Load Reduction on Rigid Culverts Beneath High Fills: Long-Term Behavior

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Three full-scale tests with the imperfect ditch method are described. The imperfect ditch method involves installing a compressible inclusion above rigid culverts to reduce the vertical earth pressure. Superlight expanded polystyrene blocks are used as the compressible material. In the first test, the instrumented culvert was a 1.95-m diameter concrete pipe beneath a 14-m-high rock-fill embankment. In the second test, a 1.71-m diameter concrete pipe was used beneath a 15-m-high rock fill, and in the third, the culvert is a cast-in-place concrete box culvert with a 2.0-m width beneath 11 m of silty clay. The culverts were built between 1988 and 1989, and the instrumentation measured earth pressure, deformation, and temperature. The full-scale measurements show considerable reduction in the vertical earth pressure: that on top of the pipes in the granular fill was reduced to less than 30 percent of the overburden and that on the box culvert beneath the clay fill was reduced to less than 50 percent of the overburden. The deformation of the expanded polystyrene was 27 percent in the rock fill and 42 percent in the clay. The long-term observations show that there is no increase in earth pressure on and deformation of the pipes beneath the rock fill. There is a slight increase in deformation of the expanded polystyrene in the clay. Use of this method in Norway has realized cost reductions of the order of 30 percent and has made it possible to use concrete pipes beneath higher fills.

The problem of earth pressure on buried structures is of great practical importance in constructing highway embankments above pipes and culverts. Both the magnitude and the distribution of earth pressure on buried culverts are known to depend on the relative stiffness of the culvert and the soil. Current design methods distinguish between a rigid culvert and a flexible culvert. The vertical earth pressure on a rigid culvert is greater than the weight of the soil above the structure (negative arching). The vertical earth pressure on a flexible culvert is less than the weight of the soil above the culvert (positive arching).

Spangler (*1*) stated that in some of Marston's early experiments, the loads on rigid embankment culverts were 90 to 95 percent greater than the weight of the soil directly above the structure. An attempt to avoid this increase in load on the structure led to the development of the imperfect ditch method (sometimes called the induced trench method) of construction.

Penman et al. (*2*) measured the earth pressure on a rigid reinforced concrete culvert below 53 m of rockfill. The measured vertical earth pressure on the culvert crown was about two times the overburden. Höeg (*3*) performed model tests on a rigid pipe and reported that the crown pressure was about 1.5 times the applied surcharge.

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The most common type of compressible material used in the imperfect ditch method is organic (baled straw, leaves, hay, compressible soil). Old tires are used in France (*4*).

The traditional imperfect ditch method involves installing a compressible layer above the culvert within the backfill. As the embankment is constructed, the soft zone compresses more than the surrounding fill and thus induces positive arching above the culvert. The construction procedure is shown in Figure 1.

CHOICE OF COMPRESSIVE MATERIAL

Although the artificially induced arching action is desirable, experience has shown that considerable care should be exercised in designing the compressible layer if the desired loading is to be realized. Indeed, some arrangements of compressible layers have led to serious stress concentration problems.

In addition, the compressible materials are often organic in nature, and they manifest a number of disadvantages, such as the difficulty of specifying their compressibility characteristics and the possibility of their decomposition.

Structural distress has been observed in some imperfect ditch full-scale tests because of the decomposition of the organic material with time as described by Rude (*5*) and Krizek et al. (*6*). This can lead to an unsatisfactory load distribution on the culvert. The reason for the observed structural distress in some imperfect ditch installations could also be attributed to the use of low-quality cohesive material at the sides of the culvert.

From the definition of the mechanisms of positive arching by Terzaghi (*7*), it is clear that positive arching involves two phases: a reduction of the earth pressure on a yielding part of the structure and an increase of the earth pressure on the adjacent nonyielding areas. Also Bjerrum et al. (*8*) noted that the increase in pressure on the adjacent nonyielding areas is equal to or larger than the reduction in pressure on the yielding part.

A zone of maximum shear stress will develop at the sides of the culvert due to the arching phenomena. The soil in the zone of higher shear stress may yield with time if it is a low-quality cohesive soil. Therefore it is important to use high-quality well-compacted granular soil at the sides of an imperfect ditch culvert. If the backfilling material is stable with time, equilibrium will be reached and the active arching will be a permanent state of stress.

Katona et al. (*9*) pointed out that very little quantifiable data are available about the stress-strain properties of the soft

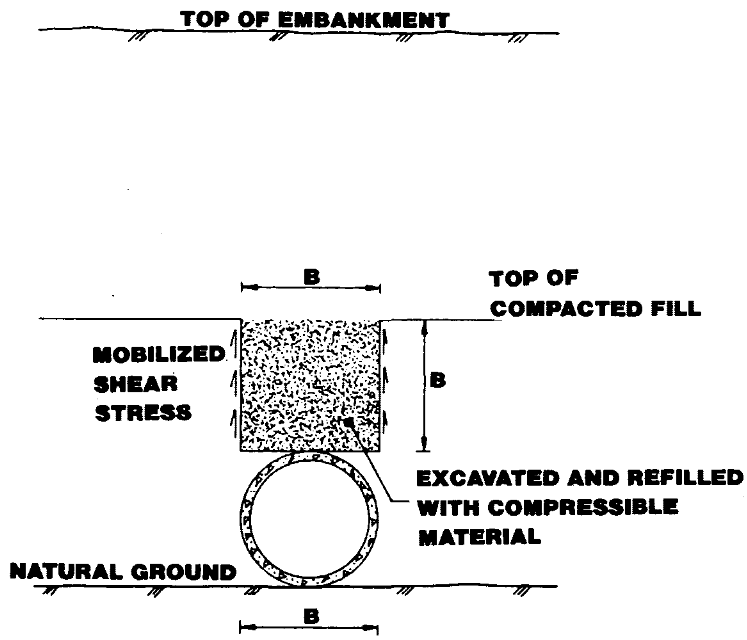

FIGURE 1 Imperfect ditch culvert—traditional installation.

organic materials used to promote arching in imperfect ditch installations. The long-term stability of the organic material was also questioned. A survey was conducted on potential soft materials for which the stress-strain properties are available. Expanded polystyrene (EPS) was identified as a possible material for culvert installations.

Hoff (*10*) conducted tests on a wide range of possible materials that could be used to promote arching. He concluded that foamed or cellular materials (like EPS) exhibited the desirable elastic-plastic behavior. The ideal stress-strain curve for an elastic-plastic material was shown, and the curve was very similar to the one shown by Katona et al. (*9*).

