Failure of a Large Diameter Steel Culvert Located Beneath a Major Forestry Road

Proceedings of the 11th Vancouver Geotechnical Society Symposium - Forestry Geotechnique and Resource Engineering

30 May 1997

Victor A. Sowa, Ph.D., P.Eng., P.Geo. [1]

Abstract
A major forestry supply road in northern Alberta crosses two large creeks using large diameter multi-plate steel culverts which were constructed in 1972. Four years after construction one of the culverts collapsed.

The deformation of the culvert that failed was monitored prior to collapse. The maximum measured deformation was between 650 mm to 860 mm which is about 10.7 to 13.9 percent of the 6.1 m height of the culvert. These deformations far exceed the value of 58 mm that was estimated during the original design, based on the properties assumed for the culvert backfill. The design deformation is the expected value associated with the normal performance of a culvert, while the larger measured deformation is associated with the culvert failure.

Compacted clay backfill was used around both culverts and it is believed that a significant deterioration of the quality of the clay backfill after construction was responsible for the culvert failure. Exceptionally high water levels during spring floods allowed water to seep through the culvert multi-plate seams into the clay backfill. The clay backfill softened due to the presence of water, but the degree and rate of softening was accelerated when the backfill around the culvert was exposed to freezing and subsequent thawing. It is believed that the weakened clay backfill eventually yielded sufficiently to cause failure. It is also believed that the use of well compacted free-draining granular backfill within the zone of frost penetration would have prevented the culvert failure.

Following the culvert failure, the owners of the forestry road were concerned with the security of the road access and, as a result, undertook remedial measures for the second large diameter culvert. The remedial measures consisted of replacing the clay backfill with compacted granular backfill, and some structural strengthening of the culvert with two reinforced concrete beams and a concrete cap on the upper part of the culvert. The modified culvert has performed well for 20 years.

Key words: Flexible culvert, large diameter, failure, culvert backfill

Introduction
In the early 1970's one of the world's largest pulp and paper mills at that time was constructed by Procter and Gamble Cellulose Ltd. (now Weyerhaeuser Canada Ltd.) on the banks of the Wapiti River, in northern Alberta, just south of Grande Prairie. The pulp mill also constructed major access roads to the timber and pulpwood stands in the foothills to the west and the south. Corrugated steel culvert pipes were used to provide drainage beneath the access roads. The majority of the culverts were in the size range of riveted and the helical Corrugated Steel Pipe (CSP). Structural Plate Corrugated Steel Pipe (SPCSP) was used when larger culverts were required for crossing several larger streams and the performance of two of these large culverts is the subject of this article.

Two of the larger streams were the Big Mountain Creek and the Bald Mountain Creek which were located about 33 km and about 27 km respectively southwest of the mill site. The culvert at Big Mountain Creek was a horizontally oriented elliptical culvert with a major horizontal axis span of 9.1 m and a minor vertical axis span of 6.1 m, and with a soil cover of 2.2 m and about 36 m in length. The culvert at Bald Mountain Creek was a vertically oriented elliptical culvert with a vertical major axis span of 8.5 m and a horizontal minor axis span of 7.8 m, and with a soil cover of 3.0 m and about 68 m in length.

The maximum loads imposed on the road over the culvert were applied by the timber haul trucks. The total haul truck weight was typically 1500 kN. The

[1] *SRK - Robinson Inc., Burnaby*

maximum wheel load combination was a load of about 400 kN supported by dual wheels and dual axles spaced 1.8 m apart.

Both culverts were designed for maximum stream flows of 128 cubic meters per second. Both culverts also carried unusually large spring runoffs during the first and second years of operation that were well in excess of the design 100-year flood, and both the culverts performed well.

Photographs of the Big Mountain Creek culvert during construction is shown on Figure 1 and the completed Bald Mountain Creek culvert is shown on Figure 2.

Culvert Construction and Performance

Both culverts were constructed in the spring of 1972, with the Bald Mountain Creek culvert constructed in late March and early April, and the Big Mountain Creek culvert constructed in April. The specifications required that the backfill be compacted and allowed the use of either clay or granular soil as the backfill. The geotechnical firm initially involved with this project recommended that granular backfill be used. The suitability of the type of backfill material was discussed at length and in the end the contractor was allowed to use a clay backfill, provided that the contractor provided a 5-year performance bond. The Bald Mountain Creek culvert was constructed first and it was during the construction of this culvert that the detailed construction procedures were established and the construction problems were resolved. The procedures used during the construction of the Big Mountain Creek culvert were similar.

