Rigid Pipe Distress in High Embankments over Soft Soil Strata

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Two case histories of severe distress in actual rigid pipe installations are presented. They illustrate how soft soils in the region below the outer haunch or adjacent to the pipe within one diameter beyond the sides of the pipe resulted in significant increases in the load on the pipe and the shear and bending stress resultants, producing extensive flexural cracking and failures in diagonal and radial tension. For each installation, the pipe design and installation designs are presented along with a description of the failure. The results of soil-structure interaction analyses using the computer program SPIDA are presented on the basis of two models, one modeling the soft soils that existed at each side and the other modeling the same installation with the soft soils replaced by compact in situ or placed granular soils. The results show how the presence of soft soils under high fills increases the earth load and structural effects on the pipe compared with pipe in conventional installations. The results also show that the pipes in each of the two installations were not properly designed for the 18- to 20-m (60- to 65-ft) finished heights of fill over the pipe, even without the presence of the soft soils.

Pipe loads that exceed the earth load calculated using conventional Marston-Spangler theories for loads on positive and negative projecting embankments and sloping wall trenches (*1*) can occur in installations where the pipe itself is placed on a firm bedding and foundation, but the in situ soil in a subtrench wall, or embankment, adjacent to the bedding or to the pipe itself is soft. Two examples are described in this paper, showing how installations of this type resulted in excessive earth loads on the respective pipe culverts, causing large crack widths and shear and radial tension failures in the concrete pipe. The results of comparative soil-structure interaction analyses for determining the earth load and pressure distribution on the pipe in each installation, with and without the soft soil strata adjacent to the pipe, are presented.

PIPE AND SOIL INSTALLATION AT SITE 1

A precast concrete pipe culvert installed at Site 1 includes approximately 345 m (1,130 ft) of 2700-mm (108-in.) pipe, 98 m (320 ft) of 2850-mm (114-in.) pipe, and 173 m (566 ft) of 3000-mm (120-in.) pipe. The specified pipe strength is Class 5. The culvert pipes were placed in a subtrench cut into the earth using a shaped bedding with a sand cushion 150 mm (6 in.) thick between the lower part of the pipe and the in situ soil. The installation configuration is shown in Figure 1. At the site, the top of the original ground varies within a range between the top of the pipe and the lower quadrant of the pipe. The nature and thickness of the top layer of original ground vary. Most typically it is composed of 1 to 2 m (3 to 6 ft) of medium to soft soils, often with substantial amounts of clay, overlying a sandstone or shale rock substratum. The earth cover above the top of the pipe varies between about 15 and 18 m (50 and 65 ft) throughout most of the finished installation. However, the pipes failed and were repaired before that height of cover was reached.

The type and relative stiffness of the existing soils shown in Figure 1 were obtained from borings taken before construction of the culvert and embankment. The standard specifications used for construction of the embankment did not require removal of the soft soils that overlie the shale or sandstone substratum before placing the embankment.

Generally, the bottom of the pipe is located close to the top of the shale or sandstone. The remainder of the lower portion of the pipe is founded on the 150-mm (6-in.) sand cushion over the medium stiff to soft clay in situ soil that overlies the hard substratum as shown in Figure 1. Note from Figure 1 that the highly compacted select granular soil in the subtrench under and adjacent to the pipe haunches is founded on the medium to soft clay in situ soil below the bottom of the subtrench, whereas the sand layer 150 mm (6 in.) thick at the invert region of the pipe bottom is founded on the much stiffer shale or sandstone substratum.

The pipe manufacturer's design for the specified Class 5 pipe uses an arrangement of circumferential reinforcement consisting of full circular cages located near the inside and outside surfaces and additional mat reinforcement located near the inside surface in 90-degree quadrants centered on the crown and the invert. The pipe also has radial ties (i.e., stirrups) anchored to the inside cages and extending over the crown and invert quadrants. The design wall thicknesses, reinforcement areas, and welded wire fabric sizes used in the pipe are given in Table 1.

Stirrups were prefabricated in three-dimensional panels using cold drawn wire conforming to ASTM A82 [minimum yield strength 448 MPa (65,000 psi) after welding into fabric]. The stirrups are loops of two No. 10 wires resistance welded to No. 7 gauge wires extending longitudinally across the inside of the inside cage. The closed ends of the stirrup loops extend across the wall almost to the outside cage.