Expanded polystyrene is also used to reduce lateral earth pressure on rigid walls (*11*). The function of the expanded polystyrene is to allow controlled horizontal deformation of the retained soil and induce soil yielding. This concept is analyzed with numerical modeling (*12,13*).

Expanded polystyrene is used as compressible material in imperfect ditch installations in Norway. The typical stress-strain curve for EPS is very similar to the ideal stress-strain curve indicated by Katona et al. (*9*) and Hoff (*10*). A typical stress-strain curve for EPS with a density of 20 kg/m^3 is shown in Figure 2.

Trade qualities of EPS are superlight (20 kg/m^3), and standard block dimensions are $0.5 \times 1.0 \times 2.0$ m with a volume of 1 m^3 and a weight of 20 kg. The compressive strength can be determined using an unconfined compression apparatus. The test specimens are $50 \times 50 \times 50$ mm, and an average strength of minimum 100 kN/m^2 at 5 percent compression (2.5 mm) is required according to Norwegian specifications.

The blocks are easy to handle and are widely used in Norway for solving soft ground problems, Frydenlund (*14*). Overall long-term performance is good, as reported by Aabøe (*15*). The moisture pickup is less than 1 percent of a volume basis for drained conditions. Precautions must be taken against solvents, such as petrol, that can dissolve EPS.

The long-term serviceability of imperfect ditch installations is an important concern. Only full-scale tests and field observations can provide the information required to evaluate the time effects of an imperfect ditch installation.

INSTRUMENTED FIELD INSTALLATIONS

Field Installation at Eidanger

The structure was situated below a 14-m-high rock-fill embankment and served as a drainage pipe for new Euroroad 18 in Telemark County, Eidanger. The concrete pipe had an inner diameter of 1.6 m, a concrete thickness 0.176 m, and an outer diameter of 1.952 m. The pipe was reinforced with two layers of reinforcing steel. The steel bars were 12 mm in diameter and were laid with a center-to-center distance of 51 mm.

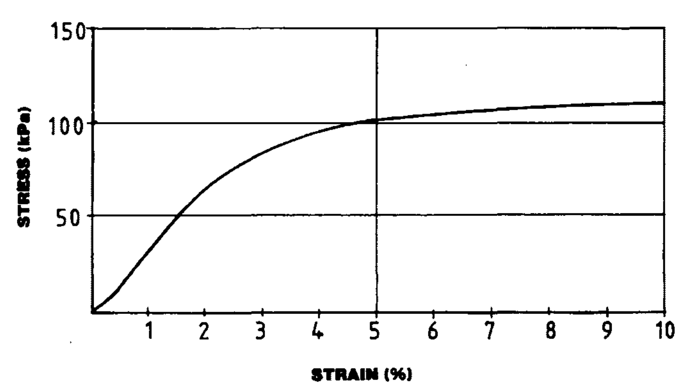

FIGURE 2 Typical stress-strain curve for EPS with density of 20 kg/m^3.

The yield strength of the steel was 500 MPa. The compressive strength of the concrete was 50 MPa. The concrete pipe was made of sections 1.5 m in length, with a total length of 70 m. A cross section of the installation with instrumentation is shown in Figure 3.

Standard EPS blocks 0.5 × 1.0 × 2.0 m were used. Six test specimens of EPS were tested in the laboratory and showed an average compression strength of 98.3 kPa. The measured density was 20.3 kg/m³. The EPS blocks were placed when the backfill had reached 0.5 m above the top of the pipe. Laying the blocks was very fast and simple, and there was no need to excavate a ditch above the pipe.

The in situ soil was excavated to about 0.5 m below the invert elevation, down to bedrock, and replaced by 0 to 16 mm of sandy gravel. The same material (0 to 16 mm of sandy gravel) was used for backfill, with an optimum dry density of 21.5 kN/m³ and an optimum moisture content of 9.3 percent. A minimum of a 97 percent Standard Proctor was required. Nuclear field density tests showed an average of 100 percent Standard Proctor (15 tests). The backfill was compacted in 20-cm-thick layers.

The backfill extended 1 m from the springline and 0.5 m over the top of the pipe. The remaining fill in the embankment was rock fill that was placed in 3-m-thick layers and compacted with 6 to 8 passes of a 6-ton vibratory roller. From field experience, this equals 95 to 97 percent Standard Proctor. The construction began in August 1988 and was completed in June 1989.

To evaluate the performance of the pipe and the EPS during construction and on a long-term basis, the pipe and the surrounding backfill were instrumented with hydraulic earth pressure cells of the Glötzl type. In addition, settlement tubes were installed on top of the pipe and on top of the EPS to measure the vertical deformation. The Glötzl cells were 20 × 30 cm; four cells were used. The location of the cells is shown in Figure 3.

One cell was placed in the springline to measure the horizontal earth pressure on the pipe (Cell 1). Two cells were placed 20 cm over the top of the pipe to measure the vertical earth pressure, one cell in the centerline (Cell 2) and one cell over the springline (Cell 3). The last cell (Cell 4) was placed in the centerline 2 m over the top of the pipe to measure the vertical earth pressure in the embankment. The temperature in the soil at the cell locations was measured with thermistors, and temperature corrections were made. Earth pressure and deformation measurements during construction are described by Vaslestad (*16*).

The measured earth pressure on Cell 2 at the top of the pipe is shown in Figure 4. The earth pressure on Cell 2 increased to 72 kPa in September 1988 when the fill height was

