analysis). Four standard variations of the Heger earth pressure distribution were developed, representing a range of installation efforts (ASCE 1993). These standard installations and the corresponding design procedures are commonly known as SIDD.

Table 1 describes the four installation types considered in SIDD. Type I specifies the highest quality of backfill materials and soil compaction effort in the embedment zone below the pipe's springline, while Type IV installation requires no imported bedding material at the foundation or the haunches,

Table 2. Characteristics of test sites.

	Cambridge	Guelph	Whitby	Barrie
Location	Hanson Pipe & Products Canada Inc.	Con-Cast Pipe	Hanson Pipe & Products Canada Inc.	Edgar's Aggregate Pit
Native fill soil	Well graded gravely silty-sand	Well graded gravely silty sand with 30% fines	Silty sand layer with some gravel changing to gray silty clay layer	Well graded gravely sand with little fines change to uniform silty sand
Bedding soil	Well graded silty-sand	Silty clay layer with cobbles	Organic silty clay of low plasticity	Uniform silty sand
Average depth of trench (m)	2.3	2.8	2.5	2.5[a]
Pipe inner dia. (mm)	675	900	750	600
Pipe outer dia. (mm)	890	1180	930	800
Segment length (m)	2.4	2.4	2.4	2.44
No. of pressure cells	10	10	10	12
GWT[b] (m)	NA	NA	2.65	NA
Date of installation	Late May 2000	Mid-June 2000	Late May 2000	Late July 2000
End of monitoring	29 March 2002	19 April 2002	28 March 2002	20 March 2002

Note: NA, not applicable.
[a]Depth measured from the surface of the approach ramp.
[b]Groundwater table was measured from the ground elevation on the date of the installation.

as well as limited field control. Type IV installation utilizes the inherent strength of the pipe with little assistance from the surrounding soil to resist the external loads and the internal pressure.

A major feature of the Heger pressure distribution is the use of a bedding reaction distribution with three separate pressure bulbs. The pressure bulb directly under the invert models the support at the pipe bedding, and the two side bulbs model the soil support offered by the upper haunch regions. Good installation practices should result in relatively large pressure bulbs in the upper haunch regions, while little or no soil support is expected if the backfill at the haunch region was not compacted.

The prism load (PL) at the top of the pipe is calculated using the following expression:

$$[1] \qquad \mathrm{PL} = \left(\frac{\gamma D_0}{1000}\right)\left[H + \left(\frac{0.107 D_0}{1000}\right)\right]$$

where PL is the prism load (N/m), γ is the unit weight of the soil (N/m^3), D_0 is the outside diameter of the pipe (mm), and H is the height of earth above the top of the pipe (m). The Heger earth pressure distribution is then used to calculate the stress envelope around the pipe from the prism load. A more complete description of SIDD and its applications can be found in ASCE (1993).

Full-scale testing

Site configurations

Four full-scale test beds located at Cambridge, Guelph, Whitby, and Barrie, Ontario, Canada were constructed in the summer of 2000. The test beds varied in terms of soil cover, trench geometry, pipe diameter, and in situ and backfill soil types. Table 2 summarizes the characteristics of these sites including geotechnical parameters, pipe dimensions, and monitoring information. Each test bed consisted of five pipe segments connected to an access manhole. The inner diameter of the pipe segments varied from 600 mm to 900 mm, and the depth of the trench varied from 2.3 m to 2.8 m. In situ soils at the sites varied from silty sand to organic clay. Trench geometries for each of the four sites are shown in Fig. 1. Access to the pipe segments was secured by installing a manhole at each of the sites, with outer diameters varying from 1.5 m to 1.8 m.

At each of the test beds the three interior pipe segments (labelled 2, 3, and 4) were instrumented to collect earth pressure data (Fig. 2). Segment 3 was typically the most heavily instrumented segment (except in the case of the Cambridge installation where it was Segment 4), and is thus referred to as the "Test Segment." The test segment was located beneath the most heavily traveled zone traversing the test bed. The two adjacent pipe segments (i.e., Segments 2 and 4) were used as controls.

Soil conditions

The site at Cambridge, Ontario is located near the entrance to the storage yard of the local Hanson Pipe & Products Canada Inc. plant. The in situ soil was classified as well-graded silty-sand with some gravel overlain by silty-sand with cobbles up to 150 mm in diameter. Compaction tests (ASTM 2000*a*) on the in situ material indicated a maximum standard Proctor density of 2040 kg/m^3 with an optimum moisture content (ASTM 1998) of 9.4%. The in situ moisture content varied between 3% and 6%.

The site at Guelph, Ontario is located in the concrete pipe storage yard of Con Cast Pipe. The soil was well-graded gravely silty sand with 30% fines. The maximum standard Proctor density (ASTM 2000*a*) was 2270 kg/m^3, and the optimum moisture content was 7.0%. The in situ moisture content (ASTM 1998) varied between 7% and 9%.

The site at Whitby, Ontario is located at the entrance to the storage yard of the Hanson Pipe & Products Canada Inc. facility, in a previously swampy area that was reclaimed using recycled asphalt. Consequently, the excavation was conducted primarily through the fill of recycled asphalt

Fig. 1. Trench geometry (units in m). For the Barrie site an additional 200 mm layer of granular soil was placed as part of the ramp to the truck weigh scale station.

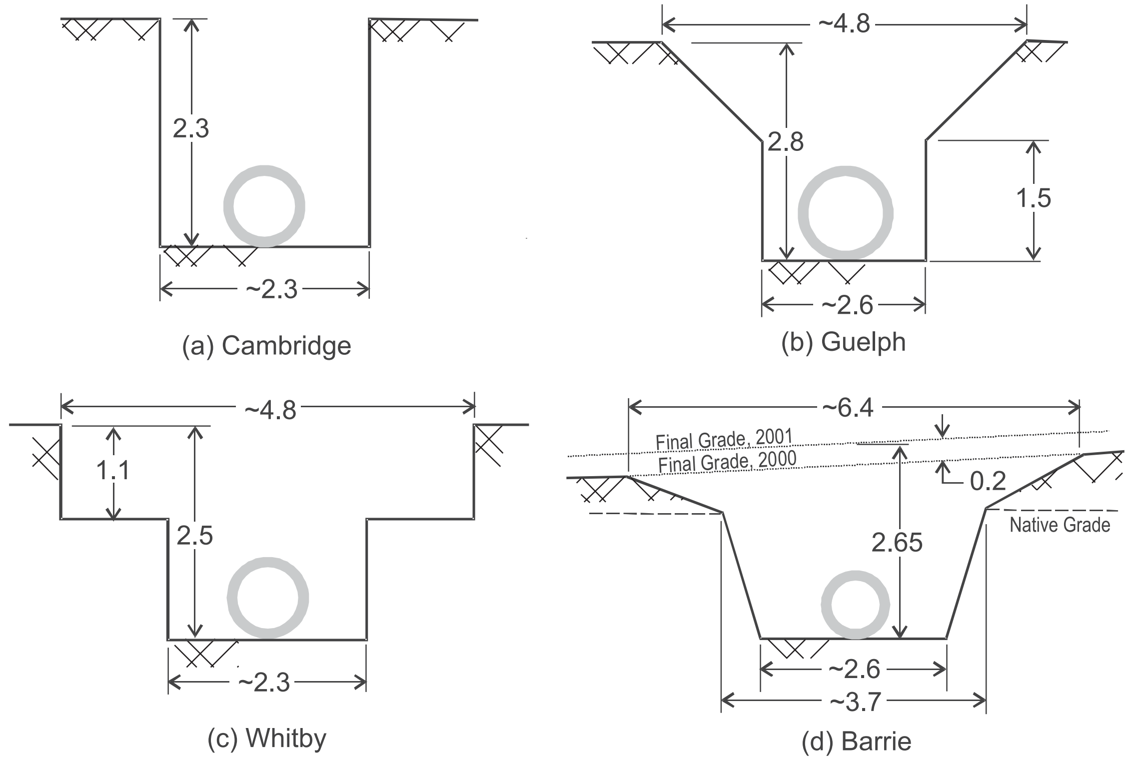
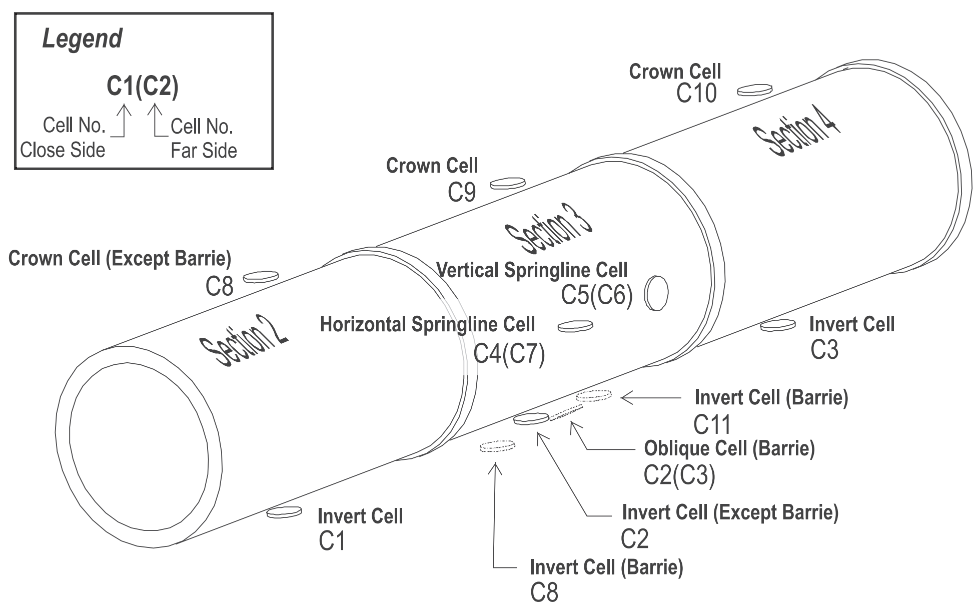