Three production pipes in each pipe size were subject to three edge bearing tests. The 0.01-in. crack strengths exceeded the specified 3000D strength by about 15 percent for the 2700- and 2850-mm (108- and 114-in.) pipe and by about 40 percent for the 3000-mm (120-in.) pipe. The ultimate strengths exceeded the specified 3750D strength within a range of about 0 to 20 percent. The test pipe typically failed in diagonal tension in the region containing stirrup reinforcing. The loads that produced diagonal tension (shear) failure exceeded the calculated strength for a pipe without stirrups and are estimated to have developed, or somewhat

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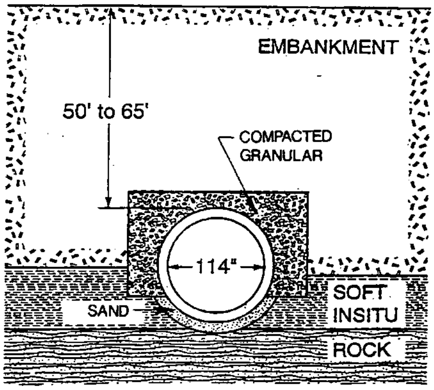

FIGURE 1 Idealization of typical installation at Site 1.

exceeded, the stirrup wire nominal yield strength of 448 MPa (65,000 psi), with failure at the anchorage of stirrups to inner reinforcing cage.

Compression tests on cores 150 mm (6 in.) in diameter taken from three cracked pipes indicated average strengths of 52.9 MPa (7,671 psi), 57.3 MPa (8,306 psi), and 38.1 MPa (5,531 psi), respectively.

When the level of backfill over the pipe culvert reached 12 to 14 m (40 to 46 ft), an inspection of the culvert interior revealed extensive longitudinal cracking with radial displacements and delamination of concrete cover over inner reinforcement in many pipes. Vertical and horizontal diameters were measured in 56 pipe sections. The largest decreases in vertical diameter were 94 mm (3.7 in.), 53 mm (2.1 in.), and 58 mm (2.3 in.) for pipe of diameter 2700, 2850, and 3000 mm (108, 114, and 120 in.), respectively. The largest increases in horizontal diameter were 76, 53, and 33 mm (3.0, 2.1, and 1.3 in.) for pipe of diameter 2700, 2850, and 3000 mm (108, 114, and 120 in.), respectively.

The difference between the measured horizontal and vertical diameters gives an indication of whether the pipe section probably has failed and also of the severity of the failure. The number of pipe sections (of the 56 sections that were measured) with various ranges of difference between measured horizontal and vertical diameter are given in Table 2. A difference between horizontal and vertical deflection greater than about 25 mm (1 in.) is indicative of failure by yielding of inner cage reinforcement, or, more likely, by diagonal or radial tension.

The predominant failure was diagonal tension (shear) and slabbing in the invert region at about 5 or 7 o'clock. This failure consisted of failure of stirrup anchorage at the inside cage, diagonal cracking with radial displacement on the inside surface, and local delamination of inside concrete cover in the vicinity of the failure area. In some of the most distressed pipes, an additional diagonal tension failure occurred just beyond the end of the stirrups and reinforcing mat in the crown region at about 2 o'clock. These shear failures exhibited greater radial offsets than the shear failures in the stirrup-reinforced invert regions. The pipe with diagonal tension failures generally had measured differences between the shortened vertical diameter and lengthened horizontal diameter that were 38 mm (1.5 in.) or greater. Several pipe sections exhibited fine vertical cracking and flaking of concrete on the inside surface of the compression zone at the springline, indicative of the onset of a flexural compression failure at the springline.