Construction of each culvert commenced by diverting the creek. The bottom of the original creek bed was excavated to a depth of about 1.0 to 1.3 m, with the base of the excavation being shaped roughly to conform to the bottom of the culvert. A 150 mm thick layer of crushed gravel was placed on the prepared subgrade of the excavated creek bottom prior to placing the bottom section plates of the culvert. The relative position of culvert and the surrounding backfill at the end of construction is shown on Figure 7b for the Big Mountain Creek culvert.

The clay soil that was used as backfill was obtained from borrow areas located on the creek banks near each of the culverts. The clay soil was a stiff clayey till material and was placed around the culvert from the level of the original creek bed to the final grade. The upper 0.6 m of the fill just below the crest of the road over the culvert consisted of crushed river gravel. The clay backfill was placed in 150 to 200 mm thick lifts and compacted with a DW20 self-propelled taper foot roller. In areas inaccessible for the compactor, especially under the haunches of the culvert, the clay fill was graded into the area and then compacted with either the rubber tire of a road grader or a vibratory hand tamper. The compaction specified for the clay backfill was a minimum of 100 percent of the maximum dry density obtained from the standard Proctor compaction test at the optimum water content. Compaction control was provided during the construction of the Big Mountain Creek culvert. No records of compaction control for the Bald Mountain Creek culvert were obtained.

Several months after the construction of the Big Mountain Creek culvert was completed and before the haul road was used by the logging trucks, a concrete slab as shown on Figure 7b was constructed over the culvert. The concrete slab was constructed 0.6 m below the top of the road and the purpose of the slab was to distribute the dynamic wheel loads more uniformly. The concrete slab was 200 mm thick and was reinforced with wire mesh. The concrete slab extended across the width of the road crest, and was 11 m wide, extending about 1.0 m past the spring line on each side of the culvert. No concrete slab was constructed for the Bald Mountain Creek culvert.

Both culverts had performed well until the Big Mountain Creek culvert collapsed on May 31, 1976, about 4 years after construction. The collapse was preceded by a distinct dip in the road as can be detected on the photograph in Figure 3. The dip in the road developed a few days before the culvert collapsed.

The crown of the culvert also deformed significantly downward as can be seen in the photograph in Figure 4. The amount of deformation was measured inside the culvert by surveyors using boats for access. The collapse of the culvert was a progressive failure, commencing with an initial reverse curvature of the crown of the culvert. The reverse curvature can also be detected by careful examination of the photographs in Figures 3 and 4. The culvert collapse was essentially symmetrical about the centerline of the crown. The collapsed Big Mountain Creek culvert, which was a snap through type of failure, is shown on the photograph in Figure 5.

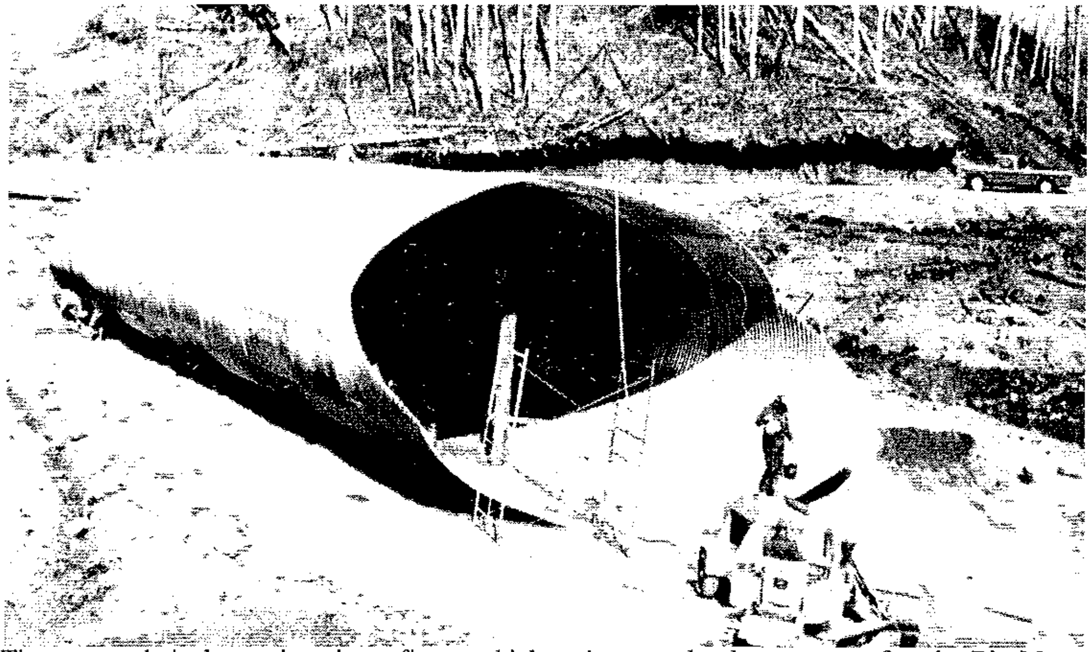


Figure 1. Big Mountain Creek culvert - assembly of culvert.

