

# Full-Scale Field Test of Uplift Resistance of Corrugated Metal Pipe Culvert

B. H. KJARTANSON, R. A. LOHNES, F. W. KLAIBER, AND B. T. MCCURNIN

Recent surveys of transportation agencies regarding corrugated metal pipe (CMP) culverts have indicated cases of longitudinal flexural failure due to uplift at the inlet. The surveys indicate that inlet tie-down design standards vary by more than 500 percent in the calculated tie-down force required for similar large diameter CMP. In response to these problems, Iowa State University (ISU) is carrying out a multi-phase research project to develop a rational design methodology for CMP inlet tie-downs. As a part of the ISU project, a full-scale field test involving a 16-m long and 3.05-m diameter CMP installed in a simulated highway embankment was conducted to gain insight into the soil-structure interaction processes and to obtain data for use in the development and verification of a CMP tie-down design methodology. Uplift forces were applied at the "inlet" end of the CMP by a system of hydraulic jacks mounted on a reaction frame. CMP deflections, strains, and backfill earth pressures were monitored as the CMP was incrementally loaded to give a maximum inlet uplift of about 76 mm. The test data indicate that the backfilled CMP subjected to inlet uplift forces behaves essentially as a cantilever beam, with the greatest bending occurring near the crest of the embankment slope. Analyses indicate that the backfill soil cover significantly increases the effective longitudinal stiffness of the CMP through soil-structure interaction processes and thus must be taken into account in a CMP inlet tie-down design.

The cost benefits associated with the use of corrugated metal pipe (CMP) culverts in place of small bridges are particularly realized in rural areas where a great number of roadways cross small streams. However, there have been some uplift problems with CMP culverts in recent years. A recent survey of transportation agencies (1) indicates cases in which longitudinal flexural failure of CMP have occurred due to bending up (uplift) of the inlet. In some instances, the entire CMP has been dislodged from its existing location (2), creating extremely dangerous conditions. The uplift failures occurred with steel culverts so the current study of CMP does not include aluminum culverts. High storm flows and/or partial blockage of the inlet can lead to a significant hydraulic head differential between the CMP inlet and outlet (3). Seepage beneath the pipe under these conditions would result in a triangular pore pressure distribution along the underside of the CMP, assuming a uniform loss of total hydraulic head from the inlet end to the outlet. Moreover, under these flow conditions, the water level outside of the CMP inlet can be higher than that inside the CMP, resulting in buoyancy effects in the CMP inlet region. Many CMP are installed with the minimum amount of soil cover and no additional hold down anchorage. In some cases, restraints (tie-downs) are installed at the ends of the CMP to prevent uplift.

A recent survey of transportation agencies regarding CMP tie-down design standards revealed that a variety of tie-down methodologies are currently being used across the country (1). The various tie-down methodologies being used by different agencies result in divergence of the magnitude of required tie-down forces by more than 500 percent for similar large diameter CMP. Thus, a more rational analysis needs to be developed to obtain the best possible combination of safety and cost for CMP tie-down designs.

In response to the problems of CMP uplift failure in the state of Iowa, the Iowa Department of Transportation (DOT) has funded a multiphase study which is being conducted at Iowa State University (ISU) to develop a rational design methodology for CMP tie-downs. Due to the lack of CMP longitudinal strength and stiffness data required in the hold-down force design process, one phase of the CMP research project involved the development of an analytical model to estimate the longitudinal bending stiffness and strength of CMP of any diameter, gage, or corrugation style. The ISU analytical model was verified through the full-scale load testing of two CMP sections without soil backfill (4).

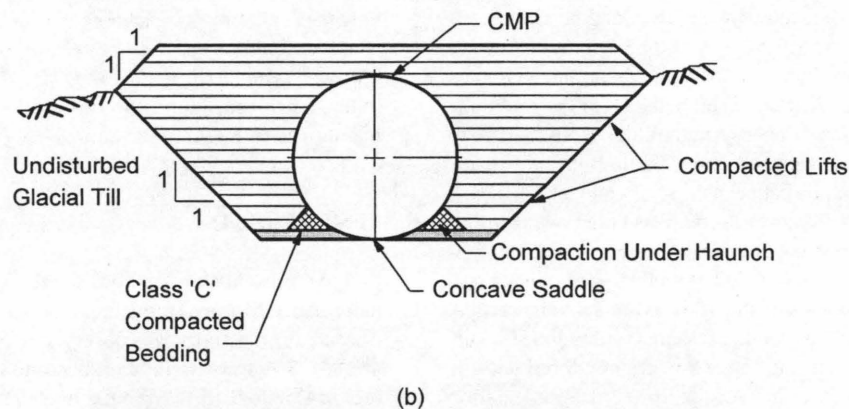
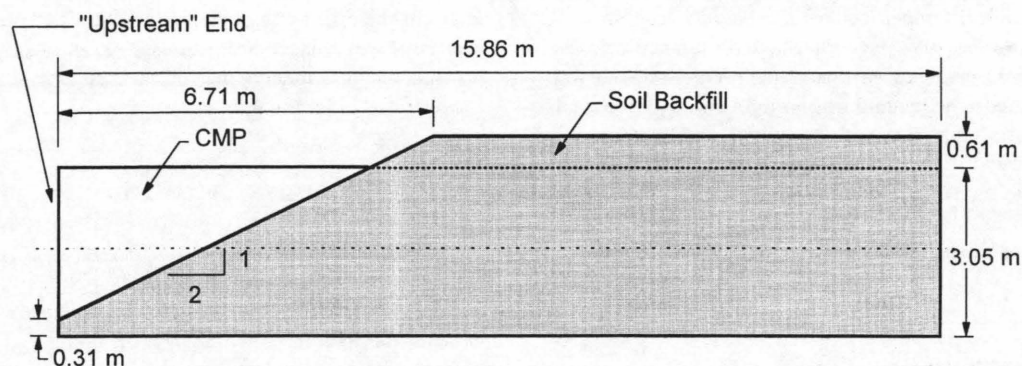
In a typical field situation, however, the CMP behavior depends not only on its own strength and stiffness characteristics but also its interaction with the surrounding soil backfill. A full-scale field test of a buried CMP was conducted as a part of the ISU project to gain insight into the soil-structure interaction processes and to obtain data for the development and verification of CMP tie-down design methodologies. This study presents the results of this full-scale field test and a preliminary analysis of the data obtained.