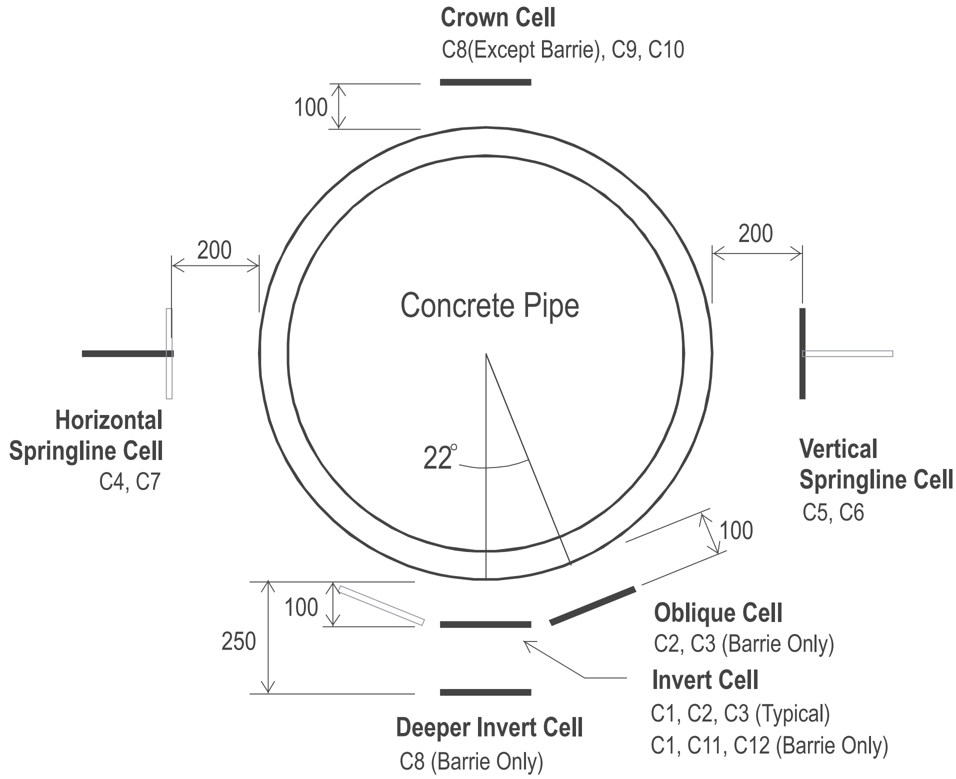
Fig. 2. Configuration of a typical monitoring system.



overlying a silty sand layer. The soil below the pipe bedding was classified as organic silty clay of low plasticity (ASTM 2000*b*). The material above the bedding was a well-graded sandy silt-clay mixture of low plasticity. The groundwater table was observed at a depth of 2.65 m, and the in situ moisture content (ASTM 1998) across the depth of the excavation varied between 8% and 28%. The maximum standard Proctor density (ASTM 2000*a*) for the in situ soil was measured to be 2060 kg/m^3 with an optimum moisture content of 9.5%.

The test bed site in Barrie, Ontario is located at the Edgar's aggregate pit, beneath the approach ramp to a truck weigh station. The bedding soil was classified as uniform silty sand. The backfill material was classified as well-graded gravely sand with little fines. A maximum standard Proctor density of 1725 kg/m^3 and an optimum moisture content of 10.0% were measured (ASTM 2000*a*). The in situ moisture content (ASTM 1998) was between 2% and 5%. Detailed geotechnical data for each of the four sites is reported by Wong (2002).

Fig. 3. Cross-sectional view of a typical test segment (units in mm). Oblique cells were installed only at the Barrie site. An invert cell located at 250 mm below the pipe invert pipe was installed only at the Barrie site. The horizontal and vertical springline cells were offset longitudinally a distance of 600 mm apart.



Instrumentation

Ten 275 mm diameter earth pressure cells (4800E Vibrating Wire; Geokon Inc., Lebanon, New Hampshire) were used to monitor the stresses in the soil adjacent to the buried pipe segments. A schematic diagram of the configuration of a typical monitoring system is given in Fig. 2. A cross sectional view of a typical test segment is shown in Fig. 3. An exception is the Barrie site where 12 pressure cells were installed using a slightly different configuration (as indicated in Figs. 2 and 3).

At a typical site, three 340 kPa cells designated C1, C2, and C3 were installed 100 mm beneath the inverts of the inner pipe segments (segments 2, 3, and 4, respectively). A similar set of three 340 kPa cells designated C8, C9, and C10 were installed 100 mm above the crown of the same test segments, respectively. In addition, Segment 3 (the Test Segment) had four earth pressure cells installed at the springline, 200 mm away from the external wall of the pipe. Two 340 kPa cells, designated C4 and C7, were placed with a horizontal orientation and two 170 kPa cells, designated C5 and C6, were installed with a vertical orientation on either side of the test segment. The horizontal and vertical cells were offset longitudinally a distance of 600 mm edge-to-edge from each other. The designations and locations of the various earth pressure cells are summarized in Table 3.

The eight cells installed in horizontal orientation have a normal operating range of 340 kPa but are capable of accommodating a 50% overload (as per the manufacturer's specifications). Pressure cells installed at a vertical orientation at the springline of the pipe have a normal operating range of 170 kPa and are also capable of accommodating a 50% overload. All cells use vibrating wire sensors to determine the pressure and are accurate up to ±0.1% of full scale. The calibration curves of a dozen randomly selected load cells were validated in the laboratory. Following this validation, the certified manufacturer's calibration curves were used throughout the study.

The configuration of the pressure cells at the Barrie site was slightly different. Two additional 340 kPa cells, designated C2 and C3, were located at the haunch region. The haunch cells were placed obliquely (22° offset from the vertical line) to monitor the radial pressure at that location. An additional 340 kPa cell, designated C8, was located 250 mm below the invert. The deeper invert cell was placed to monitor the stress gradient between the high stress concentration at the pipe invert and the geostatic stress.

A potential problem associated with earth pressure cells is the degree and uniformity of contact between the pressure cell and the soil as the cell's measurement is directly related to the contact pressure. A pressure cell may provide erroneous readings if the contact pressure is nonuniform or if it is subjected to concentrated loads (i.e., a cobblestone just above it). An installation procedure was developed where each of the pressure cells was placed in the centre of a 100 mm thick fine sand layer, ensuring that the load was evenly distributed over the soil–cell interface. The fine sand "encasement" also acted to protect the cell from damage caused by sharp stones in the in situ soil. Each "sand-encased" cell was wrapped with a piece of geotextile to prevent the erosion of fine sand due to groundwater flow or infiltration. Care was exercised during installation of the pressure cells to ensure a uniform contact between the pipe

Table 3. Designation and location of earth pressure cells.

