Investigation of Large Deformations of a Corrugated Metal Pipe in Silty Soil

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A 1.83-m-diameter corrugated metal culvert was installed as part of a flood control project in West Williamson, West Virginia. The culvert was installed in a 3.35-m-wide trench, and its crown was up to 9 m below grade. By the end of construction, the culvert experienced deformations that decreased its vertical diameter by as much as 15 percent and increased its horizontal diameter by as much as 10 percent. On the basis of the magnitude of the observed deformations, the owner decided to replace the culvert and to commission a study to determine the cause of the deformation of the pipe. The study was conducted in two phases. The first involved finite element analyses using soil properties based on conventional laboratory tests. The results indicated that the soils in the field must have been considerably more deformable than indicated by the results of the conventional tests. It seemed likely that the deformability of the in situ silty soils at the site might have been increased as a result of the vibratory compaction used to compact the soil around the culvert. The second phase involved finite element analyses using soil parameters from special vibratory loading tests on undisturbed specimens. The laboratory testing program indicated that vibratory loading can cause a significant reduction in the stiffness of silts. The finite element analyses indicated that such a loss of soil stiffness during construction could result in deformations similar to those observed in the field. On the basis of the laboratory test data and the agreement between the results of the analyses and the measured field deformations, it appears that some silts are subject to considerable softening under the influence of vibratory loads. In cases like the one described, where silts are subjected to static loads during vibration, deformations may be considerably larger than those that would occur under the influence of static loads without vibrations.

A corrugated metal pipe was installed as part of the West Williamson L.P.P. Pump Station in West Williamson, West Virginia. The 1.83-m-diameter culvert was constructed in a trench that extended as much as 9 m below the original ground surface into very soft to medium sandy and clayey silts. In some sections of the trench, steel sheet piling was required for stability of the trench walls, and a concrete mud slab was required at the bottom of the excavation over the full length of the culvert to stabilize the bottom of the trench during construction.

The culvert was installed between February 1984 and May 1985. In May 1985, before the installation of the culvert had been completed, large deformations of the culvert were noticed. The vertical diameter of the originally round pipe had decreased by as much as 27 cm, and the horizontal diameter had increased by as much as 19 cm. The deformations were so large that the culvert was considered to have failed, and it was replaced.

The investigation described in this paper was undertaken to determine the cause of the large deformations of the pipe. The investigation was conducted in two phases. In Phase I, finite element analyses were performed using soil properties derived from conventional laboratory tests performed by the Ohio River Division Laboratory of the Army Corps of Engineers. The results of these analyses indicated that the soils in the field had to be considerably more deformable than indicated by the results of the conventional test to explain the large deformations of the culvert. It seemed likely that the deformability of the natural soils at the site might have been increased by the vibrations associated with compaction of the soils around the pipe and removal of the piling at Section 4. Phase II of the investigation was undertaken to investigate this possibility.

Phase II of the study involved (*a*) obtaining undisturbed samples from the site, (*b*) conducting vibratory loading tests on these samples in the laboratory, and (*c*) performing additional finite element analyses using soil properties based on the new laboratory test results and simulating the earth pressures due to compaction in the analyses. Although the laboratory tests did not replicate exactly the type of vibratory loading imposed on the in situ silts, they showed that vibratory loading can have a deleterious effect on the stiffness and strength of silts. The Phase II studies indicated that the vibratory loading could have caused a considerable degree of disturbance in the silty soils. This disturbance is believed to have resulted in significant loss of stiffness of the natural soils adjacent to the trench, thus permitting the excessive deformation of the pipe during backfilling.

This experience affords an important lesson concerning the behavior of silty soils under the action of vibratory loadings. Although many of the details concerning this behavior are not understood yet, it seems clear that silty soils are subject to considerable softening under the action of vibratory loads. In cases where silts are subjected to static loads during vibration, deformations may be considerably larger than those that would occur under the influence of static loads without vibration.

SITE CONDITIONS

At the West Williamson L.P.P. Pump Station project, a 1.83-m-diameter corrugated metal culvert extends a distance of about 83 m from pump house to a manhole. Within 30 m of the manhole, sheet piling was used to support the trench walls. PZ-38 steel sheet piles were driven in interlocked pairs with a gap of about 30 cm between pairs. On the right side of the culvert, most of the sheet piles were removed when the trench was backfilled. Some of the sheet piles on the right side, and all of the sheet piles on the left side, were not removed.

Four sections across the trench and culvert were identified as representative of the varying conditions along the length of the culvert. At Sections 1 and 2, the backfill around the culvert was compacted against natural soils on the right side of the trench and

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against previously placed and compacted fill on the left side of the trench. Section 1 is shown in Figure 1. Section 2 is similar to Section 1 except that random fill was used in place of the impervious fill used at Section 1, and the top of fill at Section 2 was approximately at an elevation of 196.7 m.