## FIELD TEST DESCRIPTION

### Design and Installation

The most critical conditions for CMP inlet uplift involve large diameter CMP with minimum soil cover. Therefore, a 3.05-m diameter CMP with 0.61 m of soil cover was tested in a simulated highway embankment with a 2:1 foreslope (Figure 1a). To form the embankment, a trench approximately 4.3 m wide at the base with side slopes of 1:1 was excavated in a knob of undisturbed glacial till (see Figure 1b). Details for a Class "C" bedding were followed in preparing the base (5). Specifications require 10 percent of the CMP height to rest in a saddle cut from compacted soil or natural ground. Accordingly, after 0.31 m of backfill was placed, a template was used to form the concave saddle. Fill around culvert pipes should be compacted to 90 percent of the maximum dry density obtained from a standard Proctor density test (5); tests carried out on soil samples obtained from the site indicated a maximum dry unit weight of 18.9 kN/m<sup>3</sup> at an optimum moisture content of 12 percent.

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**FIGURE 1** CMP test installation: (a) longitudinal profile; (b) transverse section showing excavation and backfill compaction details; (c) test setup before backfilling.

Following construction of the concave saddle, two sections of 3.05-m diameter, galvanized steel CMP, 8.24 m and 7.62 m in length respectively, were placed and aligned in the saddle and connected with a 0.25 m wide band (see Figure 1c). Both CMP sections supplied for the test were 10 gage, with  $3 \times 1$  helical corrugations and continuously welded seams, with the exception of the last 1.53 m of the 7.62 m "downstream" section which was 8 gage. This 8-gage "downstream" section of the CMP was 9.75 m away from

the position where the load was applied to the CMP; therefore, the difference in gage was considered insignificant. To prevent a premature joint failure during the uplift testing of the CMP, the connection between the two sections of CMP was strengthened by welding thirty-four 6.4 mm thick, 63.5 mm wide by 0.46 m long steel plates around the inside circumference of the joint.

The confined haunch areas located near the base of the CMP required compaction with 51 mm/ti 102-mm timber "studs"

and a small mechanical tamper. Above the haunch region, work areas allowed the use of larger mechanical tampers. Loose lifts approximately 0.2 m thick were placed and evenly spread before compaction. The sand cone method was used for compaction quality control to confirm compliance with specifications. Data from five levels within the backfill give an average dry unit weight of 17.3 kN/m<sup>3</sup> and average moisture content of 12.3 percent. Backfilling alternated from side to side of the CMP so that the two fills were kept at approximately the same elevation at all times.

### Test Apparatus and Instrumentation

The unbalanced triangular pore pressure distribution resulting from seepage beneath the pipe, thought to be the driving force for many inlet uplift failures, was simulated by using two load points located 1.53 and 4.58 m from the "upstream" end of the CMP. As shown in Figure 2, uplift was provided by four hollow core hydraulic cylinders reacting on an overhead load frame. The loads from the hydraulic cylinders were transmitted through a system of high strength rods to 0.38-m wide steel straps that passed under the bottom half of the CMP. A grout mix was pumped between the straps and the CMP corrugations to ensure that the load would be distributed over the full 0.38-m strap width thus deterring any local failures.

Data collected during the test included strains on the inner surface of the CMP, deformations of the CMP cross section, vertical deflection of the top of the CMP, and pressures within the soil surrounding the pipe. The electronic sensors were monitored and the output was recorded using a computer controlled data acquisition system (DAS) whereas the vertical deflection measurements were taken manually during the uplift portion of the test. Details on the individual instruments employed are as follows.

Six longitudinal sections were strain gaged to measure longitudinal and hoop strains at the peaks of corrugations on the inner surface of the pipe (see Figure 3a). Two gages (one longitudinal and one circumferential) were placed at each of four locations (the top, bottom, and both ends of the horizontal diameter) at all six sections (see Figure 3b). This arrangement of strain gages allowed for calculation of the hoop to longitudinal stress ratio factor required to calculate the longitudinal bending stiffness using the ISU analytical model [see Equations 1 and 10 of Havens et al. (4)]. In addition, these gages allowed for tracking of the position of the CMP neutral axis during longitudinal bending. Note, however, that due to local bending effects (see Figure 3c), the longitudinal strain gages on the inside surface of the top of the CMP will indicate tensile (positive) strains whereas those on the bottom will indicate compressive (negative) strains.