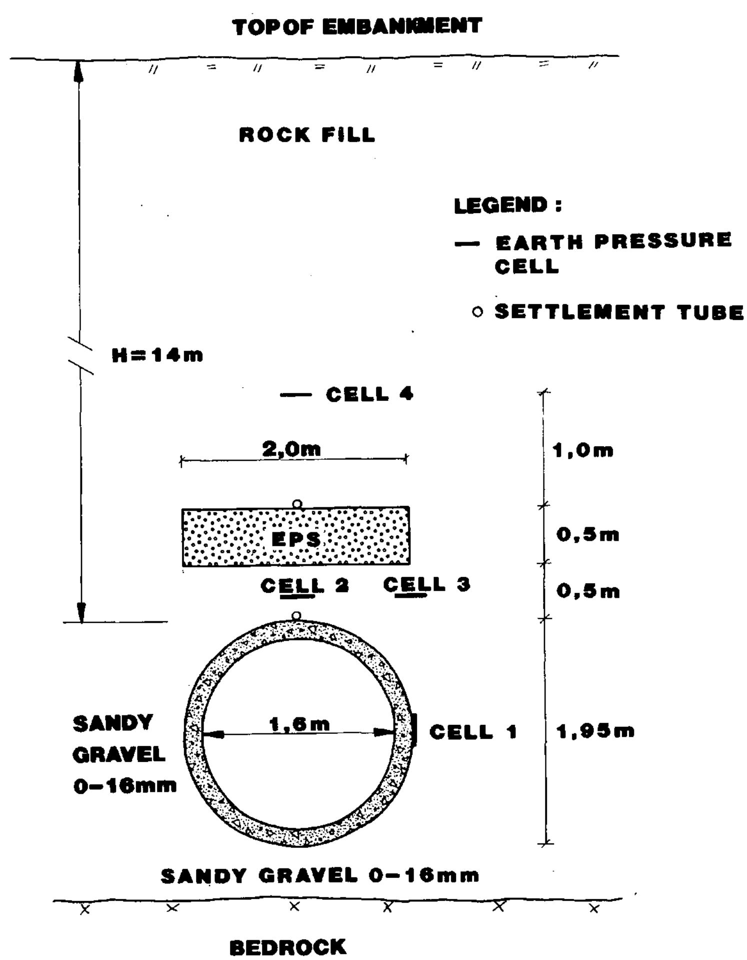

FIGURE 3 Geometry of instrumented cross section (Eidanger).

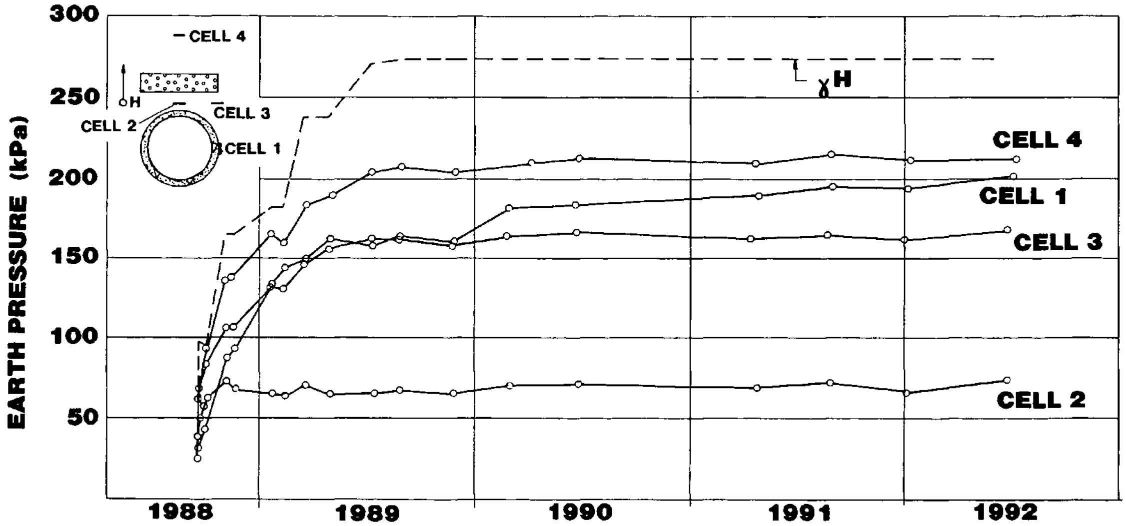

FIGURE 4 Measured earth pressure cells 1, 2, 3, and 4 (Eidanger).

8.3 m. A further increase in the fill height to 13.7 m did not increase the earth pressure on Cell 2. From the end of construction in July 1989 until June 1992, the earth pressure on the top of the pipe was relatively constant around 65 to 75 kPa, which is 25 to 27 percent of the overburden.

The earth pressure on Cell 3 at the end of construction was 161 kPa (see Figure 4). The earth pressure varied between 161 and 167 kPa over the next 3 years. This was 61 to 63 percent of the overburden. Only half of this cell was covered with EPS, explaining why the earth pressure was larger than that on Cell 2.

The measured horizontal earth pressure on the pipe (Cell 1) springline is shown in Figure 4. At the end of construction it was 164 kPa and increased slowly. After almost 3 years, the horizontal earth pressure was 201 kPa, 62 percent of the overburden.

The horizontal earth pressure was 2.7 times the measured vertical earth pressure on top of the pipe. This is not desirable and shows that it is necessary to increase the width of the EPS to decrease the horizontal earth pressure to a value more equal to the vertical earth pressure. In spite of the relatively large horizontal earth pressure, the pipe showed no sign of distress.

The earth pressure on Cell 4 above the EPS is shown in Figure 4. It was 204 kPa at the end of construction and slowly increased until June 1992 to 212 kPa, which was 90 percent of the overburden.

The measured vertical compression of the EPS is shown in Figure 5. The vertical deformation was 13.6 m at the end of construction, which was 27 percent of the initial EPS thickness. The compression was relatively constant in the measured 3-year period, and there was no increase in compression in this period.

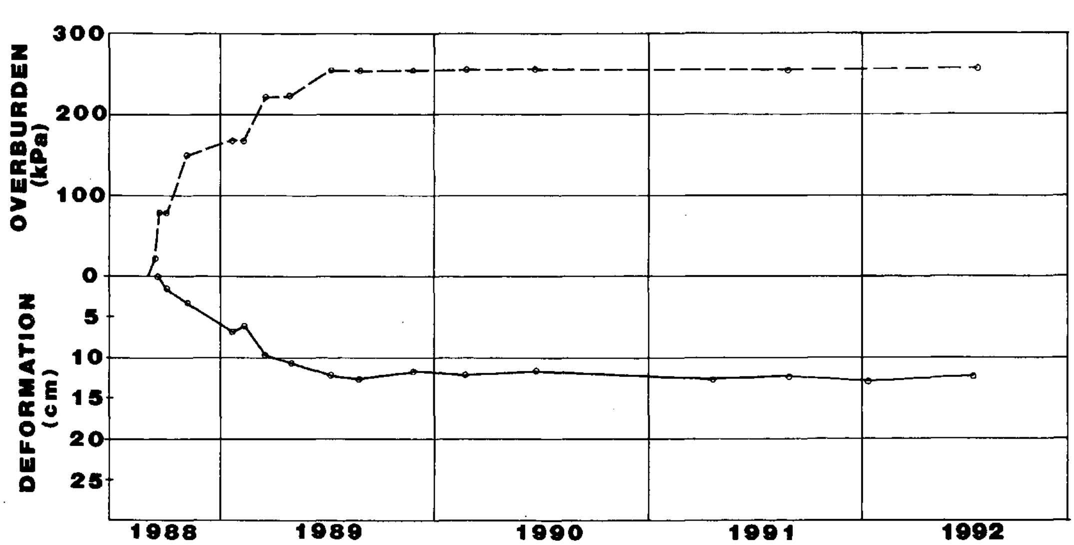

FIGURE 5 Measured deformation of EPS (Eidanger).