At Sections 3 and 4, the trench extended about 9 m below the top of natural ground, and sheet piles were used to support the trench walls. The cross section at Station 3 is shown in Figure 2. At this section, the PZ-38 piling remained in place on both sides of the culvert after backfilling. Section 4 is similar to Section 3 except that at Section 4 the piling on the right side of the culvert was removed during backfilling.

CULVERT PROPERTIES

The culvert was a 1.83-m-diameter corrugated steel circular pipe with 1.27-cm by 6.77-cm corrugations and 8 gauge (4.27-mm) metal thickness. Under normal conditions, this pipe would be considered adequate to withstand as much as 27 m of cover. The maximum cover thickness at the West Williamson project was only about 5 m. The pipe would also be considered capable of withstanding the H-20 highway loading with as little as 0.30 m of cover over the crown.

The 27-m maximum and 0.30-m minimum cover depths are based on analytical and empirical data and are consistent with standard practice. Typical specifications for design of such culverts require that deflections be no larger than 5 percent of the diameter, and standard designs meet this requirement under normal conditions of backfilling and loading. The deflections of the culvert at the West Williamson project were as much as 14 percent of the diameter. These large deformations indicate that the loading on the pipe exceeded normally encountered loadings in some way or that the support provided by the surrounding soil was less effective than normal.

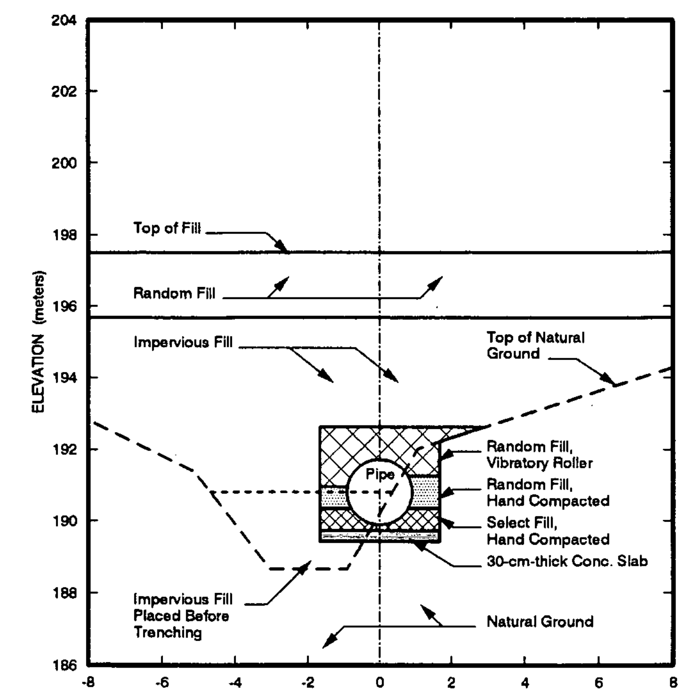

FIGURE 1 Section 1, West Williamson 1.83-m-diameter culvert.

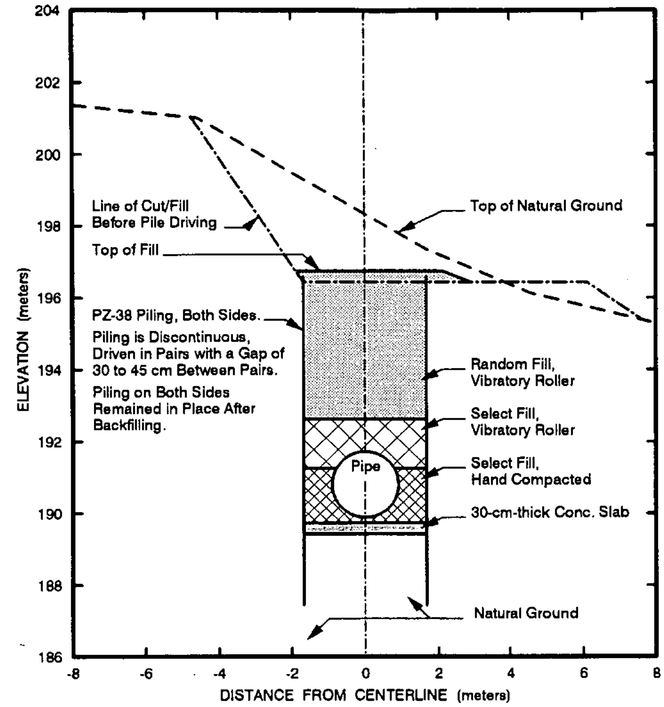

FIGURE 2 Section 3, West Williamson 1.83-m-diameter culvert.

CULVERT DEFORMATIONS

The culvert deflections measured in May 1985 are summarized in Table 1. The vertical diameter of the culvert decreased as much as 26.7 cm, and the horizontal diameter increased as much as 19.0 cm. At Section 3, where the piling remained in place on both sides, the horizontal and vertical diameters changed by more than 15 cm. At Section 4, where the deformations were the largest, detailed measurements revealed a noticeable bias in the horizontal movements, with a shift of the culvert to the right. This bias may be due to the sloping ground surface or the removal of the piling on the right side at this section, or both.