DCDTs mounted on lightweight rods connected to the inside walls of the CMP near the strain gaged sections were used to measure the changes in vertical (Figure 3d) and horizontal diameters of the pipe. The diameter change rods were slightly offset from the strain-gages to avoid introducing stress concentrations but are still referenced according to the corresponding strain gaged sections. Vertical deflections of the top surface of the CMP were measured by monitoring the movement of steel rods attached to the top of the CMP and extended vertically above the fill at the seven locations shown in Figure 3a. Scales attached to the rods were monitored with a surveying level.

Soil pressure cells were installed in the backfill adjacent to the CMP's horizontal diameter, directly above the CMP, and along the

edges of the prism of soil defined by the CMP diameter. The pressure cells were placed at Sections A, B and C as shown in Figure 4a. Specific placement of the cells at each of the transverse sections is illustrated in Figure 4, b, c, and d.

### TEST PROCEDURES

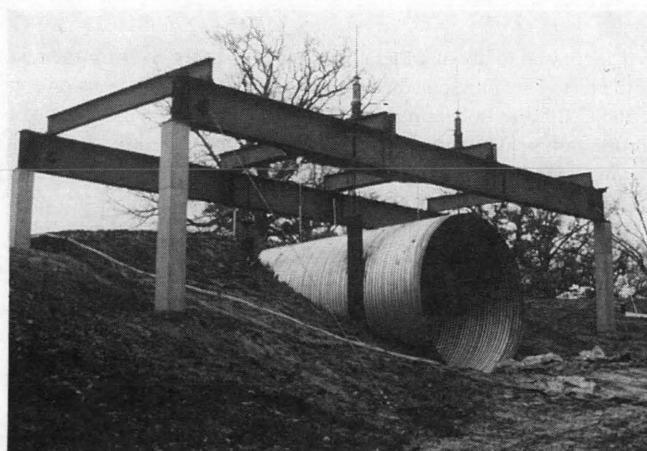
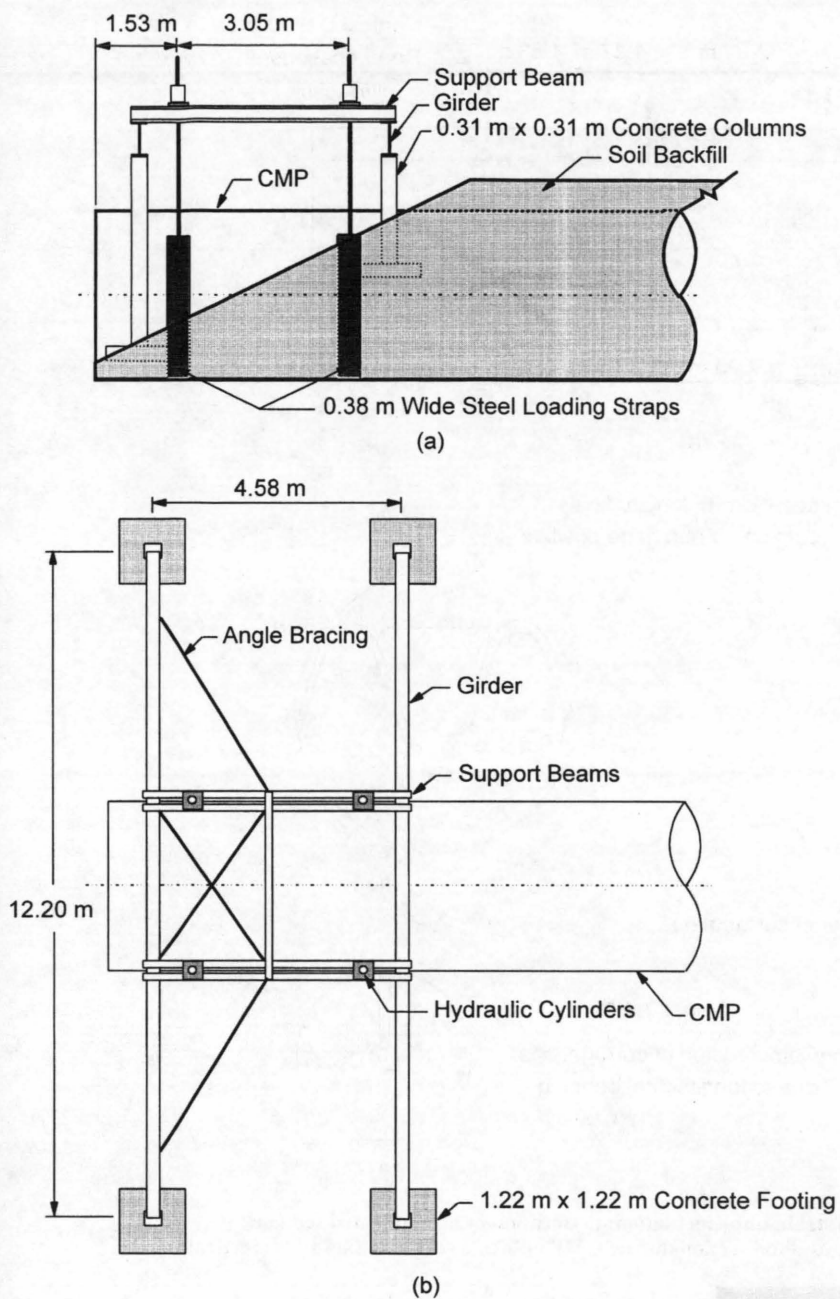
A series of load increments with a load ratio of 2 between the front lift straps and the back lift straps were applied to the CMP to simulate the unbalanced triangular pore pressure distribution. Load cells beneath the hydraulic cylinders were monitored by the DAS for accurate force measurement and test control. After obtaining the desired magnitude of uplift forces for each load increment, the applied loads, soil pressures, strains, and deformations were recorded. This sequence of events was repeated until a 76-mm upward deflection was achieved at the deflection Rod "a" location (see Figure 3a) at the "upstream" end of the CMP; at this point the test was terminated. The loading path is displayed in Figure 5. As indicated in the figure, the desired loading ratio was essentially maintained throughout the entire test.