Field Installation at Sveio

The culvert was a concrete pipe with an inner diameter of 1.4 m and an outer diameter of 1.714 m. The embankment above the pipe consisted of 15 m of rock fill. The total length of the culvert was 75 m. A cross section of the pipe is shown in Figure 6.

Based on the experience from the field installation at Eidanger and parameter studies with the CANDE program (*17*), the width of the compressible layer increased to at least 1.5 times that of the outer diameter of the pipe (*18*). Increasing the width of the EPS to 1.5 times the width of the pipe had a positive effect on the structural response of the pipe. This was also in accordance with the findings of Sladen and Oswell (*19*) and Leonhardt (*20*). This decreased the horizontal earth pressure on the pipe. The ideal situation is for the horizontal earth pressure to equal the vertical earth pressure. For practical reasons, the width of EPS was 3.0 m, and blocks with a thickness of 0.5 m were used. The EPS had a compression strength of 100 kPa and a density of 20 kg/m³.

The in situ soil consisted of dense moraine. Well-graded sandy gravel (GW) at a thickness of 400 mm was used as the bedding. As proposed by Heger (*21*), the bedding was compacted to a minimum of 95 percent Standard Proctor and loosened below the invert over a width of one-third of the pipe's outside diameter. This reduced the moments and shears at the invert and increased the structural capacity of the pipe.

Well-graded sandy gravel (0–16 mm) was also used for backfill. The backfill was compacted in 20-cm-thick layers, and nuclear field density tests showed an average of 98.5 percent Standard Proctor compaction. The backfill extended 1 m from the springline and 0.5 m above the EPS. The remaining fill in the embankment was rock. The average unit weight of the soil was $\gamma = 20$ kN/m³. The construction began in November 1989, and in February 1990 the embankment had reached 13.7 m above the pipe. The remaining fill was placed at the beginning of 1992.

The pipe and the backfill were instrumented with five hydraulic earth pressure cells of the Glötzl type. Two settlement tubes were installed on top of the EPS to measure the vertical deformation. The instrumentation is shown in Figure 6.

The measured earth pressure on Cells 1, 2, and 3 is shown in Figure 7. Unfortunately, no earth pressure readings were taken before the fill had reached 12 m above the top of the pipe.

The measured earth pressure on Cell 1 at the top of the pipe was 57 kPa when the fill had reached 13.7 m above the

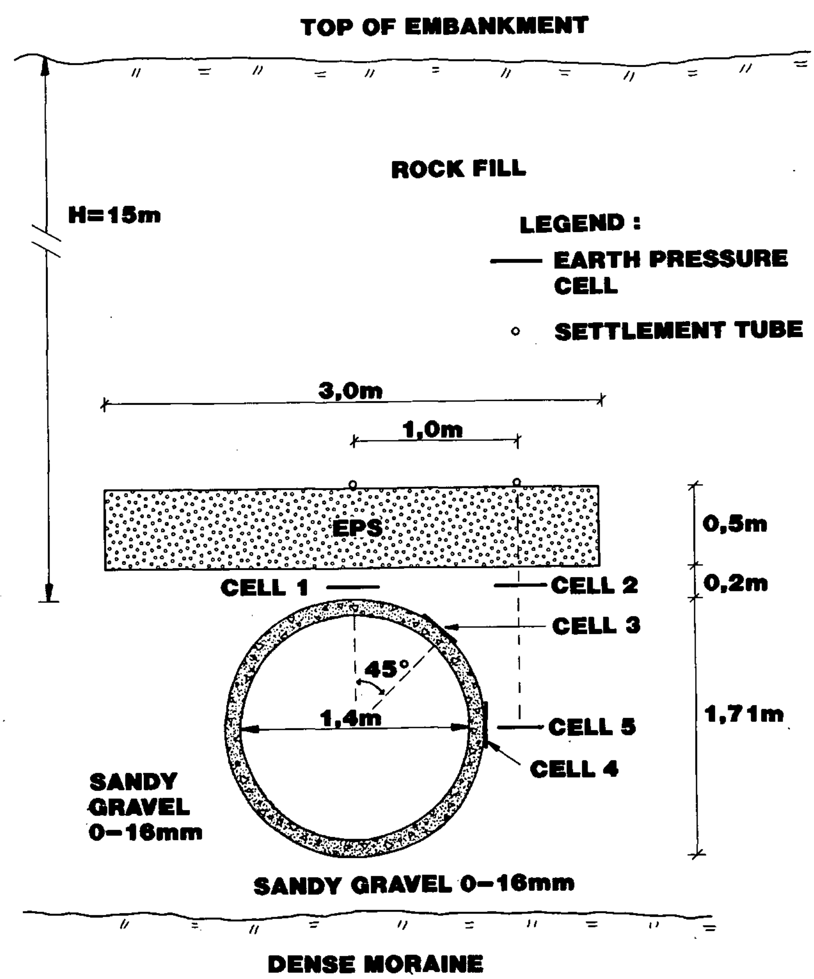

FIGURE 6 Geometry of instrumented cross section (Sveio).

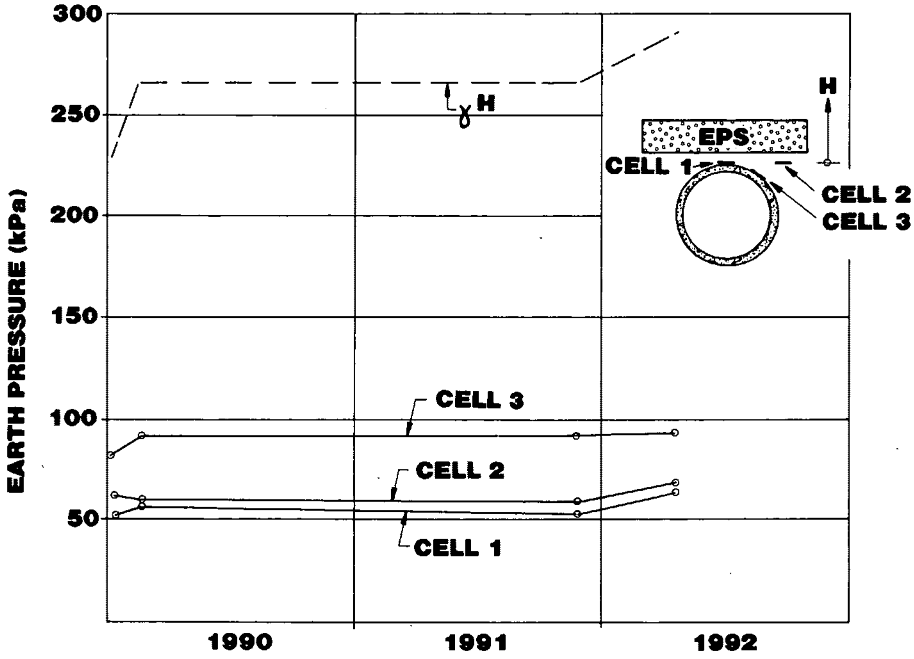

FIGURE 7 Measured earth pressure cells 1, 2, and 3 (Sveio).

pipe. This was 22 percent of the overburden. This value showed a small decrease from February 1990 to November 1991.