SOIL CONDITIONS

From the ground surface down to approximately the elevation of the culvert, the natural soil is a dark brown sandy silt with low to medium plasticity and very soft to medium consistency. Underlying the brown silt is a gray to dark gray sandy silt with low plasticity and very soft to medium consistency. Observations of the position of the groundwater level in July 1985 indicated that the phreatic surface was about 0.5 m below the invert elevation of the culvert.

Plasticity, water content, and consistency data for the natural soils from the site are summarized in Table 2. A total of 14 samples of brown silt were classified. Of these, 10 were plastic and 4 were nonplastic. For most of the plastic samples, the plasticity

TABLE 1 Deflections of 1.83-m-Diameter Corrugated Metal Culvert at the West Williamson L.P.P. Pump Station (Measured May 1, 1985)

Station (meters)	Change in Diameter† (cm) Vertical	Change in Diameter† (cm) Horizontal	Note
71.0	-6.4	-7.6	Pump plant
74.1	-14.0	2.5	
77.1	-10.2	7.6	
80.2	-17.8	7.6	Section #1
83.2	-16.5	8.9	
86.3	-20.3	10.2	
89.3	-11.4	10.2	
92.4	-20.3	10.2	
95.4	-17.8	11.4	
98.5	-16.5	8.9	Section #2
101.5	-6.4	3.8	
104.5	-6.4	1.3	
107.6	-1.3	1.3	Open end
--	--	--	No pipe
132.6	-15.2	15.2	Section #3
135.6	-15.2	15.2	
138.7	-22.9	17.8	
141.7	-20.3	15.2	
144.8	-26.7	19.1	Section #4
147.8	-17.8	15.2	
150.9	-20.3	8.9	
153.9	0.0	0.0	Manhole

† Based on initial horizontal and vertical diameters equal to 1.83 meters. Measurements were reported to the nearest 1.3 cm (0.5 inch).

data plotted above the A-line on the plasticity chart, resulting in a classification of CL according to the USCS. This classification may be somewhat misleading because the observed behavior of the soils was more "silty" than "clayey." The samples were less sticky than many CL materials and were more easily disturbed during trimming and handling. In addition, they contained layers of fine sands and nonplastic silts. Laboratory tests on four samples of the gray silt, summarized in Table 2, indicate that it is slightly less plastic than the overlying brown silt and that it has a slightly higher average water content in situ. The gray silt also contained layers of fine sand.

Even though many of the samples of the natural soils at this site are classified as clays of low plasticity, the writers believe that the engineering behavior of the soils differs appreciably from the be-

TABLE 2 Plasticity, Water Content, and Consistency of Brown and Gray Silt, West Williamson L.P.P. Pump Station (from Army Corps of Engineers, Ohio River Division Laboratory Tests)

Soil	Property	Minimum	Maximum	Average
Plastic Brown Silt (10 samples)	Liquid Limit	27	44	35
	Plastic Limit	17	25	22
	Plasticity Index	10	19	13
	Water Content	18.2	37.5	28.3
	Consistency	very soft to medium		
Non-Plastic Brown Silt (4 samples)	Water Content	18.2	27.4	23.9
Plastic Gray Silt (4 samples)	Liquid Limit	26	42	32
	Plastic Limit	17	28	21
	Plasticity Index	7	14	11
	Water Content	23.8	43.2	29.1
	Consistency	very soft to medium		

havior of typical CL soils and that they are in fact properly called silts. Accordingly, they are called silts throughout this paper.

The backfill materials are zoned as shown in Figures 1 and 2 and consist of the following:

- Select granular backfill (SM-SC): At all sections, this material was placed and hand-compacted beneath the invert and the haunches of the pipe. At Sections 3 and 4, this material also was compacted over the crown of the culvert using a small vibratory roller.
- Random backfill (CL): When used to fill over the crown of the pipe, this material was compacted using a small vibratory roller. When placed further from the pipe, it was compacted using a sheepsfoot roller.
- Impervious backfill (CL): This material was used at Section 1 and was compacted using a sheepsfoot roller.
- Unreinforced concrete: A 30.5-cm-thick unreinforced concrete slab was placed at the base of the excavation throughout the length of the culvert.

The concrete slab was used to provide a working surface. This was necessary because the slit at the bottom of the excavation was easily disturbed and quickly became muddy as it was traversed by men and machines.

PHASE INVESTIGATION

The first phase of the investigation began in September 1985. The culvert was inspected before removal, and soil samples were obtained for testing. Laboratory testing was done by the Ohio River Division Laboratory in Cincinnati. The results of the tests were used to obtain soil parameters for use in finite element analyses that were undertaken to determine the cause of the large deformations of the culvert.