		Typical			Barrie Site		
Cell	Stress rating (kPa)	Location	Orientation	Seg	Location	Orientation	Seg
C1	340	100 mm below invert	Horizontal	2	100 mm below invert	Horizontal	2
C2	340	100 mm below invert	Horizontal	3	Offset 22° from vertical between invert and springline	Oblique	2
C3	340	100 mm below invert	Horizontal	4	Offset 22° from vertical between invert and springline	Oblique	2
C4	340	At springline	Horizontal	3	At springline	Horizontal	3
C5	170	At springline	Vertical	3	At springline	Vertical	3
C6	170	At springline	Vertical	3	At springline	Vertical	3
C7	340	At springline	Horizontal	3	At springline	Horizontal	3
C8	340	100 mm above crown	Horizontal	2	250 mm below invert	Horizontal	3
C9	340	100 mm above crown	Horizontal	3	100 mm above crown	Horizontal	3
C10	340	100 mm above crown	Horizontal	4	100 mm above crown	Horizontal	4
C11[a]	680	NA	NA	NA	100 mm below invert	Horizontal	3
C12[a]	680	NA	NA	NA	100 mm below invert	Horizontal	4

Note: NA, not applicable.
[a]Cells C11 and C12 were not installed at the Cambridge, Guelph, and Whitby sites.

and the bedding. In addition, redundancy in the instrumentation reduced the effect of localized conditions on the results of the study and increased the overall reliability of the data collected.

Data acquisition system

The pressure cells were monitored using a portable GK403 readout unit during the construction process. After construction of the test bed was completed, an automated data acquisition system was installed that allowed all subsequent data to be recorded using a multiplexer and a CR10X Campbell Scientific datalogger. The data acquisition systems were programmed to read and store the data six times per day. Data were downloaded on a monthly basis.

Typical installation procedure

A standard installation procedure was adopted for all sites with only a few variations due to site geology. The installation procedure is summarized as follows:

(1) A trench was excavated to the desired depth. The respective depths at the invert pipe sections for the four sites were 2.2 m, 2.8 m, 2.5 m, and 1.8 m for the Cambridge, Guelph, Whitby, and Barrie sites, respectively. The manhole was installed and leveled.
(2) The invert cells were installed within the geotextile wrap with 50 mm layers of fine sand above and below them (a similar protection system was provided to all pressure cells).
(3) The five pipe segments were installed. The end of the last pipe was blocked with a concrete cap to prevent ingress of fill material into the pipe during backfilling.
(4) The trench was backfilled with native soil just above the springline with no compaction effort applied, and the vertical and horizontal springline pressure cells were installed.
(5) The trench was backfilled with native soil to about 300 mm above the crown of the pipe.
(6) The cells for monitoring the vertical crown pressures were then installed in the native soil approximately 100 mm above the top of the pipe. The stress cells were covered with natural backfill material and hand tamped. The trench was then backfilled to grade using native material in lifts of approximately 300 mm. After every lift a light compaction effort was applied using a plate compactor.
(7) At ground level, the soil was compacted using a roller type compactor to 95% standard Proctor density (ASTM 2000*a*).

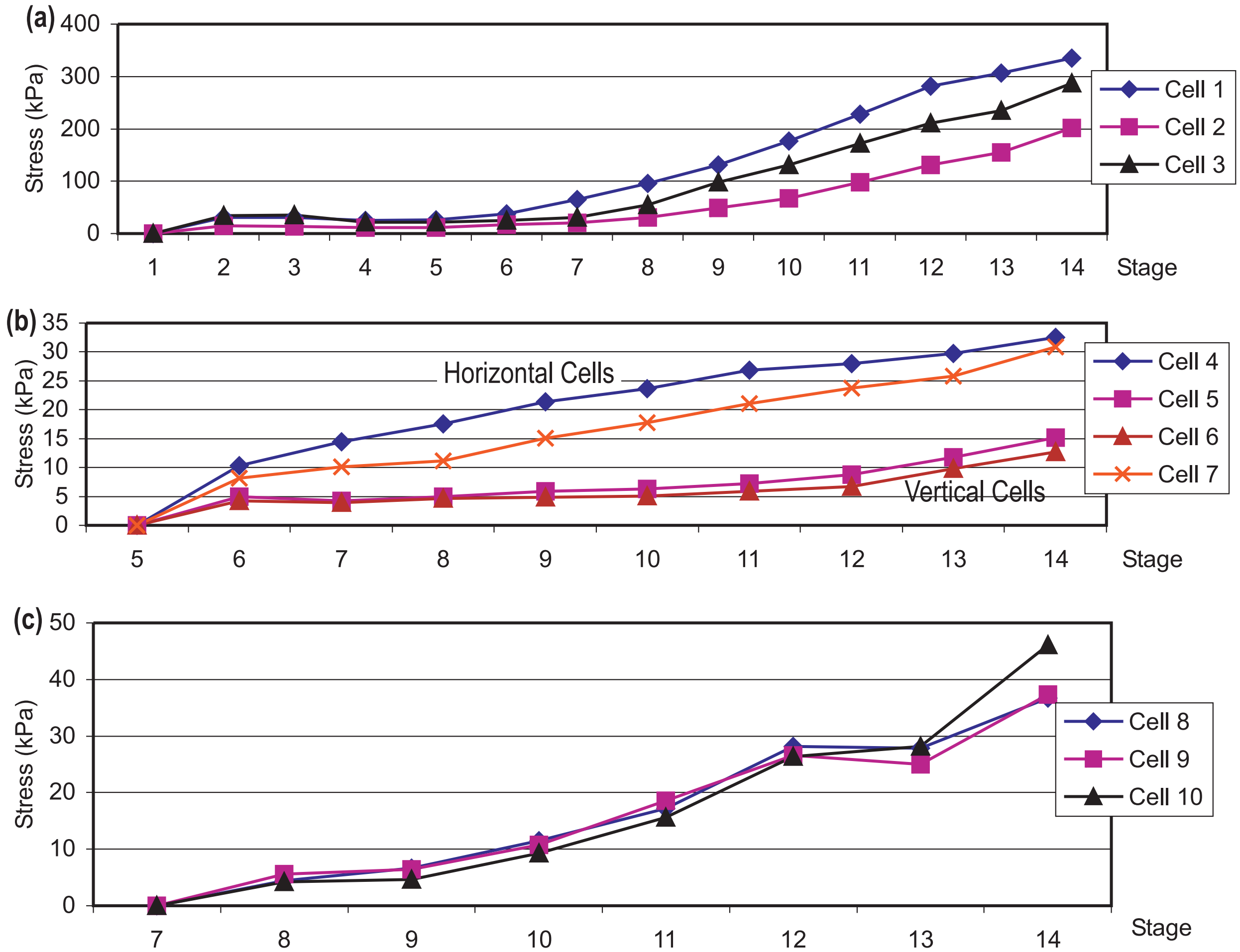
The test bed at the Barrie site was constructed beneath the approach ramp of a proposed new weigh station, which was constructed two weeks after the test bed. The construction of the ramp resulted in a total earth cover of 1.8 m above the pipe segments at the site.

Field measurements

Stresses during construction

Stresses in the soil around the pipe perimeter were monitored using the pressure cells during the construction process. Figure 4 shows the soil stresses measured at different stages of construction (Stages 1–14) at the Whitby site, with each stage defined below the figure. Measurement of stresses at the invert of Stage 1 (Fig. 4*a*) began just prior to placement of the pipe sections. Measurements at the springline and at the crown began at Stage 5 (trench backfilled to the pipe's springline level) and Stage 7 (trench backfilled to just above the top of the pipe), respectively. It can be seen from Fig. 4 that the stress increment was linear from Stage 6 (backfill above the springline level) to Stage 14 (backfill at grade), which indicated equal increments of stress gain during each stage. A light compaction effort was undertaken for the backfill soil using a small vibrating plate compactor for this site. Invert cell readings reported by C1, C2, and C3 show similar incremental trends throughout the construction process that reached 320 kPa, 200 kPa, and 290 kPa, respectively, at Stage 14 (trench backfilled to grade). Stress readings reported by C2 were lower in comparison with C1 and C3. Cells C1 and C3 showed slight drops from Stage 3 (backfilled to 200 mm level) to Stage 4

Fig. 4. Response of pressure cells during construction for the Whitby site. (*a*) Change in stress for invert pressure cells. (*b*) Change in stress for springline pressure cells. (*c*) Change in stress for crown pressure cells.