The earth pressure on Cell 2 was slightly larger, and the earth pressure on Cell 3 was 97 kPa during this period. The average earth pressure on these cells was 29 percent of the overburden.

The horizontal and vertical earth pressure on the pipe's springline (Cells 4 and 5) is shown in Figure 8. The horizontal earth pressure decreased from 52 to 41 kPa from February 1990 to November 1991.

The vertical earth pressure at the same level (Cell 5) was between 139 and 152 kPa during this period. The average measured horizontal earth pressure coefficient was $K = \sigma_H/\sigma_V = 0.32$ at the pipe's springline.

Compared with the measured vertical earth pressure at the top of the pipe, the average horizontal earth pressure was 89

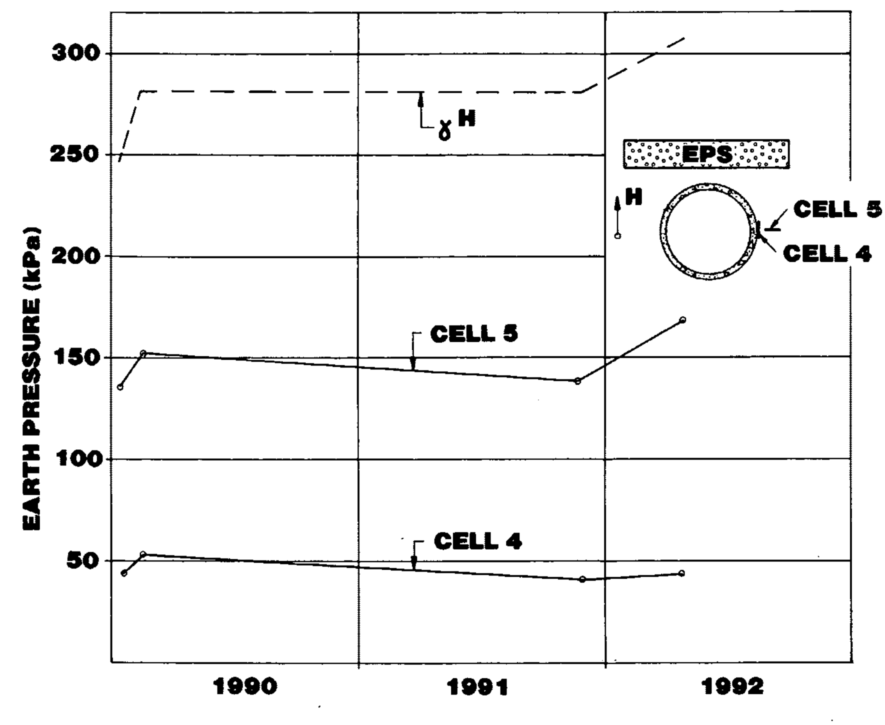

FIGURE 8 Measured earth pressure cells 4 and 5 (Sveio).

percent. This implies that the moments in the pipe were very small. This shows that increasing the width of the EPS reduced the horizontal earth pressure.

The measured vertical compression of the EPS is shown in Figure 9. The deformation was 6.5 m at a full height of 12 m and increased to 8.8 cm at a fill height of 13.7 m. From February 1990 to November 1997 the deformation increased slightly from 8.8 to 9.2 cm. When the remaining fill up to 15.0 m was placed, the deformation increased to 13.1 cm. This was 26 percent of the initial thickness of 50 cm of the EPS.

Field Installation at Hallumsdalen

The culvert was a cast-in-place box culvert with a width of 2.0 m and a height of 2.55 m. The culvert was continuous, had a total length of 385 m, and crossed a valley beneath an embankment of compacted dry crust clay up to 23 m in height.

The subsoil consisted of overconsolidated silty clay with a water content of 25 to 30 percent and undrained shear strength of 35 to 70 kPa. The culvert was instrumented with strain gauges to measure the tension in the culvert due to horizontal shear strain and settlement (*22*).

To investigate the time effects on the earth pressure in a cohesive fill using the imperfect ditch method, EPS was placed above the culvert at a length of 20 m. This section of the culvert was situated in the counterfill that was built up with silty clay. The unit weight of the silty clay was $\gamma = 20$ kN/m^3. EPS with a thickness of 0.5 m and a width of 2.0 m was placed above the culvert as shown in Figure 10.

The section was instrumented with two hydraulic earth pressure cells of the Glötzl type. The deformation of the EPS was measured using a settlement plate. To compare the earth pressure on the imperfect ditch section with a conventional section, one earth pressure cell was placed above the culvert in a cross section without EPS (see Figure 10).

The construction of the embankment began in July 1989. The measured earth pressure on Cell 1 at the top of the culvert is shown in Figure 11. At completion of the fill in February 1990, the fill height was 10.8 m above the cell level, and the overburden was 206 kPa. The measured earth pressure was 132 kPa at this fill height, which was 63 percent of the overburden. The earth pressure decreased slightly to 123 kPa in April 1991. There was a further decrease to 88 kPa in December 1991. This was probably due to stability problems and movements in the counterfills that occurred in April 1991.

The measured earth pressure on Cell 2, which was located in the fill 1 m above the EPS, is shown in Figure 12. At completion of the fill in February 1990, the earth pressure was 144 kPa, which was 81 percent of the overburden. The earth pressure decreased to 128 kPa in July 1992.

The earth pressure on Cell 3, which was located on top of the culvert in a section without EPS, is shown in Figure 13. The fill height above the culvert was 9.8 m. At completion of the fill in February 1990, the measured earth pressure was 244 kPa. The overburden was 196 kPa, and the measured earth pressure was 1.24 times the overburden. Based on extensive finite element modeling, Tadros et al. (*23*) proposed an expression for calculating the earth pressure on concrete box culverts. For silty clay soil, this expression gives an earth pressure of 1.17 times the overburden on top of the culvert.