SOIL-PIPE INTERACTION ANALYSIS AND DESIGN—SITE 1

Soil-pipe interaction analyses and design studies were performed using the computer program SPIDA (*1,2*) to analyze a representative installation with the 2850-mm (114-in.) inside diameter, 241-mm (9.5-in.) wall pipe used in a portion of the pipeline. The pipe-soil installation was modeled as shown in Figure 2, using two different soils to represent the existing in situ soil above the shale/sandstone foundation. In the first model, this soil was taken as a medium-to-soft silty clay soil, the condition most representative of the worst locations at the site, and was represented by standard 90 and 95 percent CL soils as equivalent to the silty clay in situ soils shown in certain borings located near the pipelines. In the second model, the in situ soil in Layers 2 and 3 of Figure 2 was taken as a very firm silty sand in situ soil and was repre-

TABLE 1 Site 1 Pipe Wall and Reinforcement Design

Inside Diameter mm (in.)	Wall Thickness mm (in.)	Concrete Stress f_c' MPa (psi)	Area of Circumferential Reinforcement			Welded Wire Fabric Reinforcement	Shear Stirrups (Invert and Crown Region)	
			Quadrant Location	Inner mm^2/m ($in.^2/ft$)	Outer mm^2/m ($in.^2/ft$)		Circular Spacing mm (in.)	Area/line $mm^2/m/line$ ($in.^2/ft/line$)
2700 (108)	254 (10)	41.4 (6000)	Crown & Invert	2411 (1.14)	1206 (.57)	50 mm x 200 mm (2 in. x 8 in.) (D9.5/W3.5)	117 (4.6)	360 (.17)
			Springline	1206 (.57)	1206 (.57)		NA	NA
2850 (114)	241 (9-1/2)	41.4 (6000)	Crown & Invert	2919 (1.38)	1460 (.69)	50 mm x 200 mm (2 in. x 8 in.) (D11.5/W4.5)	117 (4.6)	360 (.17)
			Springline	1460 (.69)	1460 (.69)		NA	NA
3000 (120)	279 (11)	41.4 (6000)	Crown & Invert	2665 (1.26)	1333 (.63)	50 mm x 200 mm (2 in. x 8 in.) (D10.5/W4.0)	117 (4.6)	360 (.17)
			Springline	1333 (.63)	1333 (.63)		NA	NA

NA = Not applicable

TABLE 2 Measured Differences Between Horizontal and Vertical Diameters

Pipe Diameter mm (in.)	No. of Section Measured	No. of Sections Within Ranges of Differences Between Measured Horizontal and Vertical Diameters, mm (in.)							
		0-19 (0-.74)	19-36 (.75-1.4)	38-48 (1.5-1.9)	51-74 (2-2.9)	76-99 (3-3.9)	102-125 (4-4.9)	127-150 (5-5.9)	152-175 (6-6.9)
2700 (108)	35	7	3	2	13	3	2	2	3
2850 (114)	8	1	0	1	2	3	1	0	0
3000 (120)	13	7	1	3	1	1	0	0	0

sented by standard 100 percent ML soils as equivalent to the in situ soil.

RESULTS OF SPIDA ANALYSIS AND DESIGN

The results of the soil-structure analyses for the pipe of 2850 mm (114 in.) inside diameter are given in Table 3 for each installation model. The maximum diagonal and radial tension strength of the pipe is governed by the maximum stirrup design factor, SDF, provided in the pipe design given in Table 1. SDF equals the area of stirrup reinforcing per foot of pipe length per line of stirrup times the maximum stirrup stress (yield or developable anchorage stress) divided by the stirrup spacing, circumferentially. The three edge bearing test results showed that the stirrups provided in the manufacturer's pipe design produced an SDF of approximately 1500 to 1650 N/mm/m (2,600 to 2,900 lb/in./ft) and a calculated stirrup stress of 483 to 517 MPa (70,000 to 78,000 psi).

The maximum height of fill that should have been placed on the pipe, on the basis of a SPIDA analysis and design using the load and resistance factors specified in Section 17 of the AASHTO Bridge Specification (*3*) and the ASCE SIDD Standard Practice (*4*), is about 3.7 m (12 ft). The AASHTO and SIDD load factors are 1.3, except that the load factor for thrust is taken as 1.0, for determining ultimate strength based on tensile yield strength and shear and radial tension strength.

A separate SPIDA analysis and design using load factors of 1.0 for failure by tensile yield and shear (diagonal tension) and radial tension and capacity reduction factors specified by AASHTO (*3*) shows that the maximum fill height that causes yielding of the stirrup reinforcement and failure of the stirrup anchorages at the invert is 5.5 to 6.1 m (18 to 20 ft).