### TEST RESULTS

Figure 6 presents longitudinal profiles of the CMP at different load increments. Minimal movement (<5 mm) was observed at Section d whereas no activity was observed at Sections e, f, and g (see Figure 3a). The most significant bending in the CMP occurred in the region 4.58 to 9.15 m from the inlet. The steel load straps provided some support at the "upstream" end of the CMP and limited bending in the first 4.58 m of the CMP. Observed changes in the horizontal and vertical diameters of the CMP during uplift were very small, generally less than 5 mm. Thus, the base of the CMP is essentially tracking the movement of the top of the CMP.

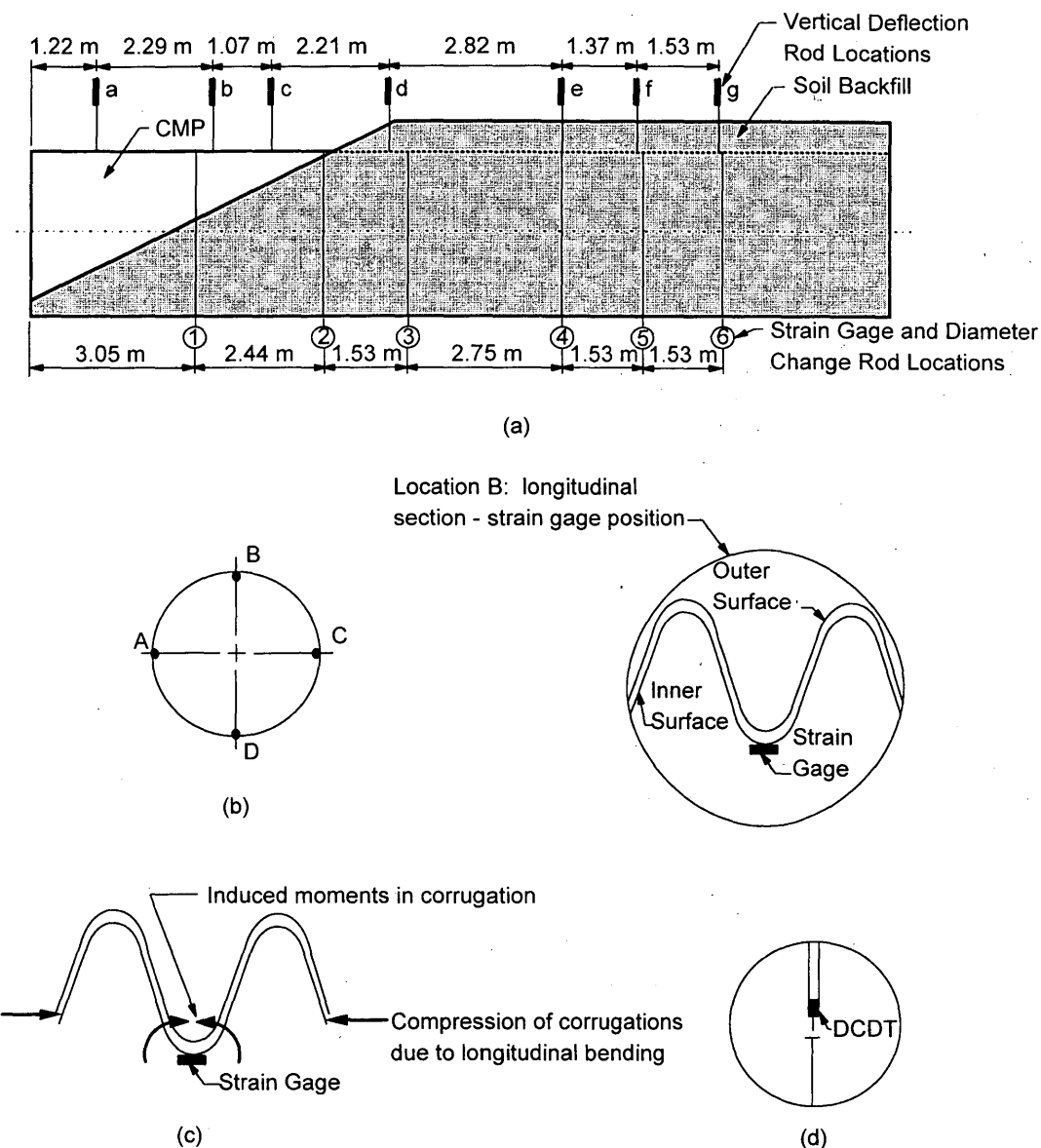
The patterns of longitudinal and hoop strains measured on the inner surface of the CMP were consistent with the deformations of the top of the CMP noted above. As indicated in Figure 7, the longitudinal strains at Section 3 (Figure 7a) were significantly larger than those in the hoop direction (Figure 7b). Additional longitudinal strain data at Sections 1 and 4 are shown in Figure 7, c and d, respectively. The longitudinal strains on the sides of the CMP at midheight (indicated by left and right) were very small relative to those at the top and bottom. This indicates that the CMP's neutral axis remained near the midheight of the pipe.