Stage 1: prior to placement of pipe test section (invert pressure cells only)
Stage 2: after pipe segments were installed (invert cells only)
Stage 3: trench backfilled to 200 mm (invert cells only)
Stage 4: trench backfilled to 400 mm (invert cells only)
Stage 5: trench backfilled to springline level - minimum compaction effort using a small vibrating plate compactor (invert and springline cells)
Stage 6: trench filled to 200 mm above springline - minimum compaction effort using a small vibrating plate compactor (invert and springline cells)
Stage 7: trench filled to just above crown level (all pressure cells)
Stage 8: trench filled to 250 mm above the pipe crown
Stage 9: trench filled to 500 mm above the crown - minimum compaction effort using a small vibrating plate compactor
Stage 10: trench filled to 700 mm above the crown - minimum compaction effort using a small vibrating plate compactor
Stage 11: trench filled to 900 mm above the crown - minimum compaction effort using a small vibrating plate compactor
Stage 12: trench filled to 1.2 m above the crown - minimum compaction effort using a small vibrating plate compactor
Stage 13: trench filled to 1.5 m above the crown - one pass of excavator over excavation
Stage 14: trench filled to grade - one pass of excavator over excavation

(backfilled to 400 mm level). These drops are attributed to the uplift force from the haunch soil.

Readings from the pressure cells located at the springline and crown levels of the pipe have been plotted in Figs. 4*b* and 4*c*. Measurements from different pressure cells at similar locations showed good agreement. Average values of the springline stresses were 32 kPa and 14 kPa in the vertical and horizontal directions, respectively, which provided for a lateral earth pressure coefficient (K_0) of 0.44. The average crown stress (40 kPa) was somewhat greater than that at the springline (32 kPa), thus providing evidence of a negative arching effect. Irregular stress increments were reported by

Table 4. Summary of pressure cell response during selected construction stages.

	Stresses for various construction stages (kPa)			
	Pipe installed without backfill	Backfill to springline	Backfill 300 mm above crown	Completion of the installation backfill to grade
Guelph				
Invert (C1)	69	60	7	276
Hor. spln (C6)	NR	0	7	12
Ver. spln (C4)	NR	0	17	27
Crown (C9)	NR	NR	3	65
Whitby				
Invert (C2)	31	26	65	280
Hor. spln (C6)	NR	0	5.0	16
Ver. spln (C4)	NR	0	17.6	36
Crown (C9)	NR	NR	4	46
Barrie[a]				
Invert (C11)	38	45	84	131
Deeper invert (C8)	9	14	26	74
Hor. spln (C5)	NR	0	10	15
Ver. spln (C4)	NR	0	18	35
Oblique (C3)	NR	8	19	30
Crown (C10)	NR	NR	13	22

Note: NR, no reading was obtained. Hor. spln, horizontal springline; ver. spln, vertical springline.
[a]Prior to ramp construction.

the crown cells between Stages 12 and 14 and can be attributed to the change of compaction method from a small plate compactor to a roller compactor between these stages. This fact was not evident in the invert cells and springline cells because of the lesser effect of compaction at greater depth. Similar patterns of stress variation with backfill level for the different pressure cell groups were noted at the other three sites. A summary of the changes in stress levels for the Guelph, Whitby, and Barrie test sites is given in Table 4. Data provided include stress readings taken immediately after placement of the pipe segment, backfilling to springline level, backfilling to 300 mm above the crown, and completion of construction (backfill to grade level). The lateral earth pressure coefficients, calculated as the ratio of horizontal to vertical total stresses at the pipe springline, were 0.60, 0.40, 0.44, and 0.40 for the Cambridge, Guelph, Whitby, and Barrie installations, respectively.

Some variations in stress readings were observed in all four test sites among pressure cells placed at similar positions because of variable soil conditions and the limitations of repeatability in the installation procedure itself. As expected, maximum stresses occurred at the invert. These invert stresses developed because of the weight of the overburden soil and the pipe. An exception is the Cambridge installation where adequate contact between the pipe and the bedding soil at the location of the invert was not achieved during installation. Therefore, measurements at the Cambridge site are considered only for the crown and springline pressure cell groups in this study. Based on the experience in Cambridge, a more matriculate procedure for placement of the pipe segments in the trench was implemented in the other three field installations. This ensured a uniform contact of the pipe barrel with the bedding along the entire length of each segment.

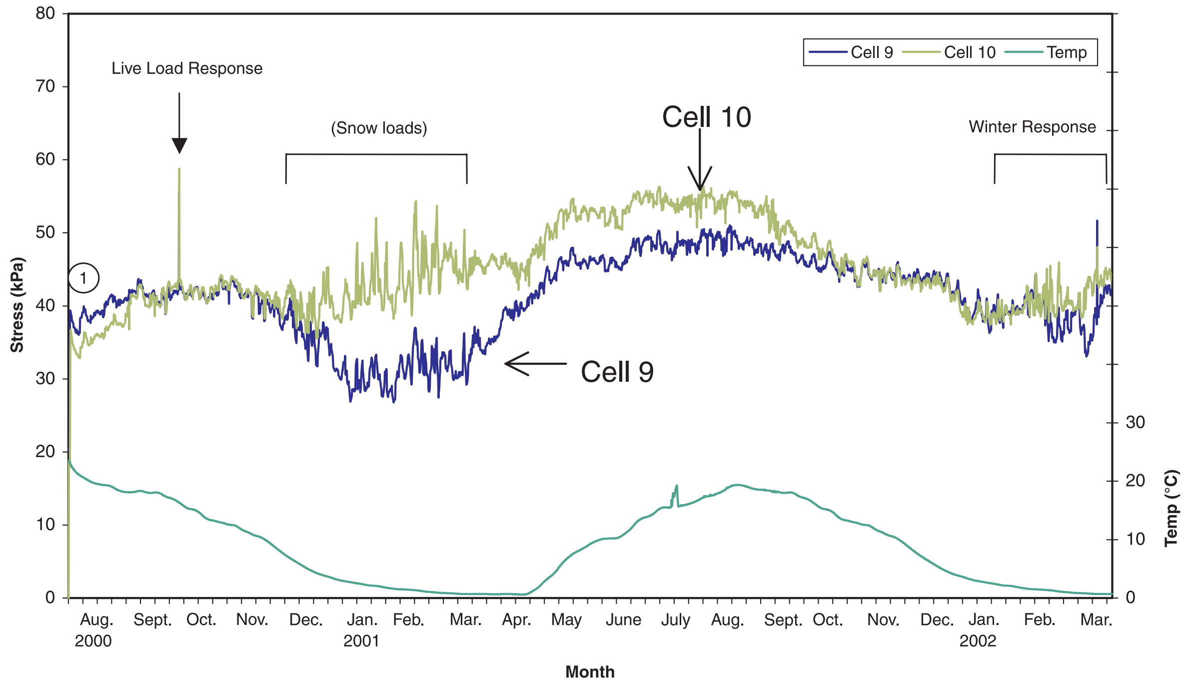
Long-term monitoring

Following completion of the installation a long-term monitoring program commenced for each of the sites. Several general observations were made regarding the long-term monitoring data with respect to consolidation, seasonal oscillations, and the impact of live load.

Pressure cell readings reported by each cell group were in close agreement over the monitoring period. However, fluctuations about an average value were evident, particularly during the winter months. Seasonal oscillations were most pronounced at the Barrie site, potentially due to a more severe winter than in the southern locations, and at the Whitby site where the soil is known to be frost sensitive. During the winter of 2001, 2.5 m of snow were measured at the Barrie site, which resulted in measurable fluctuations in crown cell readings (see Fig. 5). A lesser impact was noted during the winter of 2002 because of a higher mean temperature (i.e., –3 °C compared with –7 °C in 2001 for the months of January and February) and a reduced snowfall. Similar observations were made at all the other sites (Wong 2002). This variation was attributed to the sensitivity of the vibrating wire system to exterior temperature, which seems to be only partially compensated by the manufacturer's calibration chart (Daigle and Zhao 2003). However, since it was not practical to recover and recalibrate each of the earth pressure cells, the manufacturer's provided temperature correction factors were used in the data interpretations. Within each cell's data set, any error will be systematic and its significance mitigated by calculating the monthly mean value for the entire cell group.

To represent the long-term trend of the stresses, the monthly maximum, monthly mean, and monthly minimum values of the pressure cell readings were calculated. Figure 6 shows the statistical summary of the pressure cell readings from 27

Fig. 5. Response of crown cells at the Barrie site. Construction of ramp to truck weigh scale was completed on 27 July 2000. A 200 mm layer of granular "A" soil was placed on top of the original ground profile.



August 2000 to 20 March 2002 for the Barrie site. Stress readings for the other sites are available in Wong (2002).