If the natural soil in the regions below the shaped bedding had been a firm silty sand soil, instead of the medium to soft silty clay soils at Site 1, the maximum design height of fill for a pipe in the specified installation using the Section 17 design limits would have been 8 m (26 ft). The estimated fill height to cause failure would have been 12.2 to 13.7 m (40 to 45 ft). In this case, the installation meets the requirements for a SIDD Type 2 Installation (*4*). A design for this condition using the SIDD computer program (*5*) with a Type 2 installation indicates a maximum design depth of fill of 8 m (26 ft), as governed by stirrup yield strength, and a maximum depth of cover of about 12.2 m (40 ft) using load factors of 1.0 instead of 1.3, again governed by yield or anchorage failure of the stirrups at about the nominal 448 MPa (65,000 ksi) stirrup yield strength.

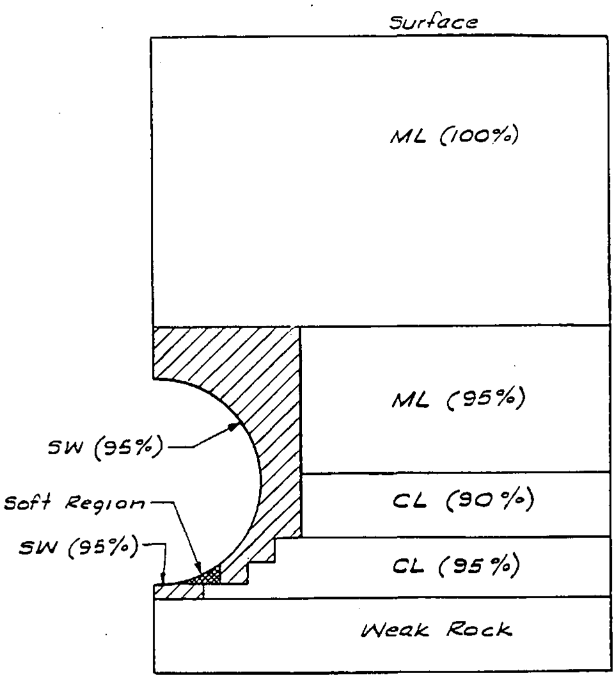

FIGURE 2 Finite element model of pipe-soil installation at Site 1.

DISCUSSION OF SOIL-PIPE INSTALLATION AT SITE 1

The SPIDA soil-pipe interaction analyses show that the specified Class 5 pipe design strength is completely inadequate for a pipe-soil installation with 15.3 to 19.8 m (50 to 65 ft) of backfill over the pipe regardless of the type of soil below and adjacent to the pipe (Cases 2 and 3 in Table 3). The results of the failure analysis for the actual installation (Case 1 in Table 3) shows why many of the pipes exhibited severe distress with major diagonal tension failures probably occurring when the backfill height was considerably less than the 12.2 to 13.7 m (40 to 45 ft) of backfill that was in place at the time that the failure was discovered. The results of the failure analysis for the same installation except that the soft soil is replaced with firm in situ silty sand (Case 4) shows that pipe in this type installation with 12.2 to 13.7 m (40 to 45 ft) of backfill, though very highly stressed, might not have reached a state of visible failure. Since natural soil conditions at the actual site were highly variable, as shown by the borings into original ground before construction, this analysis explains the existence of some pipe without visible evidence of failure.

A comparison of the soil-structure interaction results given in Table 3 for the soft-to-medium in situ clay and the firm in situ silty sand below and adjacent to the pipe shows the following significant characteristics of these installations:

• Vertical arching factor (VAF): The VAF is the ratio of total earth load on the pipe to the weight of a prism of earth directly over the pipe (prism load). Thus, it represents a nondimensional weight of earth on the pipe. In Installation Case 3, with firm silty sand in situ soil adjacent to the shaped bedding and below the subtrench, the VAF is 1.4, representing a magnitude of earth load that is typical of that found in many conventional designs of reinforced concrete pipe in embankment installations. However, in Installation Case 2, with the medium-to-soft silty clay soil below much of the shaped bedding (except near invert) and below the subtrench at the pipe haunch and adjacent side fill, the VAF increased to 1.65. This is because the firm support below the pipe invert causes the pipe to act as a hard object compared with the softer soil adjacent to it. The embankment soil over the pipe behaves as a "shear beam," receiving relatively greater support from the stiff pipe than from the soil adjacent to the pipe, which is underlain by a relatively thin layer of soft soils. As the depth of fill over the pipe in Installation Cases 1 and 2 is increased, as in the analysis for the depth causing failure in Case 1, the VAF increases still more from 1.65 for H = 3.7 m (12 ft) to 1.76 for H = 5.5 m (18 ft).