Top and bottom longitudinal strains at Section 1 (see Figure 7c) were essentially equal in magnitude throughout the entire test. However, at Section 3 where the entire CMP was covered with soil, the strain on the top was less than the corresponding strain on the bottom (see Figure 7a). The base of the CMP at this particular location (Section d in Figure 3a) was beginning to separate from the soil as indicated by the deflection data in Figure 6. This separation accompanied by the expansion of the corrugations on the bottom of the CMP may have prevented the adjacent soil from providing any additional resistance to local deformations. In contrast, the top regions of the CMP were being compressed into the soil. The responses of the soil pressure cells located directly above the top of the CMP confirms this behavior. For example, the earth pressure cell located 0.61 m "downstream" from Section 3 began showing an increase in pressure at the same time as longitudinal strains were beginning to develop on top of the CMP at this location. The pres-



**FIGURE 2** Load frame description: (a) profile of load frame; (b) plan view of load frame; (c) load frame and CMP after backfilling.

(c)



**FIGURE 3** Location of deflection and strain implementation: (a) location of instrumented sections; (b) strain gage locations; (c) location B longitudinal section during CMP uplift; (d) deformation rods (vertical).

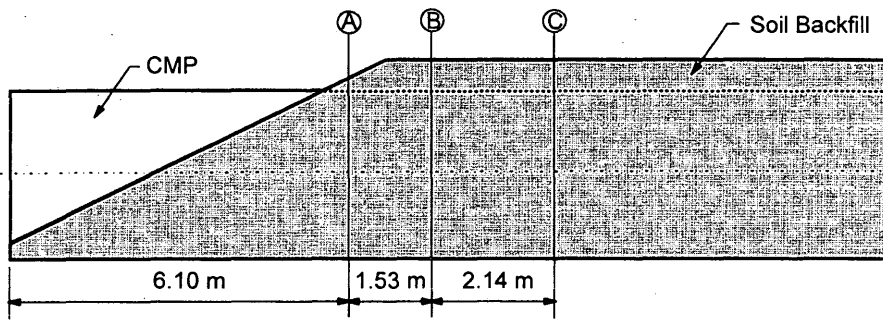
sure increased to about 1 kPa and remained at this level for the rest of the test. The anticipated compression of the corrugations on top of the CMP due to bending (i.e., shortening of the length between corrugation peaks) may have been partially restricted by the interlocking action of soil in the corrugations. This suggests that the top portion of the CMP may have gained additional stiffness from the interacting soil, whereas the bottom portion of the CMP received little benefit from the soil.

### ANALYSIS OF CMP UPLIFT

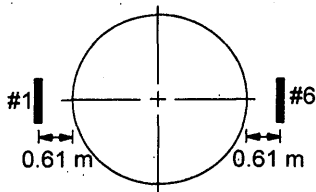
A CMP-soil system subjected to inlet uplift forces and longitudinal bending is a three dimensional, statically indeterminate, soil-structure interaction problem. As such, the problem should be modeled and analyzed utilizing three dimensional numerical techniques with structural, interface, and soil model element capabilities. In

geotechnical engineering, however, analytical simplifications are quite often made in the design stage without compromising the performance and reliability of the structure. Thus, several levels of analysis may be considered, each with its own assumptions and potential limitations. The following three levels of analysis could be used in this CMP-soil interaction problem.

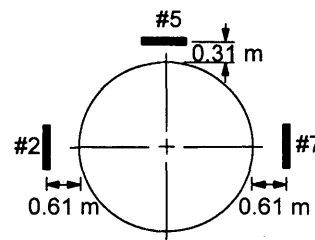
1. Given the deformation patterns observed during the test (Figure 6), it is not unreasonable to consider the CMP as a cantilever beam, rigidly restrained at the "downstream" end by the soil cover. In this case, a cantilever beam analytical approach could be used in which the deflected shape of the CMP is analyzed and the longitudinal rigidity of the CMP-soil system is lumped into the CMP beam. The soil, therefore, is modeled simply as a distributed load on the top of the CMP beam. This approach, however, is very site (CMP and soil backfill) specific.



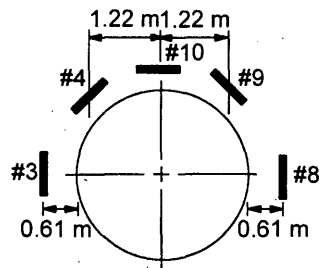
(a)



(b)



(c)



(d)

FIGURE 4 Soil pressure cell locations: (a) soil pressure cells; (b) Section A; (c) Section B; (d) Section C.