Overall soil stress variation with time

A long-term change in soil stresses is expected and is due to consolidation of the backfill materials under surface loads. This is particularly applicable for the SIDD Type IV bedding because only a minimal compaction effort was applied to the backfill during installation.

A summary of stresses at the completion of the installation, after 6 months, and at the end of the 20 month monitoring period for the Guelph, Whitby, and Barrie test beds is shown in Table 5. The data reveal that the overall stress in the soil around the pipe increased appreciably with time. The maximum stress increase occurred at the invert of the test beds in Guelph and Barrie. The increase of invert stress at the Whitby site was minor, potentially because of limited traffic activity above the test bed. A higher increase in stress is expected at the pipe's invert because the rigid pipe attracts loads from its surroundings during settlement of the soil, which is transferred to the bedding through the invert. The increase of crown stress was 23%, 34%, and 45%, for the Guelph, Whitby, and Barrie test beds, respectively. Table 5 also reveals that, overall, the stress increase occurred primarily within the first 6 months after installation with little change thereafter. An exception was the Barrie site, where a somewhat more notable increase in stress was recorded following the first 6 months, particularly by the crown cells. The Barrie test bed is located underneath an approach ramp to a newly constructed weigh scale station, which was operational for only a short time period during the fall of 2001, resuming operations in late spring of 2002. The frequent heavy traffic loads (as high as 14 400 kg per axle; Allouche et al. 2004) during late spring and summer of 2002 resulted in the additional compaction of the backfill materials.

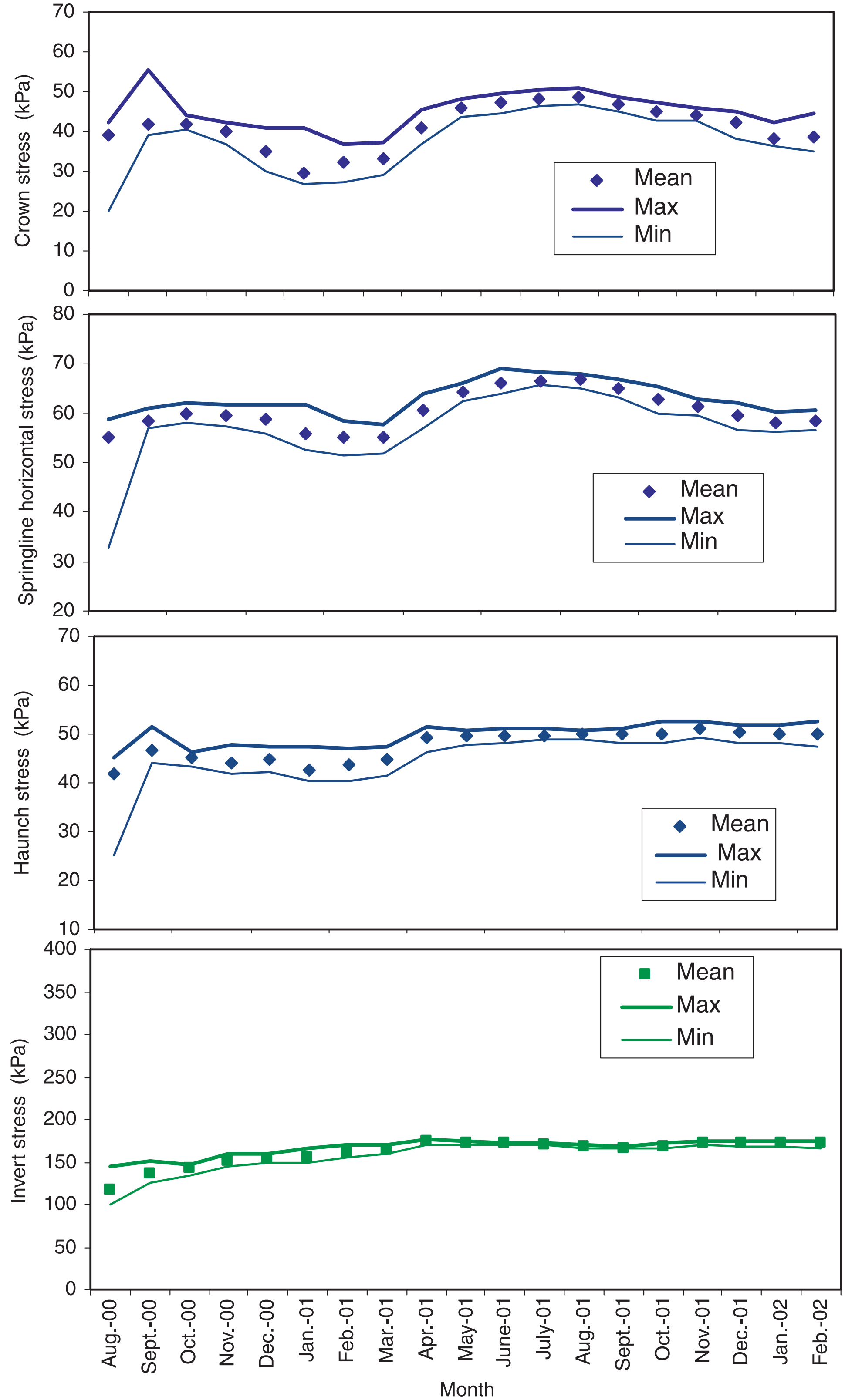
Surface settlements

Located in the thoroughfare of storage yards of pre-cast concrete plants, most of the test beds were subjected to significant and frequent live load. With no compaction effort applied to the bedding and limited compaction effort applied to the backfill, surface settlement was anticipated. While the main objective of the study was to measure soil stresses around the pipe, surface settlement was also of interest. However, efforts to collect systematic data on surface settlement were found to be in conflict with normal operation of the plants, as the storage yard personnel took unscheduled, undocumented remediation actions to fill any settlement above the test beds. Consequently, settlement-related data collection efforts were abandoned.

Comparison with current design standards

Field measurements were compared with the soil stresses predicted by two design standards, SIDD and OPSS 421 (OPSS 1995), to examine the adequacy of these standards in capturing soil stresses that develop around buried rigid pipes installed using limited construction effort (Type IV or Class "D"). A statistical analysis was undertaken to derive representative values that could be compared with calculated values. All data points collected by a given pressure cell group (e.g., crown cells) during each month at a particular test site were grouped into a data set, and the mean value of the data set was computed. The mean value of the monthly means was then computed. Also, the maximum monthly mean and the minimum monthly mean values were identified. Overall, the minimum monthly mean could be considered to represent the stresses in the period immediately following the installation, while the average and maximum monthly means represent the long-term in situ stresses. The statistical summary (maximum, average, and minimum monthly means) of the

Fig. 6. Response of pressure cells at the Barrie site.



soil stresses measured over a period of 20 months was then compared with the values computed by the design standards.

Cambridge site

Table 6 shows a comparison of the long-term soil stress measurements with the calculations made based on SIDD and OPSS 421 (OPSS 1995) for the Cambridge installation. The SIDD calculation of crown stress (σ_{crown} = 48.1 kPa) is approximately 16% higher than the field monthly mean average (41.2 kPa) for the Cambridge site. However, the SIDD

Table 5. Long-term changes in measured soil stresses.

	At the end of installation (kPa)	Stresses in 1st 6 months: Maximum stress (kPa)	Stresses in 1st 6 months: % Increase	Long-term stress: Maximum stress (kPa)	Long-term stress: % Increase
Guelph					
Invert	276	400	45	410	49
Hor. spln	12	14	19	14	19
Ver. spln	27	38	39	38	39
Crown	65	80	23	80	23
Whitby					
Invert	280	282	1	280	1
Hor. spln	16	20	25	20.4	28
Ver. spln	36	41	14	43	19
Crown	41	55	34	55	34
Barrie					
Invert	110	162	47	175	59
Hor. spln	22	26	18	29	32
Ver. spln	54	60	11	67	24
Oblique	39	51	31	52	33
Crown	38	45	18	55	45

Note: Hor. spln, horizontal springline; ver. spln, vertical springline.

value matches well with the field monthly mean maximum stress. A geostatic calculation based on the unit weight (γ) and the burial depth (z) (specified in OPSS 1995) provides a stress value of 21.4 kPa, while the measured values were found to be nearly twice that value. This could potentially be attributed to negative arching developing around the rigid concrete pipe, attracting loads from the surrounding backfill, and causing higher stresses above the pipe's crown.