The measured deformation of the EPS is shown in Figure 14. The deformation was 6 cm when the overburden was 100 kPa, which corresponded to the compression strength of the EPS. The deformation increased to 18.5 cm at completion of fill, when the overburden was 196 kPa. From February 1990 to May 1990, the deformation increased to 19.8 cm. For the next 2 years, the deformation slightly increased to 20.9 cm,

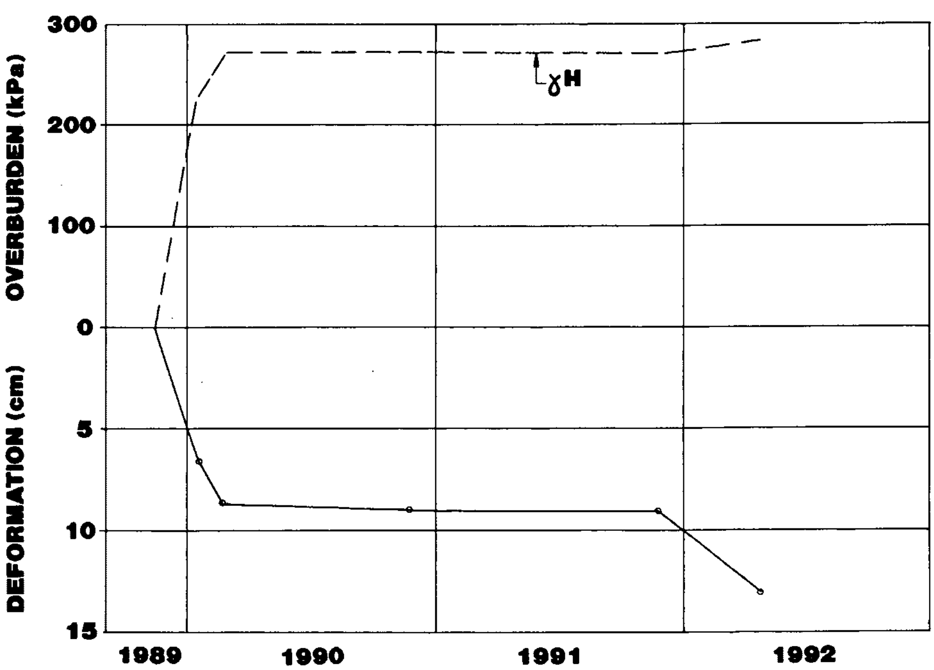

FIGURE 9 Measured deformation of EPS (Sveio).

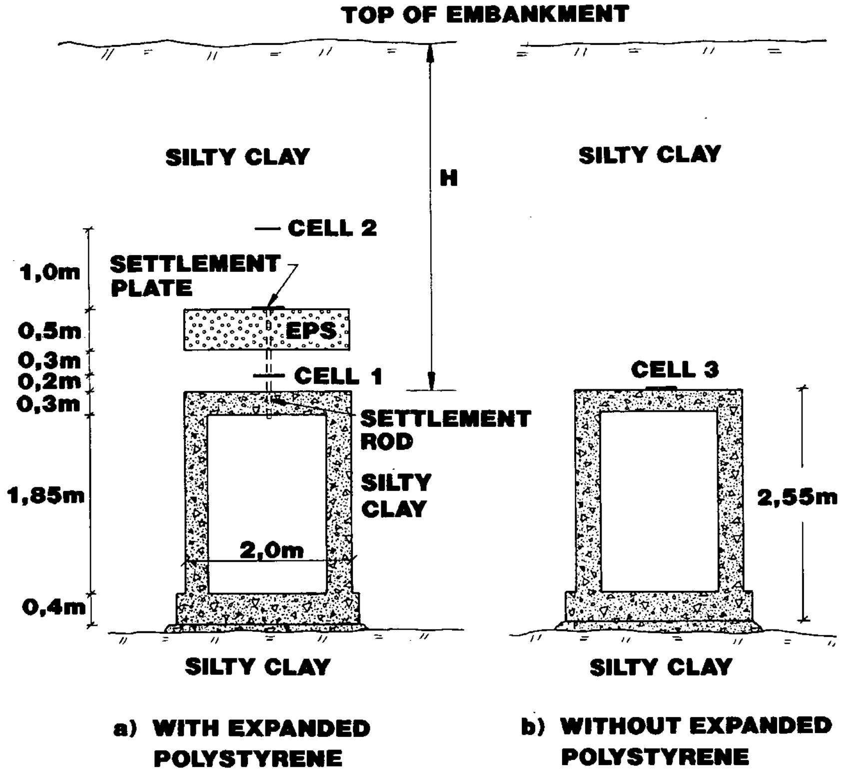

FIGURE 10 Geometry of instrumented cross section (Hallumsdalen).

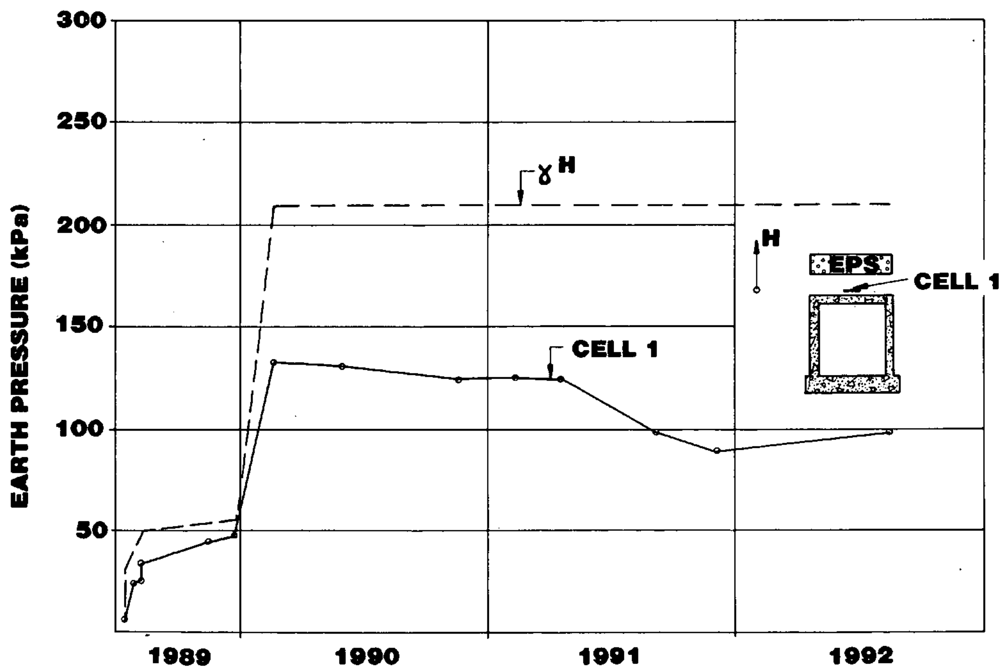

FIGURE 11 Measured earth pressure cell 1 (Hallumsdalen).