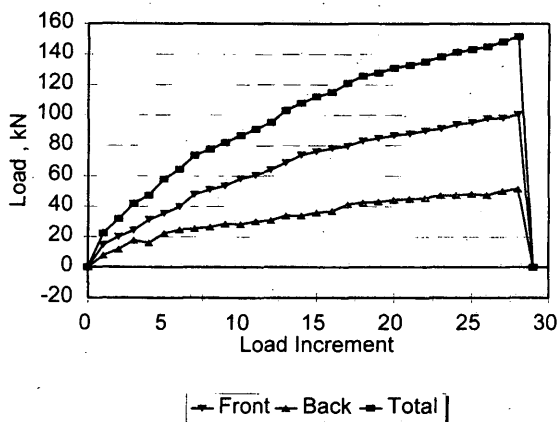


FIGURE 5 CMP uplift loading sequence.

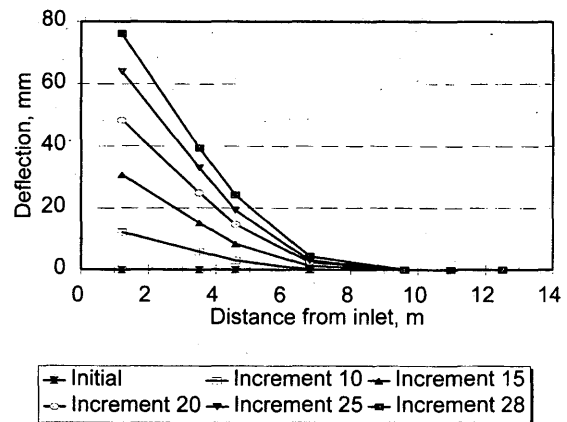


FIGURE 6 Profile of deflected CMP at selected load increments during uplift.



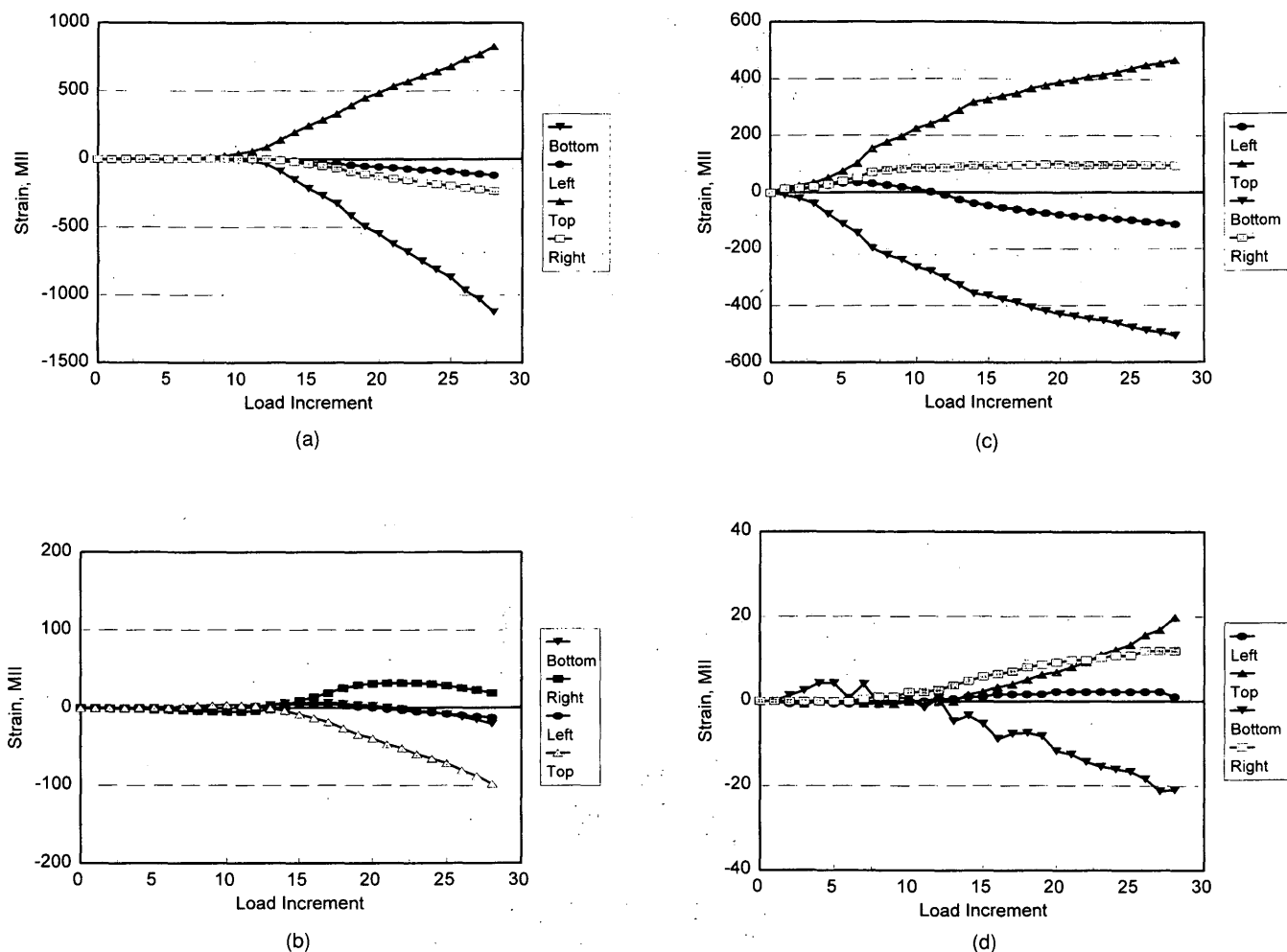


FIGURE 7 CMP uplift strain data: (a) longitudinal strains versus load increment (Section 3); (b) hoop strains versus load increment (Section 3); (c) longitudinal strains versus load increment (Section 1); (d) longitudinal strains versus load increment (Section 4).