The measured values for horizontal ($\sigma_{\text{springline(H)}}$) and vertical ($\sigma_{\text{springline(V)}}$) springline stresses range from 11.3 kPa to 15.2 kPa, and 11.9 kPa to 24.8 kPa, respectively. The SIDD standard has no provision for calculating vertical stress at the springline. However, the horizontal stress calculated by SIDD at the springline is 2.1 kPa, a value significantly lower than the measured values. The OPSS 421 standard (OPSS 1995) does not provide stress predictions at the springline.

The field monthly mean average for invert soil stress (σ_{invert}) readings is 39.5 kPa, a value significantly lower than both the SIDD (152.0 kPa) and the OPSS (281.2 kPa) calculations. Poor contact between the invert pressure cells and the bottom of the pipe is believed to be the cause of this discrepancy. Lessons learned from the Cambridge site regarding placement of the instrumentation were implemented in subsequent installations, resulting in greatly improved data quality.

Table 6. General comparison for the Cambridge site.

	σ_{invert} (kPa)	$\sigma_{\text{springline(H)}}$ (kPa)	σ_{crown} (kPa)	$\sigma_{\text{springline(V)}}$[a] (kPa)
Field measurements[b] (monthly mean)				
Max	49.1	15.2	47.8	24.8
Min	9.7[c]	11.3[c]	19.6	11.9
Average	39.5	13.4[c]	41.2	21.1
Design standards				
OPS	281.2	NA	21.4	NA
SIDD (Type IV)	152.0	2.1	48.1	NA

Note: NA, not applicable.
[a]Vertical springline stresses are based on C7 only.
[b]Field measurement for invert stress is taken only from C3.
[c]This value is summarized using limited dataset due to erroneous readings.

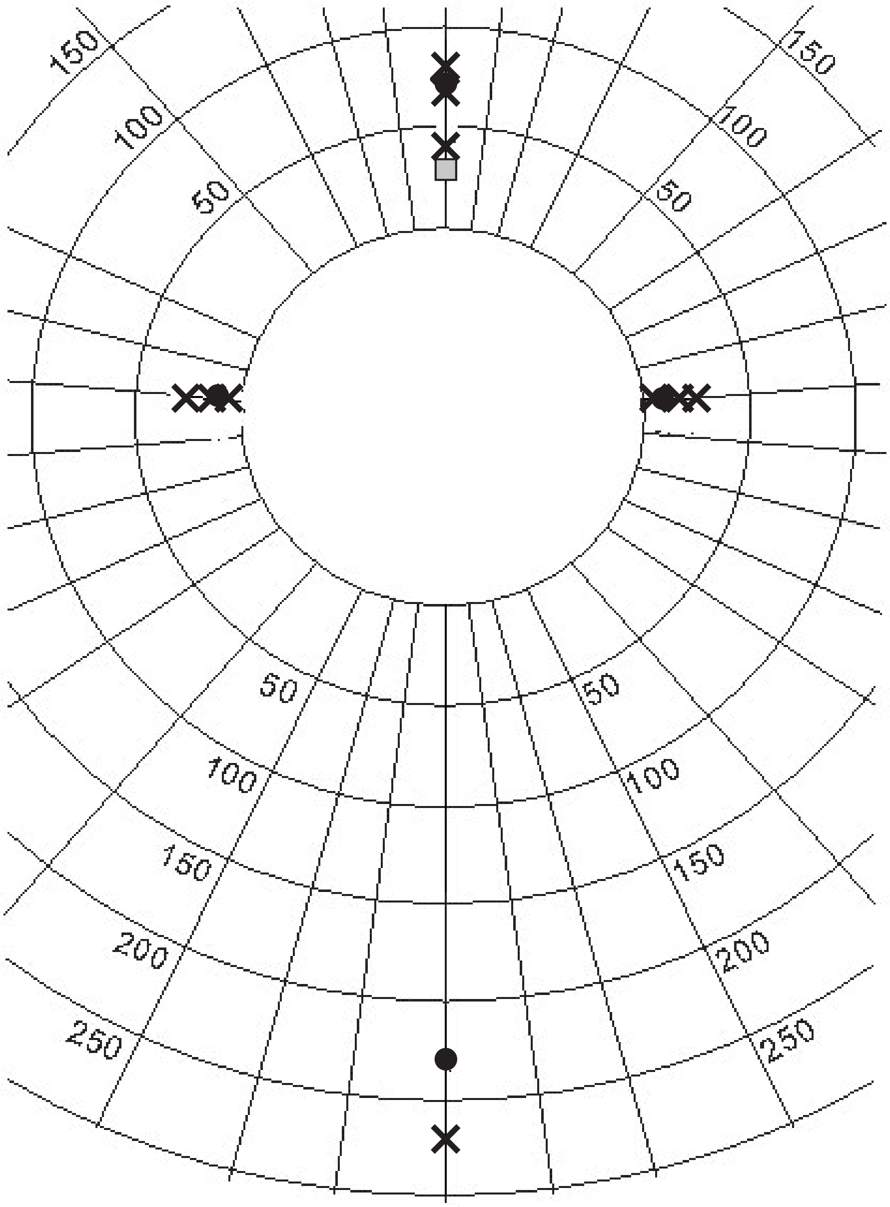
Guelph site

Figure 7 summarizes the radial soil stresses around the pipe from the field measurements at the Guelph site. Calculations made using SIDD and OPSS 421 (OPSS 1995) are also shown. The table beneath the figure shows the numeric values of the stresses used in the comparison. Calculation of soil stress above the pipe using SIDD (73 kPa) falls well within the range of the monthly mean field observation (44 kPa – 80 kPa), however the geostatically computed value (32 kPa) underestimated the field measurements.

The observed lateral springline stress ranges between 2 kPa and 14 kPa for the Guelph site. The SIDD calculation for the lateral stress at the springline of the pipe is 3 kPa, providing a lower bound to the field measurements. Greater lateral soil stress at the springline of the pipe provides higher lateral support, and hence higher confining pressure, which stabilizes the pipe and reduces the bending moment at the pipe invert. Therefore, underestimation of the lateral springline stresses by SIDD provides a conservative estimate for design purposes.

The long-term field measurements of the invert soil stress ranged from 266 kPa to 410 kPa while the readings after installation were 224 kPa to 330 kPa. The SIDD calculation (231 kPa; see Fig. 7) falls into the initial stress range. The OPSS 421 (OPSS 1995) calculation was 480 kPa. Comparison of the calculated and observed stresses at the invert does not yield conclusive results because of the relatively high variation in the field measurements, which is attributed to

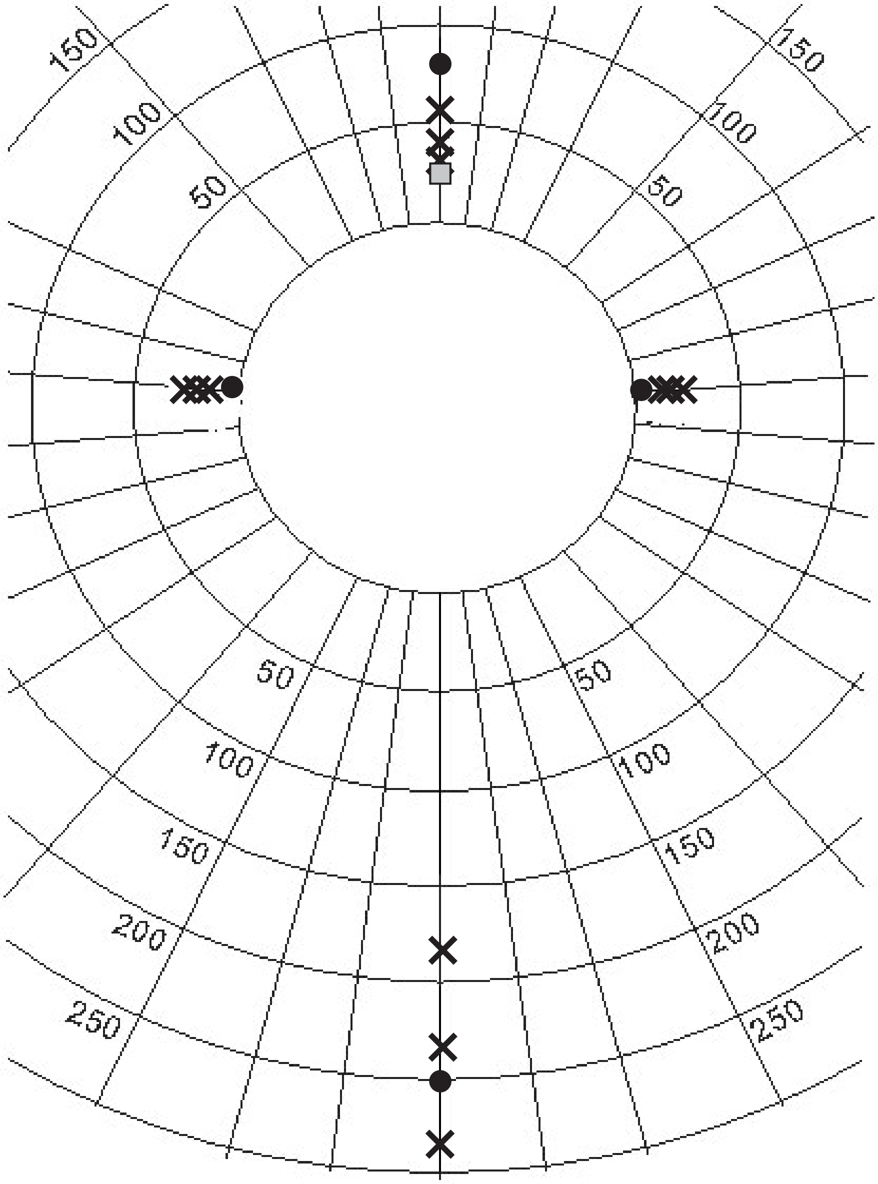
Fig. 7. Predicted vs. measured stresses at the Guelph site.