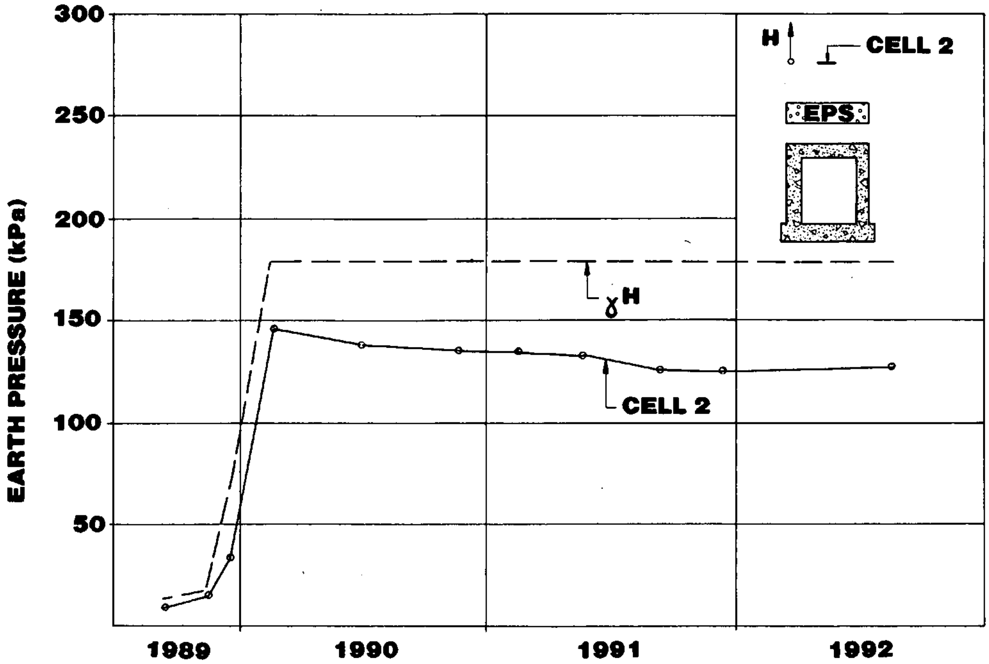

FIGURE 12 Measured earth pressure cell 2 (Hallumsdalen).

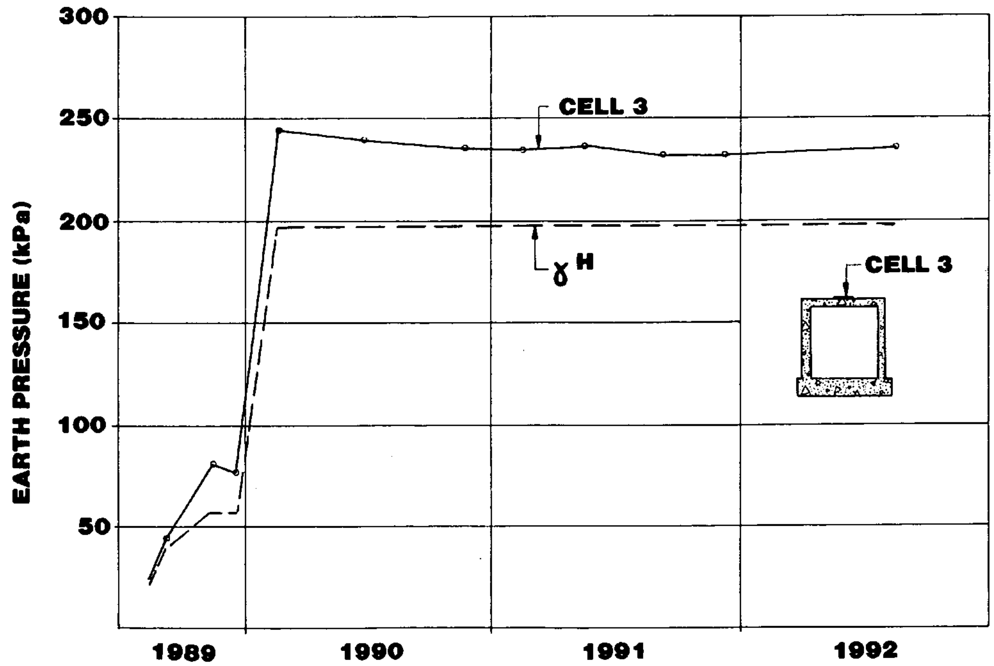

FIGURE 13 Measured earth pressure cell 3 (Hallumsdalen).

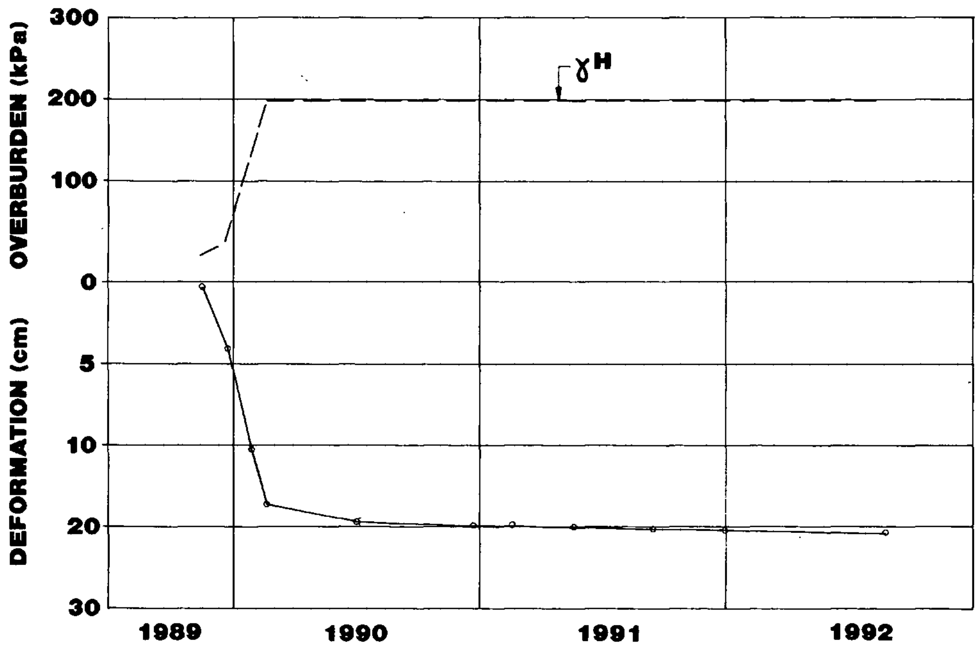

FIGURE 14 Measured deformation of EPS (Hallumsdalen).

which was 42 percent of the initial thickness of the EPS. This shows that the deformation of the EPS in a cohesive fill is greater than that in granular fills. The observed settlement of the culvert was 7 to 11 cm in the instrumented sections during the observation period.