• Concentration of bearing reaction: The pipe in Installation Cases 1 and 2 with the stiff support below the invert and softer soil below the haunches is also subject to an increased invert moment and shear condition because of the more concentrated bearing reaction as well as the larger total load on the pipe. Because of the low stiffness of the in situ clay soil below the subtrench, the pipe receives little support from the compacted granular soil in the subtrench, and almost all of the bearing reaction is concentrated below the invert region. As a consequence, in spite of the highly compacted select granular embedment soil in the subtrench (see Figure 1), the bottom support reaction is concentrated near the invert instead of being uniformly distributed across the pipe width. This situation was no doubt compounded by the experience of the contractor that construction of the specified shaped bedding (a curved layer of sand) was a constant problem.

• Horizontal arching factor (HAF): The HAF is the ratio of total horizontal load on the pipe to the vertical prism load. The HAF of 0.42 for Installation Case 3 is a typical magnitude for embankment installations in firm, well-compacted granular embedments. The HAF of 0.45 for Installation Case 2 is increased somewhat compared with typical embankments because the very high vertical load and concentration of support near the invert produce a larger extension of the horizontal diameter into the soil embedment at the sides of the pipe, increasing the lateral support forces. The increased lateral support in Installation Case 2 is beneficial but only counteracts a very small portion of the detrimental effects from increased vertical load with this installation.

A review of the soil-pipe interaction analyses that are summarized in Table 3 indicates that an acceptable concrete pipe design for the 15.2- to 19.8-m (50- to 65-ft) height of fill over the culvert at this site requires the following basic design changes:

1. The soft in situ soils below the pipe and the subtrench should have been removed and replaced with select granular soil placed in layers compacted to at least 95 percent of standard Proctor density.
2. The soft in situ soils beneath the embankment for at least one pipe diameter on each side of the pipe section should have been removed and replaced with the same type of site backfill that was used in the main embankment, compacted to at least 95 percent of standard Proctor density.
3. The pipe strength should have been much greater than Class 5. A pipe wall thickness that is greater than the standard A, B, or C wall thicknesses should have been used together with sufficient stirrup reinforcement. An inner and outer circular cage with additional inner reinforcement in the form of mats at the invert and crown was a cost-effective arrangement for the required circumferential reinforcement.

PIPE AND SOIL INSTALLATION AT SITE 2

At Site 2, approximately 268 m (880 ft) of 1350-mm (54-in.) Class 5 concrete pipe was placed in a subtrench cut into the first 3-m (10-ft) height of embankment. This embankment was placed over the original ground surface without removing relatively shallow depths of varying soft soils that were present at most locations along the culvert alignment. However, when the pipe subtrench was cut to the specified depth through the first 3 m (10 ft) of placed embankment and underlying natural soil, any soft soil remaining below the bottom of the trench was removed by order of

TABLE 3 Results of SPIDA Soil-Structure Interaction Analyses for Site 1 Installation

Fill Height Condition	H m (ft)	Arching Factors VAF	HAF	Governing Condition
1. Estimated Maximum H That Produces Pipe Failure (without Water or Live Load) with Actual Soil Conditions	5.5 to 6.1 (18 to 20)	1.76	.46	Shear-stirrup yield and anchorage failure
2. Design Maximum H (with Water + HS20 Live Load) with Actual Soil Conditions	3.7 (12)	1.65	.45	Shear-stirrup yield - SDF = 1500 N/mm/m (2600 lbs/in./ft)
3. Design Maximum H (With Water + HS20 Live Load) if Clay is Replaced with Firm Silty Sand Insitu Soil	7.9 (26)	1.40	.42	Shear-stirrup yield - SDF = 1500 N/mm/m (2600 lbs/in./ft)
4. Estimated Maximum H to Produce Pipe Failure (without Water or Live Load) if Clay is Replaced with Firm Silty Sand Insitu Soil	12.2 to 13.7 (40 to 45)	1.41	.45	Shear-stirrup yield and anchorage failure

the owner's inspectors and replaced with highly compacted natural soil (broken mica schist) from the site. The installation arrangement at Site 2 is shown in Figure 3.