Legend	Stress (kPa)	σ_{invert}	$\sigma_{springline\ (H)}$	σ_{crown}	$\sigma_{springline\ (V)}$
	Field measurements (monthly mean)				
×	**Max**	410	14	80	38
×	**Min**	266	2	44	21
×	**Average**	386	9	62	29
	Design standard				
□	**OPSS**	480	NA	32	NA
●	**SIDD (TYPE IV)**	231	3	73	NA

the extremely poor soil conditions (cobbles up to 50 mm diameter and 450 mm diameter for the bedding and the backfill, respectively). The calculations from SIDD fell within the lower half of the range of the field observations (265 kPa – 410 kPa), while the OPSS calculation was nearly 20% higher than the maximum stress value recorded by the field instrumentation. Overall, the calculations from SIDD were in reasonable agreement with field measurements, capturing the stress envelope developed around the pipe due to the overburden load.

Fig. 8. Predited vs. measured stresses at the Whitby site.



Legend	Stress (kPa)	σ_{invert}	$\sigma_{springline (H)}$	σ_{crown}	$\sigma_{springline (V)}$
	Field measurements (monthly mean)				
✕	**Max**	280	20.4	55	34
✕	**Min**	181	10.4	29	43
✕	**Average**	236	15.4	39	38
	Design standard				
□	**OPSS**	505	NA	36	NA
●	**SIDD (TYPE IV)**	251	3.0	80	NA

Whitby site

Figure 8 summarizes the long-term field measurements and the calculations made using SIDD and the current Ontario Provincial Standard for the Whitby site. Comparison of crown stresses indicate that the SIDD estimate (80 kPa) is approximately twice the long-term monthly mean average (39 kPa) and 40% higher than the maximum recorded value. The overburden stress obtained using the unit weight (γ) and the burial depth (z) is 36 kPa, which is slightly less than the field range. The negative arching phenomenon was not as pronounced at this site, perhaps due to the geometry of the trench (see Fig. 1).

Similar to the observations at the previously described test sites, the lateral springline stress was underestimated by

SIDD. The SIDD calculation for the lateral stress was computed to be 3.0 kPa, a value 70% lower than the lower bound of the field measurement (10.4 kPa).

The monthly mean average of the field measurements for the invert soil stress was found to be 236 kPa, while the SIDD calculation was 251 kPa, a value within 10% of the measured stress. The current OPSS method calculated a stress of 505 kPa, a value higher by 80% than the maximum monthly mean value observed in the field.

Overall, good agreement was found between the calculations based on the SIDD Type IV installation clauses and the field measurements for the Whitby site. The OPSS 421 (OPSS 1995) prediction was found to be somewhat overly conservative, overestimating by 120% and 80% the average and maximum monthly mean field measurements, respectively.

Barrie site

Figure 9 compares the long-term field measurements of the Barrie site with calculations made using SIDD and OPSS. Field measurements of the invert soil stress show an average monthly mean of 138 kPa, while the SIDD (Type IV) procedure estimate is 246 kPa. Using the relevant clauses of OPSS 421 (OPSS1995), the stress at the invert was estimated to be 548 kPa, a value nearly four times greater than the average monthly mean observed in the field. As a result of the favourable soil conditions at the site (uniform sand) and the greater compaction effort used (a weigh station ramp was constructed above the location of the test bed), the installation was evaluated to be somewhere between SIDD Type III and Type IV. Therefore, a SIDD calculation was performed for each of these two installation types.

In general, the SIDD procedure overestimates the long-term monthly mean average value by 80% to 140% for this installation, depending on whether a Type III or a Type IV clause is used. The OPSS 421 (OPSS 1995) standard overestimates the invert stress by nearly 400%. These results reveal that SIDD provides a reasonable calculation with a safety factor of approximately 1.8, and the current OPSS method is overly conservative.

For a Type IV installation, SIDD assumes zero stress in the haunch region caused by poorly compacted soil that cannot sustain a load. Measurements of soil stress in the haunch region (the monthly mean average values) gave an average of 45 kPa, indicating that the installation of the pipe at the Barrie site closely resembled a SIDD Type III installation. Calculation for a SIDD Type III installation provided a stress of 52 kPa, 14% greater than the measured value, which further supports the aforementioned observation.

Moreover, Fig. 9 shows that SIDD predicts a conservative horizontal stress at the springline. The SIDD Type IV calculation was 85% lower than the monthly mean average of the field measurement (22 kPa). Calculations of crown stresses (78 kPa and 75 kPa for Type III and IV, respectively) were approximately double the long-term monthly mean average recorded at the crown of the pipe.

Influence of horizontal soil stress on concrete pipe design

Field measurements at all four test sites revealed that the horizontal soil stress measured at the springline of the pipe was significantly greater than that calculated using SIDD (by 10, 2.7, 5.1, and 4.5 times for the Cambridge, Guelph, Whitby, and Barrie sites, respectively). Greater lateral soil stress at the springline of the pipe provides a higher confining pressure that supports the pipe at the springline and thus reduces the bending moment at the pipe's invert. Therefore, underestimation of the lateral springline stress represents a conservative estimation for design purposes on one hand, but higher reinforcement requirements on the other hand.

Moments and thrusts at the invert of the concrete pipes at the four test beds were calculated using the software "PIPECAR" (Simpson Gumpertz & Heger Inc. 1994). PIPECAR calculates the resulting bending moments and thrusts in the pipe wall for a given soil stress envelope around the pipe, as well as the amount of steel reinforcement required to resist these loads. The Heger pressure distribution for SIDD Type IV installations was used with varying values for the HAF to calculate the horizontal soil stress at each of the four sites. The HAF is a coefficient used by Heger's earth pressure distribution to estimate the horizontal soil stress at the springline as a function of the prism load, PL. The resulting earth pressure envelope was then put into PIPECAR. Figure 10 shows the invert moments calculated for the pipes at the four sites for a range of HAF values. It can be seen that a less conservative estimate of the HAF results in a noticeable reduction in the calculated invert moment. The larger the pipe diameter, the more significant is the reduction in the calculated invert moment (i.e., the Guelph test bed).