Soil Properties for Phase I Finite Element Analyses

Several laboratory tests were performed on the natural soils and the compacted backfill materials to evaluate the strength and modulus parameters needed for the finite element analyses. The testing program included: (*a*) unconsolidated undrained (Q) triaxial tests on samples of the brown silt, (*b*) consolidated undrained (R) triaxial tests on samples of the gray silt, (*c*) consolidation tests on samples of the gray silt, (*d*) consolidated drained (S) triaxial tests on recompacted samples of the select backfill material, and (*e*) classification tests. Detailed test results are presented by Duncan and Sehn (*1*).

On the basis of the results of the laboratory consolidation tests and construction records, it seems likely that the gray silt would have sufficient time for dissipation of excess pore pressures and should be modeled using strength and stiffness parameters based on data from drained tests. With the data available during Phase I of the investigation, it was believed that the brown silt would probably experience undrained loading, and strength and stiffness parameters for this material were based on data from undrained laboratory tests.

No laboratory test data were available for the impervious backfill or the random backfill. For these materials, the parameters needed for the analyses were estimated on the basis of typical hyperbolic parameters for compacted soils reported by Duncan et al. (*2*). The typical values reported by Duncan et al. (*2*) are based on the analysis of several hundred triaxial tests on various types of soil and, in the absence of laboratory data, provide a reasonable means of estimating the hyperbolic parameters for a soil based on soil classification and relative compaction.

The finite element analyses performed during this investigation used hyperbolic stress-strain relationships (*2,3*) to model the nonlinear stress-strain and volume change behavior of soils. On the basis of the data available during Phase I, the parameters given in Table 3 were believed to represent the best estimate of the material properties. The information used to derive these parameter values is summarized below the table.

The deformations calculated using the parameters in Table 3 were considerably smaller than the observed deflections. For this reason, a second set of parameters, intended to represent the weakest and most deformable possible behavior of the soils at the site, was selected for use in further analyses. These properties are summarized in Table 4 and are termed ''softened'' soil properties. The properties selected for the backfill materials were chosen to represent the possible existence of very poorly compacted backfill adjacent to the culvert, even though none was found. The basis of selection of these parameter values is summarized below the table.

Phase I Finite Element Analyses Procedures

The Phase I finite element analyses simulate the placement of backfill around and above the culvert and the loadings imposed on the culvert by compaction equipment. In the finite element mesh used for the analyses, the culvert was represented by 12 beam elements, and the soil was represented by 156 two-dimensional quadrilateral or triangular elements. At sections where piling was present, the piling was modeled by beam elements. Each analysis began with the culvert in place and the backfill up to the springline. The analysis then proceeded in a number of steps to simulate placement of the remaining backfill and application of equipment loads over the culvert. The equipment loads were represented by vertical nodal point forces, and no changes were made in the soil properties to represent the effects of vibrations during compaction. This is one of the major differences between the Phase I and the Phase II analyses.

Cases Analyzed in Phase I

The finite element analyses performed during Phase I are summarized in Table 5. A total of 22 cases were analyzed. The cases include study of (*a*) the four sections described previously, (*b*) the effects of the strength and stiffness of the soil on the calculated deflections, (*c*) the influence of the concrete slab on the calculated deflections, and (*d*) the effects of equipment loading and backfill cover depth on the deflections and bending moments in the culvert.

Two types of equipment loads were investigated. One was a 15.4-Mg crawler tractor, represented by a load of 6.37 Mg per meter of length of the culvert. This load was divided among four nodes to represent the loads from two tracks of the tractor. In all cases, twice the actual load was applied to study the possible effects of load concentration on the calculated values of deflection and bending moment in the culvert wall.

The second type of equipment loading was used to model the vibratory compactor used to compact the backfill around the culvert and was modeled by a line load of 2.23 kg per meter of length of culvert to represent its static weight. Studies by D'Appolonia et al. (*4*) indicate that the loads applied by vibratory compactors are higher than the static weight of the compactor. For the size of the compactor being modeled here, it is likely that loads as high as three to four times the static weight may be produced. Loads as high as five times the static weight were used in the analyses.

Results of the Phase I Analyses

The results of the Phase I analyses are summarized in Table 5. The largest calculated deflections (Anal. 7) were a 3.3-cm decrease in vertical diameter and a 3.8-cm increase in horizontal diameter. These deflections are for the full fill loading at Section 4 with the softened soil properties. The calculated deflections are only about one-sixth to one-seventh of the observed deflections. The heaviest roller load (Anal. 16) caused only about 2.3 cm of vertical deflection and 1.8 cm of horizontal deflection. Even if these values are added to those calculated in Anal. 7, the resulting deflections would be only about one-fourth as large as those measured.

The results of the Phase I analyses indicate that the finite element analyses did not adequately represent the field behavior. It was concluded that the silty soils in the field may have deformed under the combination of static and vibratory loads imposed on them during the backfilling operations or during the removal of piling at Section 4. Under the influence of the vibratory loads, the silts might have become even softer than modeled by the "softened" soil parameters. The Phase II investigation described subsequently was undertaken to investigate this possibility.