After the embankment was completed to a maximum height of 18.3 m (60 ft) above the top of the pipe, extensive distress was discovered throughout the 1350-mm (54-in.) pipe culvert. A large number of sections of pipe with fill heights in excess of 12.2 m (40 ft) were observed with radial tension (slabbing) or shear (diagonal) tension failures, or both. In addition, many sections with fill heights between 9.5 and 12.2 m (31 and 40 ft), and a few in the range 6.4 to 9.5 m (21 to 31 ft), exhibited this type of failure.

The pipe was manufactured using the Packerhead process. The reinforced concrete pipe design 1350 mm (54 in.) in diameter was provided by the pipe manufacturer and was intended to meet ASTM C76 Class 5 strengths as required in the project specifications. The nominal wall thickness was 140 mm (5.5 in.) (B-wall) and contained inner and outer circular reinforcing cages with 25-mm (1-in.) nominal concrete cover thickness. The nominal area of each reinforcing cage was 1269 mm^2/m (0.60 $in.^2/ft$), and the design concrete strength was 41.4 MPa (6,000 psi). Since no three-edge bearing tests were required, or provided, the design was based on the manufacturer's empirical experience.

A minimum inner cage reinforcement area of 1544 mm^2/m (0.73 $in.^2/ft$) is specified in ASTM C76 for 1200-mm (48-in.), Wall B, Class 5 pipe, which is the largest Wall B Class 5 pipe diameter given in the table. Thus, the manufacturer's design appears to be questionable. The authors estimate that a minimum inside cage area of 1798 mm^2/m (0.85 $in.^2/ft$) is required for 1350-mm (54-in.) Wall B, Class 5 pipe with f'_c = 41.4 MPa (6,000 psi).

Observation of reinforcing in cores cut from a few pipes after the failure by one investigating agency indicates that the actual reinforcement may have been only 1015 mm^2/m (0.48 $in.^2/ft$) or less and that some pipe sections have concrete cover that exceeds the 25-mm (1-in.) specified cover plus the tolerance permitted in ASTM 76. Compressive tests of cores removed from two pipe sections after the distress described above was discovered show compressive strengths of 60.1 and 69.1 MPa (8,720 and 10,030 psi), respectively. Petrographic examinations of some cores show voids and disturbance of concrete structure, common characteristics of pipe made by older Packerhead machines.

SOIL-PIPE INTERACTION ANALYSIS AND DESIGN—SITE 2

Soil-pipe interaction analyses and design studies were performed using the computer program SPIDA (*1,2*) to determine the maximum allowable height of earth fill that could be placed on a 1350-mm (54-in.) pipe with the nominal design properties given above and installed as shown in Figure 3. The pipe-soil installation was modeled as shown in Figure 4, using two different soils to represent soil conditions near the top of the original ground adjacent to the pipe bedding. In the first model, the soil in Layer 3 was taken as a soft silty sand or silty clay soil, the condition most representative of the worst locations at the site. This soil was represented by standard 80 percent ML soil as equivalent to the silty sand and silty clay in situ soils shown in certain borings located near the pipeline. In the second model, the soil in Layer 3 was taken as a firm silty sand in situ soil and was represented by standard 100 percent ML soil as equivalent to very firm silty sand in situ soil.

The SPIDA analysis and design studies give the results shown in Table 4 for the actual soil conditions (Cases 1 and 2). These indicate the following: allowable maximum fill height, 8.4 m (27.5 ft); calculated fill height at failure, 12.8 m (42 ft); and governing failure criterion, shear (diagonal tension) followed by radial tension.

Another SPIDA analysis was performed for the same pipe in an installation where the medium-to-soft silty sand layer on each side of the pipe bedding layer was replaced by a very firm in situ silty sand soil. The results of this study are also given in Table 4

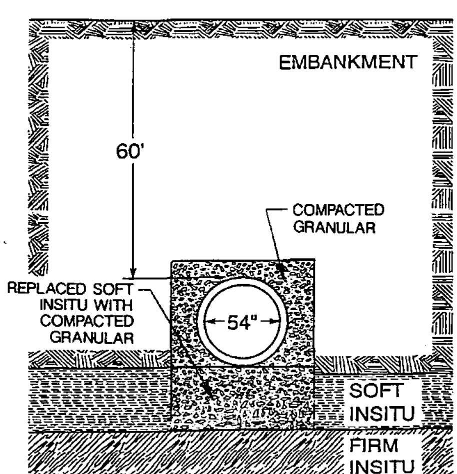

FIGURE 3 Idealization of typical installation at Site 2.