2. The three dimensional problem could be reduced to two interlinked two dimensional problems; that is, a plane strain analysis of the CMP-soil hoop (transverse) section loosely coupled with a longitudinal bending analysis, as described above. In this case, the transverse section analysis would form the basis for the soil-structure interaction modeling whereas the out-of-plane longitudinal responses would be incorporated through a superposition scheme.

3. Full three dimensional numerical analysis, in which the longitudinal bending of the CMP is fully coupled with the interaction of the surrounding soil would represent the most comprehensive analysis. In this case, CMP's of various sizes, gages and corrugation geometries embedded in backfill with a variety of strength and deformation properties could be analyzed without the need for lumping and superposition of responses.

Considering the experiments and analysis carried out by the ISU researchers on the longitudinal stiffness and strength of large diameter CMP sections without soil (1), it is useful and instructive at this point to analyze the CMP-soil system as a soil-stiffened cantilever beam (i.e., Level 1 previously described) so that a direct comparison of results can be made. For the reader's information, the other

two levels of analysis (Levels 2 and 3), along with further full-scale field testing, are in progress at ISU.

In the Level 1 analytical approach, the deflected shape of the CMP is defined mathematically by fitting a fifth order polynomial to the deflection data points for a particular load increment; slopes of the deflected shape of the CMP can then be approximated by taking the first derivative of the deflection function. A plot of the CMP's calculated deflected shape versus that actually measured, for the last load increment, is shown in Figure 8. This figure shows that the deflection function precisely fits the deflected shape of the CMP and that the vertical deflection and slope become essentially zero at a distance of 9.15 m from the inlet; this effectively defines the boundaries of the cantilever beam substructure. The CMP-soil substructure can now be analyzed using customary structural analysis techniques. Two unknowns are introduced into the analysis: 1) the soil response along the CMP during uplift, which is assumed to be a distributed load related to the weight of the soil above the midheight of the CMP and 2) the overall flexural stiffness of the CMP, in terms of  $EI$ , including the rigidity added by the soil backfill. In this analysis,  $E$  represents the CMP material modulus of elasticity which is assumed to be 200,000 MPa whereas  $I$  represents the moment of inertia.

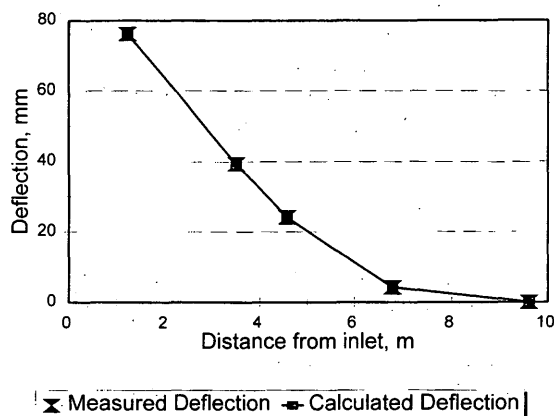


FIGURE 8 Deflected shape of CMP (calculated and measured deflections for load Increment 28).

The next step is to formulate a moment diagram using the external loads shown in Figure 9. The CMP weight ( $W_p$ ), the concentrated uplift forces, and the unknown soil response were the loads used to construct the moment diagram. For this analysis, as described above, the soil response (defined as  $W_s$ ) is simply considered to be a distributed load related to the shape of the backfill cover overlying the midheight of the CMP.

Relationships between the two unknown parameters, the moment of inertia ( $I$ ) and the soil response ( $W_s$ ) were developed using the moment-area theorems for elastically loaded beams undergoing small deflections. The angle change and tangential deviation between the various points were computed using the mathematically determined deflection curve previously described. The unknown moment of inertia ( $I$ ) and unknown soil response ( $W_s$ ) were then determined using numerical integration techniques. The analysis yielded  $I = 0.0003900 \text{ m}^4$  and  $W_s = 86.48 \text{ kN/m}$ . The weight of the soil above the midheight of the CMP in the region with 0.61 m of cover is 49.29 kN/m. The difference between  $W_s$  and the actual soil weight can be attributed to the three dimensional soil strength and stiffness effects which are not accounted for in this analysis.

After determining experimental values for  $I$  and  $W_s$ , two methods were used to construct moment diagrams resulting from the applied loading. One moment diagram was computed using the external load data shown in Figure 9 assuming cantilever conditions whereas the other was computed using the differential equation of the elastic curvature (i.e.,  $y'' = M/EI$  where  $y''$  is an approximation of cur-

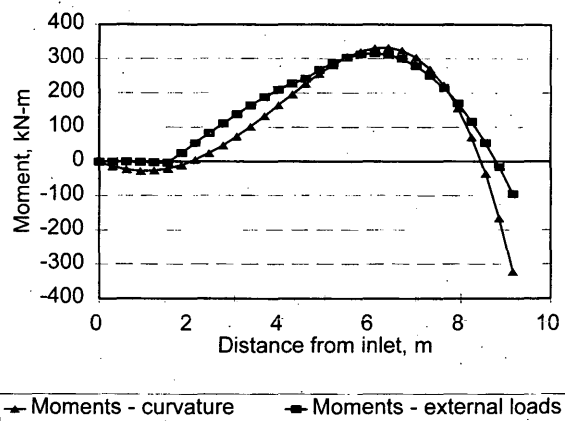


FIGURE 10 Comparison of moment diagrams resulting from the CMP applied loading.

vature). As illustrated in Figure 10, the two methods give similar moment diagrams. The moments calculated from the external loading are very small in the first 1.43 m of the CMP due to the small self-weight of the CMP.