PHASE II INVESTIGATION

The objectives of the Phase II investigation were to develop a better understanding of the behavior of silts when they are subjected to static and vibratory loads and to use this information in

TABLE 3 First Estimate of Material Properties

Parameter	Material Number					
	1	2	3	4	5	6
	Gray Silt (ML)	Concrete (24 MPa)	Select Backfill (SM-SC)	Brown Silt (ML)	Random Backfill (CL) (RC = 90)	Imperv. Backfill (CL) (RC = 95)
K, modulus number	250	326,200	145	65	90	120
n, modulus exponent	0.75	0.00	0.60	0.00	0.45	0.45
R_f, failure ratio	0.70	0.55	0.45	0.75	0.70	0.70
ϕ_0, friction angle	35	40	44	0	30	30
$\Delta\phi$, friction increment	0	0	7	0	0	0
c (kPa), cohesion	0	11,975	0	115	9.6	14.4
K_b, bulk mod. number	230	155,300	100	1,150	80	110
m, bulk mod. exponent	0.5	0.0	0.5	0.0	0.2	0.2
ρ (Mg/m^3) density	2.08	2.40	2.24	2.00	2.00	2.08

Notes:
Material 1 - K, n, R_f, ϕ_0, $\Delta\phi$, c, and ρ from lab tests on "undisturbed" samples; K_b and m estimated.
Material 2 - Estimated properties for 24 MPa compressive strength concrete.
Material 3 - K, n, R_f, ϕ_0, $\Delta\phi$, and c from lab tests on lab-compacted samples; K_b and m estimated.
Material 4 - K, n, R_f, ϕ_0, $\Delta\phi$, c, and ρ from lab tests on "undisturbed" samples; K_b and m estimated.
Material 5 - Estimated density and properties.
Material 6 - Estimated density and properties.

conjunction with finite element analyses to explain the large deformations of the culvert.

To study the behavior of the silts, high-quality samples were obtained from the site using a 12.7-cm-diameter fixed-piston thin-walled sampling device. The samples were tested in the soil mechanics laboratory at Virginia Tech. On the basis of the information from the laboratory tests, soil parameters were selected for use in the analyses of the culvert. During the Phase II analyses, the effects of compaction were included in two ways. The effects of the vibrations during compaction were represented by using information from special vibratory loading triaxial tests in evaluating the soil properties, and the effects of compaction on the stresses and deformations were represented using the analytical methods developed by Duncan and Seed (*5*).

Phase II Laboratory Tests and Material Properties

Laboratory testing during Phase II of the investigation included (*a*) classification tests, (*b*) unconsolidated undrained (Q) triaxial tests on undisturbed specimens of the gray and brown silts, (*c*) unconsolidated undrained (Q) triaxial tests on remolded specimens of the brown silt, (*d*) a consolidation test on a specimen of the brown silt, and (*e*) special vibratory load triaxial tests on specimens of the gray silt. Details of the testing program and the test results were reported by Duncan and Sehn (*1*).

The purpose of the vibratory load triaxial tests was to investigate the effects of vibration loading on the strength and deformation characteristics of the natural silt soils from the project site. The vibratory triaxial testing consisted of (*a*) applying a static deviatoric stress, equal to about 50 percent of the expected undrained strength, at a constant rate of loading; (*b*) holding the load at the 50 percent level for about 5 min, (*c*) applying five sets of cyclic loading, with each set consisting of 10 cycles of loading between 25 and 75 percent of the expected undrained strength of the specimen, followed by 5 min at the 50 percent load level; and (*d*) constant rate of strain loading to failure or nearly to failure followed by observing creep behavior. The test sequence was not intended to model exactly the field loading conditions, but it provides a reasonable basis for estimating the effects of vibratory loading on the properties of the silts.

TABLE 4 Softened Material Properties

Parameter	Material Number					
	1	2	3	4	5	6
	Gray Silt (ML)	Concrete (24 MPa)	Select Backfill (SM-SC) (RC=85)	Brown Silt (ML)	Random Backfill (CL) (RC=85)	Imperv. Backfill (CL) (RC=85)
K, modulus number	**120**	326,200	**100**	**32.5**	**60**	**60**
n, modulus exponent	0.75	0.00	0.60	0.00	0.45	0.45
R_f, failure ratio	0.70	0.55	**0.70**	0.75	0.70	0.70
ϕ_0, friction angle	35	40	**33**	0	30	30
$\Delta\phi$, friction increment	0	0	0	0	0	0
c (kPa), cohesion	0	11,975	**9.6**	115	**4.8**	**4.8**
K_b, bulk mod. number	**115**	155,300	**50**	**575**	**50**	**50**
m, bulk mod. exponent	0.5	0.0	0.5	0.0	0.2	0.2
ρ (Mg/m^3) density	2.08	2.40	2.24	2.00	2.00	2.08

Notes:
Material 1 - K and K_b reduced to 50% of first estimate, other values unchanged.
Material 2 - Estimated properties for 24 MPa compressive strength concrete.
Material 3 - Estimated density and properties.
Material 4 - K and K_b reduced to 50% of first estimate, other values unchanged.
Material 5 - Estimated density and properties.
Material 6 - Estimated density and properties.
Values that differ from the the first estimate presented in Table 3 are in boldface type.