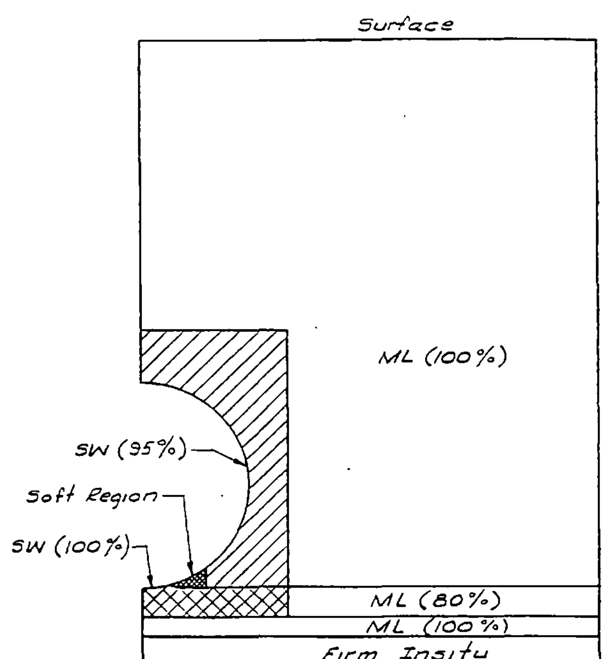

FIGURE 4 Finite element model of pipe-soil installation at Site 2.

TABLE 4 Results of SPIDA Soil-Structure Interaction Analyses for Site 2 Installation

Fill Height		Arching Factors		Governing Condition for Pipe Design
Condition	H m (ft)	VAF	HAF	
1. Estimated Maximum H That Produces Pipe Failure (without Water or Live Load) with Actual Soil Conditions	12.8 (42)	1.72	.44	Diagonal Tension
2. Design Maximum H (with Water + HS20 Live Load) with Actual Soil Condition	8.4 (27.5)	1.72	.43	Diagonal Tension
3. Design Maximum H (with Water + HS20 Live Load) if Soft Soil is Replaced with Very Firm Silty Sand Insitu Soil	13.7 (45)	1.35	.41	Diagonal Tension
4. Estimated Maximum H to Produce Pipe Failure (without Water or Live Load) if Soft Soil is Replaced with Very Firm Silty Sand Insitu Soil	22.2 (73)	1.35	.41	Diagonal Tension

(Cases 3 and 4). When the soft soil layer 600 mm (24 in.) thick is replaced by very firm silty sand in situ soil, the allowable maximum fill height increases to 13.7 m (45 ft). The height of fill that produces diagonal tension failure with load factors of 1.0 is about 22.2 m (73 ft).

When the soft silty soil layer below the embankment on each side of the pipe is replaced by a very firm in situ soil, the installation conforms to a SIDD Type 1 Installation (*4*). An analysis using the SIDD computer program for a Type 1 installation gives a maximum design fill height over the culvert of 12.4 m (40.5 ft). If the load factor is reduced from 1.3 to 1.0, a fill height of 18 m (59 ft) produces diagonal tension failure. These limits are about 11 and 24 percent more conservative than the comparable SPIDA results.

DISCUSSION OF PIPE AT SITE 2

The SPIDA soil-pipe interaction studies show that the subject pipe is overloaded by the specified maximum fill height of 18.3 m (60 ft), regardless of the stiffness of the soil below the embankment on either side of the pipe. However, the presence of soft soil below the embankment in a region adjacent to the pipe foundation on each side of the pipe greatly increases the overload. This is evident from a comparison of both the design maximum fill heights and the VAFs given in Table 4 for the installation cases with soft and firm soils adjacent to the pipe, respectively. The 1.35 VAF calculated for the pipe with firm in situ soil below the embankment adjacent to the pipe is typical of a normal embankment installation. The presence of a layer of soft soil 600 mm (24 in.) thick below the embankment on each side of a pipe that was placed on a firm soil foundation increases the load on the pipe by 27 percent.