As previously noted, one of the key reasons for pursuing this simple cantilever beam analytical approach was to compare the longitudinal stiffness of the backfilled CMP with the longitudinal stiffness of the CMP without soil backfill. As noted above, the back analysis of the backfilled CMP uplift test gave an  $I$  value of  $0.0003900 \text{ m}^4$  whereas an  $I$  value of  $0.0002668 \text{ m}^4$  was determined for the CMP without soil backfill using the ISU analytical model (4) and the assumed material modulus of elasticity  $E$  noted previously. Thus, the CMP-soil interaction effects apparently result in a 46 percent increase in the effective  $I$  of the CMP. These CMP-soil interaction effects include the restriction of compression of the corrugations on the upper surface of the CMP and the resistance due to the stiffness and shear strength of the surrounding soil.

As an additional assessment of the effects of the soil-structure interaction on the CMP's overall longitudinal stiffness, a CMP effective longitudinal moment of inertia,  $I$ , was calculated using the ISU analytical model [Equation 10 of Havens et al. (4)]. The model requires calculation of the hoop to longitudinal stress ratio factor [Equation 1 of Havens et al. (4)] using hoop and longitudinal strain gage data collected from the CMP wall during bending. The remainder of the terms in Equation 10 are geometric parameters that are fixed for the given CMP. Strain gage data collected from Section 3

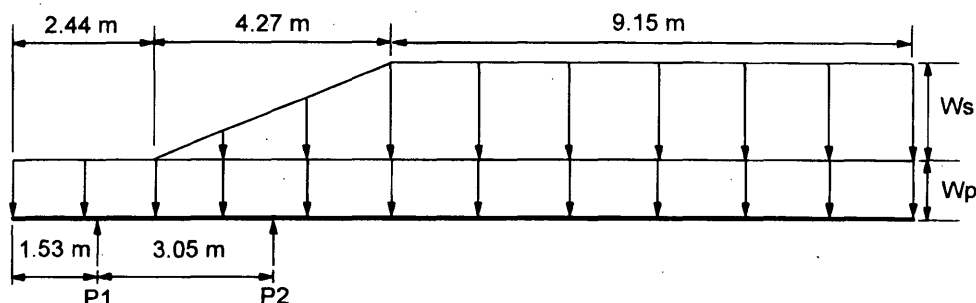


FIGURE 9 CMP cantilever beam analysis (external loads).



of this CMP-soil backfill field test gave a hoop to longitudinal strain ratio of 0.12 which in turn yielded a stress ratio factor of 0.19; substitution of this value of stress ratio factor into the ISU analytical model gives an effective  $I$  value of 0.0004549  $\text{m}^4$ . This analysis, using the field test strain gage data and the ISU analytical model alone, gives an effective  $I$  value which agrees to within 17 percent of the  $I$  value back calculated from the deflected shape of the CMP. Thus, two independent methods of calculating  $I$  give comparable results and indicate the effects of soil-structure interaction in increasing the effective longitudinal stiffness of the CMP. These analyses indicate that for these field test conditions the buried culvert is essentially behaving as a cantilever beam. Moreover, the ISU CMP longitudinal bending model (4), developed assuming no soil backfill, may be used in the analysis of a CMP under inlet uplift conditions.

## CONCLUSIONS

The data from this full-scale field test indicate that the backfilled CMP subjected to inlet uplift forces behaves essentially as a cantilever beam, with the greatest bending occurring near the crest of the soil embankment slope. The strain gage data, which shows smaller strains on the top of the CMP than the bottom, indicates that the top region of the CMP is stiffened due to the interacting soil whereas the bottom of the CMP, which separates from the soil, does not receive this stiffening effect. The strain gage data also indicate that the neutral axis of the CMP remains near its midheight. When analyzed as a cantilever beam, the moment of inertia,  $I$ , calculated from the deflected shape of the CMP agrees very well with the  $I$  independently calculated using the ISU CMP longitudinal bending model in conjunction with the strain data from this test.

These results represent a significant advancement in the understanding of the behavior and in the analysis of the longitudinal bending of buried CMP under inlet uplift conditions, which will be useful for the formulation of rational tie-down methodologies for the inlets of CMP culverts. It must be stressed, however, that the analysis described in this report is very dependent on the particular CMP and condition of the soil backfill (e.g., unit weight, shear strength, and stiffness) used in this field test. The design of field

CMP inlet tie-downs will depend on the CMP diameter, corrugation profile, CMP gage, CMP material properties, backfill material properties and placement methodologies. For this reason, two- and three-dimensional soil-structure interaction numerical modeling, along with further full-scale field testing, is in progress at ISU.

## ACKNOWLEDGMENTS

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