TABLE 5 Summary of Phase I Finite Element Analyses and Results

Anal. No.	Sect. No.	Soil	Loading and Special Conditions	$\frac{M_{max}}{M_p}$	Change in Horizontal and Vertical Diameters (cm) Calculated h	Calculated v	Measured h	Measured v
1	1	First Est.	Fill loading only	0.136	1.3	1.5	7.6	17.8
2	2	First Est.	Fill loading only	0.015	1.5	1.3	8.9	16.5
3	3	First Est.	Fill loading only	0.300	2.0	2.3	15.2	15.2
4	4	First Est.	Fill loading only	0.300	2.0	2.3	19.0	26.7
5	1	Softened	Fill loading only	0.265	2.3	2.5	7.6	17.8
6	2	Softened	Fill loading only	0.226	2.0	2.3	8.9	16.5
7	4	Softened	Fill loading only	0.492	3.3	3.8	19.0	26.7
8	1	First Est.	Fill loading only, no conc. slab	0.207	1.5	1.8	7.6	17.8
9	2	First Est.	Fill loading only, no conc. slab	0.145	1.3	1.3	8.9	16.5
10	4	First Est.	Fill loading only, no conc. slab	0.380	2.8	3.0	19.0	26.7
11	4	First Est.	Crawler, 2X static, symmetric, 25 cm of cover	0.349	1.3	0.5	19.0	26.7
12	4	First Est.	Crawler, 2X static, load at right, 25 cm of cover	0.426	1.3	1.8	19.0	26.7
13	4	First Est.	Crawler, 2X static, symmetric load, no cover	0.345	1.3	0.5	19.0	26.7
14	4	First Est.	Crawler, 2X static, load at right, no cover	0.544	1.0	1.8	19.0	26.7
15	4	First Est.	Vib. Comp., 5X static, load at left, no cover	1.350	1.3	0.8	19.0	26.7
15	4	First Est.	Vib. Comp., 4X static, load at left, no cover	1.080	1.0	0.5	19.0	26.7
15	4	First Est.	Vib. Comp., 3X static, load at left, no cover	0.804	.08	0.3	19.0	26.7
15	4	First Est.	Vib. Comp., 2X static, load at left, no cover	0.530	.05	0.3	19.0	26.7
16	4	Softened	Vib. Comp., 5X static, load at left, no cover	1.520	2.3	1.8	19.0	26.7
16	4	Softened	Vib. Comp., 4X static, load at left, no cover	1.210	2.0	1.3	19.0	26.7
16	4	Softened	Vib. Comp., 3X static, load at left, no cover	0.903	1.5	1.0	19.0	26.7
16	4	Softened	Vib. Comp., 2X static, load at left, no cover	0.595	1.0	0.8	19.0	26.7

The results of a test on a specimen of the gray silt are shown in Figure 3. For this specimen, the axial strain increased from about 5.0 percent to about 7.8 percent during the vibratory loading portion of the test.

The results of another vibratory load triaxial test on the gray silt are shown in Figure 4. In this figure, the inset is an expanded view of the cyclic load portion of the test. For each cycle of load, there is an increment in the permanent deformation of the specimen, and although the incremental deformation caused by each successive load cycle decreases, the 50th load cycle still produced a small increment of permanent deformation.

On the basis of the results of the laboratory testing program, the idealized behavior shown in Figure 5 was developed. The solid lines show the general relationship between typical laboratory test

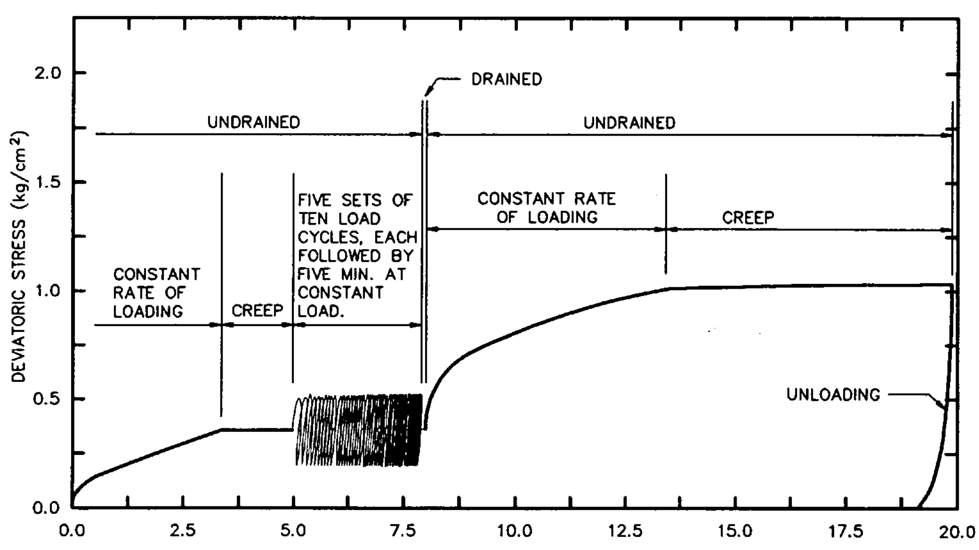

FIGURE 3 Vibratory loading triaxial test on gray silt, West Williamson L.P.P. Pump Station (Sample UD-101-S2-C1).