The results given in Table 4 show that the capacity of the pipe is greatly enhanced if the 600-mm (24-in.) layer of soft soil below the embankment on each side of the pipe is replaced by firm in situ soil (or compacted granular soil) for about one diameter beyond the pipe. However, a review of the soil-structure interaction analyses shows that even with the improved installation without soft soil, an adequate pipe design that meets the direct design limits given in Section 17 of the AASHTO bridge specification requires either a thicker wall or stirrup reinforcement and greater circumferential reinforcement for the maximum fill height of 18.3 m (60 ft) at this site.

The results of the failure analyses (Cases 1 and 4 in Table 4) show that pipes that have soft soil under the adjacent embankment are expected to have failed in diagonal tension under 18.3 m (60 ft) of fill, whereas pipes with firm in situ soil or compact granular soil in this region may not have reached the failure state in diagonal tension or radial tension. Since the site conditions were variable, this is consistent with observations that not all pipe sections had visible evidence of diagonal tension failure.

CONCLUSIONS

Investigation of the unsatisfactory performance of the concrete pipe in the Site 1 and the Site 2 installations described in this paper leads to the following general conclusions:

1. The SPIDA soil-pipe interaction analyses predict the observed failures for cases with soft soils below the pipe haunches or below the embankment adjacent to the pipes. They also indicate that the pipe at both sites, though overstressed, may not exhibit failure by diagonal and radial tension if soft soils are not present in these regions. Since conditions at both sites are variable, the SPIDA analyses provide valuable insight that corroborates the observed pipe behavior at each of the sites described in the paper.

2. After the improperly designed pipes at Sites 1 and 2 had essentially failed in diagonal and radial tension, they continued to remain intact as they deflected downward vertically and outward horizontally up to 1 to 3 percent of their diameters without collapse. Because of these large deflections, sufficient vertical load was relieved and sufficient lateral load mobilized to enable support of the remaining earth load by ring compression.

3. The design of rigid pipe under high embankments should be based on an adequate investigation of existing soil conditions and consistent with the specified compaction for embankment soils and the embankment subgrade. The design should include a soil-structure interaction analysis to establish the earth load and pressure distribution resulting from the in situ soil conditions and specified embankment soil conditions. The pipe designs at Sites 1 and 2 were based on erroneous and grossly inadequate assessments of the required pipe strength for these sites, installation conditions, and heights of backfill over the pipes.

4. Before placing soils for an embankment, soft soils should be removed in the vicinity of culvert alignments for a distance of at

least one diameter beyond each side of the culvert outside diameter. At both Sites 1 and 2, the embankments were constructed without removing soft in situ soils, but the pipe was founded on firm soil, leading to substantial increases in earth load on the pipe compared with a pipe in a normal embankment installation.

5. Ideally, the stiffness of the bedding or foundation of a rigid culvert should be low under the middle one-third of the pipe diameter (invert region) and sufficiently high under the outer thirds of the pipe diameter to support the full load on the pipe. The reverse condition, in which a stiff foundation or bedding is provided below the invert and soft soil is permitted below the haunches or below a subtrench in the haunch region (the condition that existed at Site 1), greatly increases the bending moment and shear stress resultants on the pipe wall in the critical invert region, leading to premature distress and potential failure.

6. The extreme failure of the pipes with diameters of 2700 mm (108 in.), 2850 mm (114 in.), and 3000 mm (120 in.) at Site 1 resulted from failure to recognize the criteria summarized in Conclusions 3, 4, and 5. The failure of the pipe 1350 mm (54 in.) in diameter at Site 2 resulted from failure to recognize the criteria summarized in Conclusions 3 and 4.

7. It is feasible to design cost-effective precast concrete pipe to provide adequate performance for the backfill heights and general installation conditions at Sites 1 and 2 on the basis of a proper soil-structure interaction analysis and recognition of the interdependence of the design requirements for the embedment and embankment soils and the pipe walls. The traditional method of estimating the pipe loads does not account for the effect of soft soil supporting the embankment adjacent to the pipe and, hence, underestimates the earth load on the pipe.

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