SUMMARY AND CONCLUSIONS

The full-scale tests described show that the imperfect ditch method can be used to reduce the vertical earth pressure on rigid culverts. The EPS blocks used as the compressible material are superlight and easy to handle, and they simplify the construction procedure. Use of organic material in imperfect ditch culverts is not recommended because of the possibility of decomposition and the difficulty of specifying the material characteristics.

Two full-scale tests on concrete pipes backfilled with well-compacted sandy gravel beneath high rock fills show that the vertical earth pressure on top of the pipes was reduced to less than 30 percent of the overburden. The compression of the expanded polystyrene was 26 to 27 percent. Long-term measurements over a period of 3 years show that there was no marked increase in vertical earth pressure and compression after end of construction. Based on the full-scale tests and finite element analyses, it is recommended that a width of the EPS be used that is 1.5 times the outer diameter of the pipe.

One full-scale test was performed on a concrete box culvert backfilled with silty clay and situated below a silty clay embankment. One section with EPS and one section without EPS were instrumented.

The vertical earth pressure in the section with EPS was reduced to less than 50 percent of the overburden. The vertical earth pressure on the section without EPS was 1.24 times the overburden. The measured compression of the EPS was 42 percent of the initial thickness of 50 cm. Long-term measurements show that there was a slight increase in deformation with time.

Based on these full-scale tests and theoretical analyses, recommendations have been made for using the imperfect ditch method in Norway, where it has shown cost reductions of the order of 30 percent and has made it possible to use concrete pipes beneath higher fills (*24*).

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REFERENCES

1. Spangler, M. G. A Practical Application of the Imperfect Ditch Method of Construction. *HRB Proc.*, Vol. 37, Washington, D.C., 1958.
2. Penman, A. D. M., J. A. Charles, J. K. Nash, and J. D. Humphreys. Performance of Culvert under Winscar Dam. *Géotechnique*, Vol. 25, No. 4, 1975, pp. 713–730.
3. Höeg, K. Stresses against underground Structural Cylinders. *Journal of the Soil Mechanics and Foundation Division*, ASCE, Vol. 94, No. SM4, 1968, pp. 833–858.
4. Jean, P. A., and N. T. Long. Creation of Arching (Pneusol and other techniques): Geotechnical Instrumentation in Practice. *Proc., Institution of Civil Engineers*, Nottingham, 1990, pp. 663–670.
5. Rude, L. C. *A Study of the Imperfect Ditch Method for Rigid Culverts*. Ph.D. thesis. University of Virginia, Charlottesville, 1979.
6. Krizek, R. J., R. A. Parmelee, J. N. Kay, and H. A. Elnaggar. *NCHRP Report 116: Structural Analyses and Design of Pipe Cul-*

verts. HRB, National Research Council, Washington, D.C., 1971.
7. Terzaghi, K. *Theoretical Soil Mechanics.* John Wiley and Sons, Inc., 1943.
8. Bjerrum, L., C. J. Frimann Clausen, and J. M. Duncan. Earth Pressures on Flexible Structures—A State-of-the-Art Report. *Proc., 5th European Conference on Soil Mechanics and Foundation Engineering,* Madrid, Vol. 2, 1972, pp. 169–196.
9. Katona, M. G., et al. *Structural Evaluation of New Concepts for Long-Span Culverts and Culvert Installations.* Report FHWA-RD-79-115. FHWA, U.S. Department of Transportation, 1979.
10. Hoff, G. C. *Shock Absorbing Materials.* Technical Report 6-763. U.S. Army Engineer Waterways Experiment Station, Vicksburg, Miss., 1967.
11. Parlos, A. M., and P. M. Karananiwsky. Geoboard Reduces Lateral Earth Pressures. *Proc., Geosynthetics '87,* pp. 638–639.
12. Horvath, J. S. Using Geosynthetics To Reduce Earth Loads on Rigid Retaining Structures. *Proc., Geosynthetics '91,* IFAI, pp. 409–424.
13. Karpupapu, R., and R. J. Bathurst. Numerical Investigation of Controlled Yielding of Soil-Retaining Wall Structures. *Geotextiles and Geomembranes,* Vol. 11, 1992, pp. 115–131.
14. Frydenlund, T. E. Soft Ground Problems. *Meddelelse,* No. 61, Norwegian Road Research Laboratory, 1987.
15. Aabøe, R. 13 Years of Experience with Expanded Polystyrene as a Lightweight Fill Material in Road Embankments. *Meddelelse,* No. 61, Norwegian Road Research Laboratory, 1987.
16. Vaslestad, J. Load Reduction on Buried Rigid Pipes Below High Embankments. *Proc., Pipeline Crossings,* Pipeline Division, ASCE, Denver, 1991, pp. 47–58.
17. Katona, M. G., et al. *CANDE—A Modern Approach for Structural Design and Analyses of Buried Culverts:* User Manual; System Manual; and Reports FHWA-RD-77-5, 77-6, and 77-7. U.S. Naval Civil Engineering Laboratory, 1977.
18. Vaslestad, J. *Soil Structure Interaction of Buried Culverts.* Ph.D. thesis. The Norwegian Institute of Technology, 1990.
19. Sladen, J. H., and J. M. Oswell. The Induced Trench Method—a Critical Review and Case History. *Canadian Geotechnical Journal,* Vol. 25, 1988, pp. 541–549.
20. Leonhardt, G. Die Abmindering der Erdlast durch Anordnung von Deformationsschicten bei Rohren grosser Steifigkeit. *Steinzeuginformation,* 1978.
21. Heger, E. J. New Installation Designs for Buried Concrete Pipe. *Proc., Conference on Pipeline Infrastructure,* ASCE, Massachusetts, 1988, pp. 117–135.
22. Baardvik, G., T. H. Johansen, and F. Oset. Measurement of Tension in Culverts Under High Fills. *Proc., Field Measurements in Geomechanics,* Oslo, 1991, pp. 729–733.
23. Tadros, M. K., J. V. Benak, A. M. Abdel-Karin, and K. Bexten. Field Testing of a Concrete Box Culvert. In *Transportation Research Board 1231,* TRB, National Research Council, Washington, D.C., 1989, pp. 39–54.
24. *Foundation Engineering in Road Construction: Code of Practice* (in Norwegian). Handbook 016. Norwegian Public Roads Administration, 1990.

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