results from triaxial testing on undisturbed and disturbed or remolded specimens. The line from Point A to Point B represents the axial deformation of an undisturbed specimen due to the cyclic loading portion of a vibratory load triaxial test. The dashed stress-strain curve represents the idealized response of a specimen subjected to static and vibratory loads and is, qualitatively, the type of response used in the finite element analyses to model the effects of vibratory loads on the soil properties.

The material properties used for the Phase II finite element analyses are given in Table 6.

Phase II Finite Element Analyses Procedures

The Phase II finite element analyses modeled the conditions at Section 4, where the largest deformations occurred. The Phase I analyses showed that for a given set of soil properties, the calculated deformations at Sections 3 and 4 were always equal and were larger than those calculated for Sections 1 and 2. These relative magnitudes are consistent with the field measurements.

Since the Phase I analyses showed that the unsymmetric ground conditions were not a significant factor in the calculated distortion of the culvert, the Phase II analyses used a mesh representing one half of the culvert and the backfill. This models the culvert as symmetrical about the centerline and allows a finer mesh to be used. The culvert was represented by eight beam elements, and the soil was represented by 125 two-dimensional elements.

In addition to the factors considered in the Phase I investigation, the Phase II investigation includes the effects of compaction on the properties of the silt, as explained earlier, and the effects of earth pressures induced by compaction.

Analyses were performed using both Properties 4a and 4b in Table 6 to represent the brown silt.

Results of the Phase II Analyses

The results of the Phase II finite element analyses are summarized in Table 7. The calculated deflections for all of the cases are in reasonable agreement with the values measured at Section 4. For the three cases analyzed, the calculated vertical deflections ranged from 58 to 73 percent of the measured values, and the calculated horizontal deflections ranged from 80 to 96 percent of the measured horizontal deflection.

On the basis of the agreement between the measured and calculated deflections, it appears that the analyses performed reflect the actual behavior of the culvert. Since the major difference between the Phase I analyses and the Phase II analyses was in the simulation of the effects of vibration on the stiffness and strength of the silt, it seems clear that these effects played a major role in the large deformations suffered by the culvert during construction.

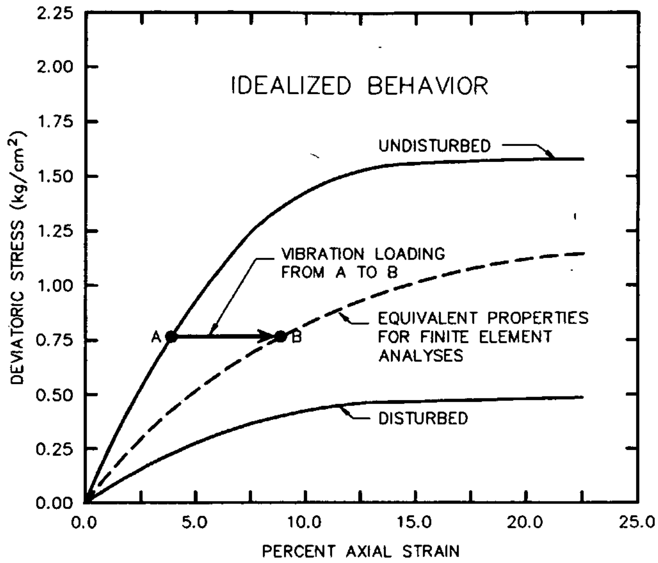

FIGURE 5 Effect of vibratory loading on stress-strain behavior.