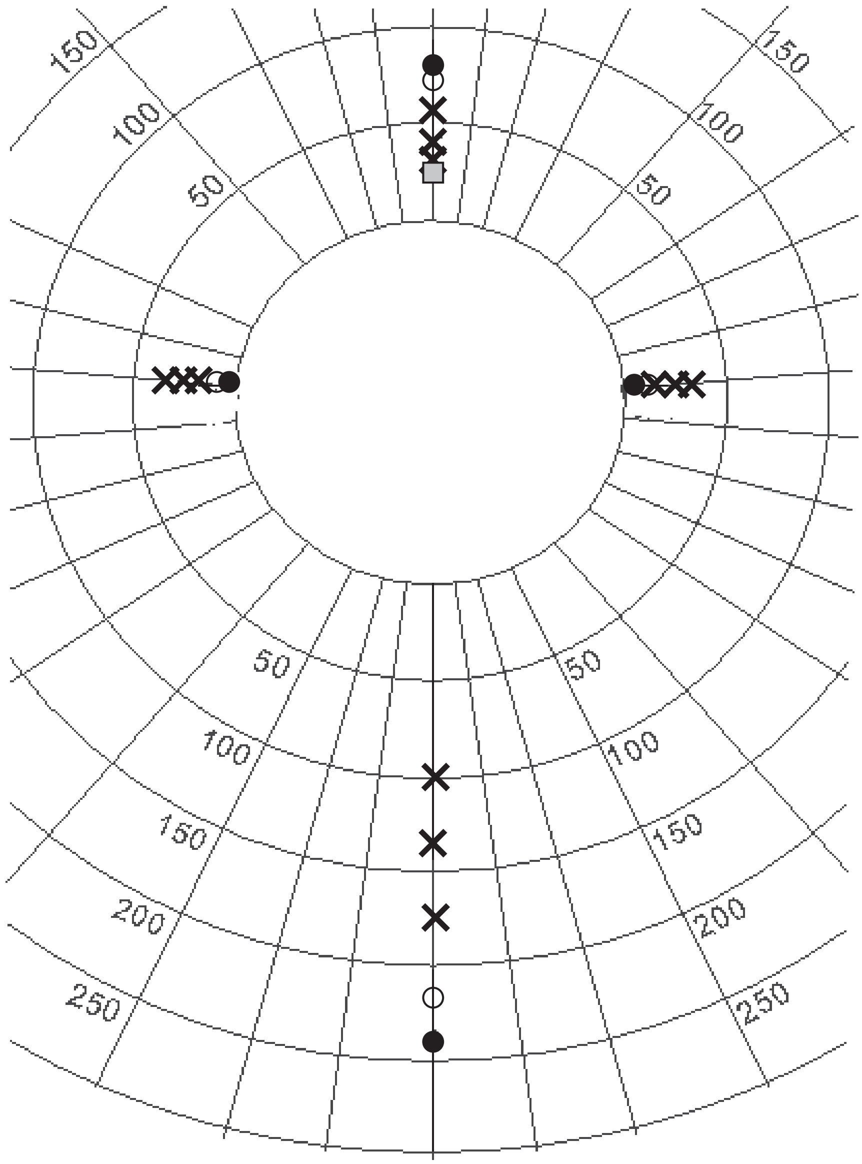
To examine potential savings from a less conservative consideration of horizontal soil stresses, flexural reinforcement requirements (in mm^2/m) were calculated for the four pipes using HAF values raging from 0.3 to 2.4. It was found that for the specific pipe geometries and burial depths used in this testing program, the minimum reinforcement requirement (148 mm^2/m) governs the design of the pipes in Cambridge, Whitby, and Barrie. Thus, no savings can be realized by assuming a HAF value greater than 0.3. Calculation for the Guelph site based on the current HAF value (= 0.3) resulted in a reinforcement requirement of 161.5 mm^2/m. Thus, savings could be realized if a less conservative approach to the horizontal soil stress is taken (as great as 9% for HAF = 0.6). It is recommended to investigate the level of conservatism in the current HAF values for all of SIDD bedding types, potentially resulting in a less conservative set of values.

Summary and conclusions

Field tests performed in this research provide a unique set of short- and long-term experimental data on the behaviour of buried concrete pipe installed according to SIDD Type IV specifications. Data obtained from four full-scale test beds with a wide range of in situ soils and installation conditions has provided a valuable means for evaluating current design codes of practice. In addition, a study of the long-term response has provided an insight regarding changes in the soil stress envelope with time around shallow rigid pipes subjected to repeated traffic loads.

A full-scale test site such as the one used in this study re-

Fig. 9. Predicted vs. measured stresses at the Barrie site.



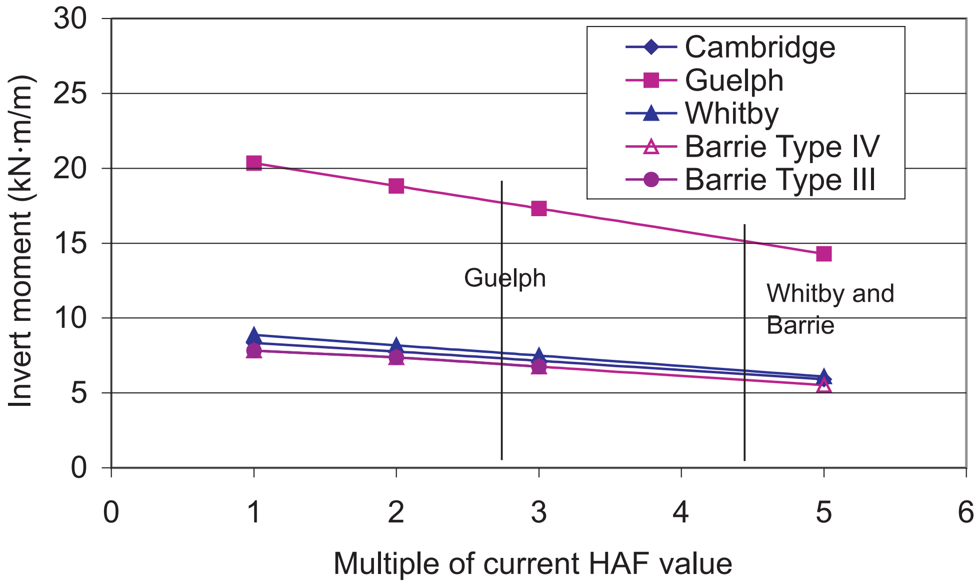
Legend	Stress (kPa)	σ_{invert}	$\sigma_{oblique}$	$\sigma_{springline\ (H)}$	σ_{crown}	$\sigma_{springline\ (V)}$
	Field measurements (monthly mean)					
✕	**Max**	175	52	29	55	67
✕	**Min**	100	40	15	30	51
✕	**Average**	138	45	22	42	60
	Design standard					
□	**OPS**S	548	NA	NA	35	NA
●	**SIDD (TYPE IV)**	246	0	3	78	NA
○	**SIDD (TYPE III)**	220	52	5	75	NA

quires multiple measurement points at each location (e.g., invert) because of the variable nature of the in situ soil and the difficulty of maintaining tight quality control during the construction process. A high degree of care in the placement of the instrumentation and ensuring a uniform contact area between the pipe product and the bedding are essential for obtaining representative values from the instrumentation.

Field measurements reflected certain field activities such as snow and surface traffic. A seasonal oscillation was observed in some of the data sets, likely due to incomplete temperature variation compensation of the load cells as well as temperature induced changes within the soil mass. Seasonal oscillations were most pronounced at the Whitby site, where the soil conditions are known to be frost sensitive and the pipe invert is located just above the water table.

Overall, field measurements at similar locations were

Fig. 10. Calculated invert moment vs. HAF.



found to be in fair to good agreement within each earth pressure cell group (i.e., crown) in each of the test sites. The variations in stress readings observed among the cells placed at similar locations were attributed to variable soil conditions and minor variations in the installation procedure. Higher variations in the invert cell readings compared to other cell groups reflect the sensitivity of those readings to the degree of contact between the pipe's outer wall and the bedding as well as the stiffness of the soil in the haunch.

A general increase of stresses with time was observed at all sites and was attributed to soil settlement under repeated surface activity and the weight of the overburden soil. This increase took place primarily within the first six months following installation, with little increase thereafter. The long-term increase ranged between 20% and 45% for crown stress, 20% and 40% for vertical springline stress, 20% and 30% for horizontal stress, and up to 50% for invert stress.

Several conclusions regarding current design approaches can be made based on the comparison with field measurements.

(1) Overall, the indirect design method currently used by OPSS 421 (OPSS 1995) provides a somewhat overly conservative calculation of the stresses at the invert of rigid pipes installed using Type IV standard installation practices.
(2) The SIDD method provides a reasonable calculation of the vertical stresses at the invert of rigid pipes. However, the method overestimated the stresses at the crown for the Whitby (clayey soil) and Barrie (sandy soil) installations.
(3) The SIDD Type IV installation assumes no stress at the haunch. However, it is clear from field data that the haunch carries some of the load. Therefore, SIDD is conservative in this regard.
(4) The SIDD standard tends to underestimate the lateral stresses at the springline level of the pipe. This results in a conservative estimation of the bending moments at the pipe's invert according to the culvert design software PIPECAR.
(5) The HAF back calculated from the field measurements was found to be in the range of two to ten times the value currently recommended by SIDD for Type IV installations (i.e., 0.3). Based on these observations, it is recommended to further investigate the level of conservatism in current HAF values for all of SIDD bedding types, potentially resulting in a less conservative set of values.

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