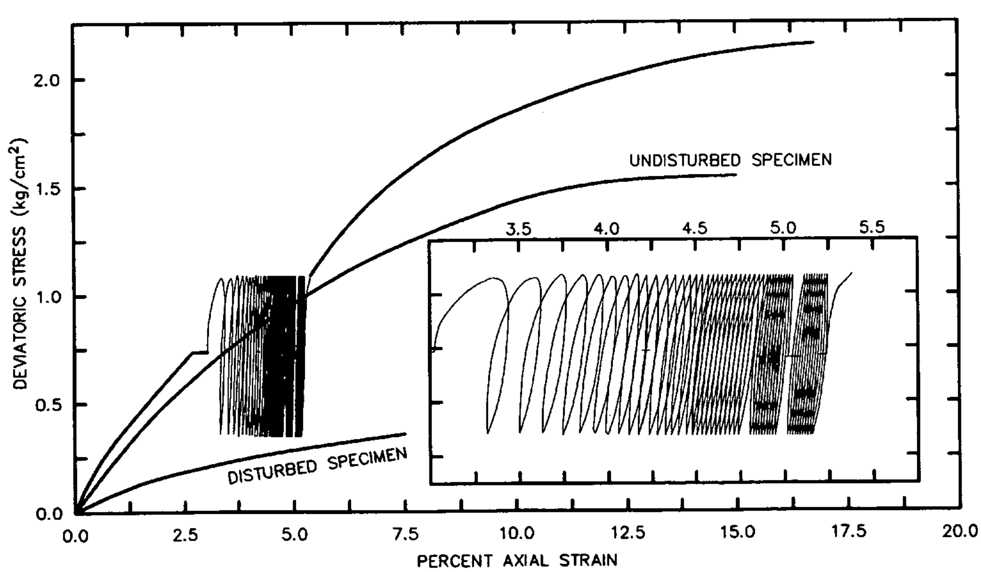

FIGURE 4 Vibratory loading triaxial test on gray silt, West Williamson L.P.P. Pump Station (Sample UD-100-S4-A3).

TABLE 6 Material Properties Used for Phase II Finite Element Analyses

Parameter	Material Number					
	1	2	3	4a	4b	5
	Gray Silt (ML)	Concrete (24 MPa)	Select Backfill (SM-SC)	Brown Silt (ML)	Brown Silt (ML)	Random Backfill (CL) (RC = 90)
K, modulus number	**14**	326,200	145	**50**	**30**	90
n, modulus exponent	**0.00**	0.00	0.60	0.00	0.00	0.45
R_f, failure ratio	**0.65**	0.55	0.45	**0.90**	**0.80**	0.70
ϕ_o, friction angle	0	40	44	0	0	30
$\Delta\phi$, friction increment	0	0	7	0	0	0
c (kPa), cohesion	**57**	11,975	0	**67**	**57**	9.6
K_b, bulk mod. number	**240**	155,300	100	**850**	**510**	80
m, bulk mod. exponent	**0.0**	0.0	0.5	0.0	0.0	0.2
ρ (Mg/m^3) density	2.08	2.40	2.24	2.00	2.00	2.00

Notes:
Values that differ from the the first estimate presented in Table 3 are in boldface type.

TABLE 7 Summary of Phase II Finite Element Analyses and Results

Anal. No.	Sect. No.	Loading and Special Conditions	$\frac{M_{max}}{M_p}$ Note 1	$\frac{M_{max}}{M_p}$ Note 2	Change in Horizontal and Vertical Diameters (cm)			
					Calculated		Measured	
					h	v	h	v
1	4	Compaction-induced earth pressures and loads, Brown silt as material 4a in Table 6	1.45	1.10	17.8	18.3	19.0	26.7
2	4	No compaction-induced earth pressures or loads, Brown silt as material 4a in Table 6	1.22	0.92	15.2	15.5	19.0	26.7
3	4	Compaction-induced earth pressures and loads, Brown silt as material 4b in Table 6	1.78	1.34	18.3	19.6	19.0	26.7

Note:
1) The values of M_{max}/M_p in this column are based on a moment required to form a plastic hinge in the culvert wall, M_p, of 4900 N-m per meter of culvert assuming that the culvert material has a yield stress of 276 MPa.
2) The values of M_{max}/M_p in this column are based on a moment required to form a plastic hinge in the culvert wall, M_p, of 6500 N-m per meter of culvert assuming that the culvert material has a yield stress of 366 MPa.

CONCLUSIONS

On the basis of the results of the investigation of the large deformations of the culvert at the West Williamson L.P.P. Pump Station, the following conclusions were reached:

1. Vibratory loading has a large effect on the undrained stress-stain properties of the silt at the site. The results of unconsolidated undrained triaxial tests with vibratory loading showed that the strains under undrained conditions were increased by about 80 percent by the application of 50 cycles of vibratory loading.
2. The deformations of the culvert calculated in the Phase II finite element analyses are in substantial agreement with the deformations measured in the field. Since the major difference between the Phase I and the Phase II analyses was the fact that the Phase II analyses modeled the effects of vibration on the stress-strain behavior of the silt, it appears that the effects of the vibration on the silt are the principal reason that the deflections of the culvert were so much larger than expected. As the silts deformed because of the vibratory loading, their deformation allowed the compacted backfill and the culvert to spread horizontally, leading to the large deformations that were measured in the field.
3. Since the silts at the West Williamson L.P.P. Pump Station site do not appear to be unusual, similar behavior can be expected in other cases where silts are subjected to vibratory loads.
4. The behavior of silts under vibratory loading is complex, and many aspects of their behavior are still not well understood. Research into the effects of stress conditions during vibration, the effects of vibration frequency, and the effects of the number of load cycles on the stress-strain behavior and strength of silts would be desirable.

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