The geotechnical engineering firm which employed the writer was requested by Weyerhaeuser Canada Ltd. in June, 1976 to investigate the cause of the culvert failure. This geotechnical firm had no previous involvement with both of these culverts. The investigation was completed and a report with possible causes of failure identified was submitted to Weyerhaeuser.

The owner then became concerned with the security of the Bald Mountain Creek culvert. While the Bald Mountain Creek culvert was showing some signs of deformation, the deformation did not appear to be sufficiently excessive to suggest that failure was imminent. Nevertheless, the owner could not risk the possibility of a culvert failure that would jeopardize access to their timber supply, which if extended for a significant period, could have resulted in a mill shutdown.

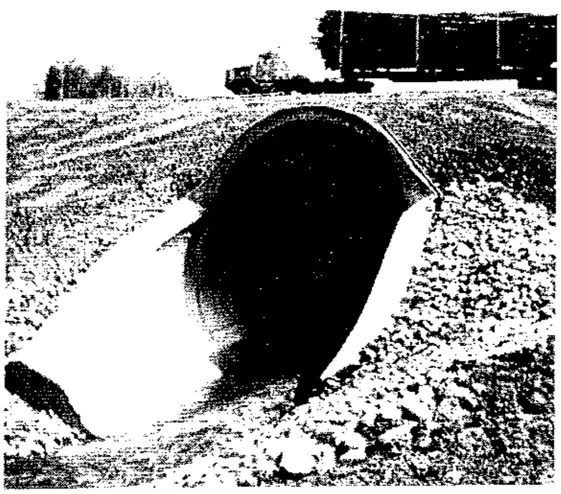


Figure 2. Bald Mountain Creek culvert - after completion of construction.

As a result, about a year after the Big Mountain Creek culvert failure, the owner decided to become proactive and undertake remedial measures to strengthen the Bald Mountain Creek culvert.

The remedial measures for the Bald Mountain Creek culvert consisted of excavating the existing compacted clay backfill to about 6.7 m below the crown of the culvert (within about 1.8 m of the invert) and replacing with compacted granular backfill. The photograph in Figure 6 shows the Bald Mountain Creek culvert while the clay backfill was being excavated. Reinforced concrete beams were constructed at approximately the 10:00 and 2:00 o'clock positions along the length of the culvert, and a thin concrete cap was constructed over the top portion of the culvert between the beams. The Bald Mountain Creek culvert has performed adequately to date for 20 years

The owner requested that the actual condition of the existing backfill should be investigated while the backfill for the Bald Mountain Creek culvert was being excavated. This investigation provided the opportunity to examine certain aspects of the compacted clay backfill that was not possible for the Big Mountain Creek culvert because the upper portion of the culvert backfill had collapsed during failure.

The results of the investigations for both the Big Mountain Creek and the Bald Mountain Creek cul verts are reported in this article.

Figure 3. Big Mountain Creek culvert - culvert deformation prior to failure.

Basic Approach Used In The Investigations

The design and performance of a steel culvert structure is very much a soil-structure interaction problem. Failure of the Big Mountain Creek culvert could have been caused by a variety of factors and two likely factors are structural failure of the steel culvert, and failure of the compacted backfill around the culvert. The structural aspects of the culvert design was examined by others and writer's firms' terms of reference was to examine the geotechnical aspects of the design, and in particular the compacted backfill. It is the results of the geotech nical investigations that are described in this article.

The main focus of the geotechnical investigation was on the quality and the condition of the backfill. The investigations were directed towards establishing the quality of the backfill in the general vicinity of the culvert, and in particular the quality of the backfill that was located immediately adjacent to the culvert. The investigations were also directed towards assessing the quality of the backfill at the end of construction and the possible deterioration of the backfill with time. The backfill in the immediate vicinity of the culvert was the area of greatest interest since it is this backfill that has the greatest influence on the performance of the culvert and may have had a major impact on the failure of the culvert.

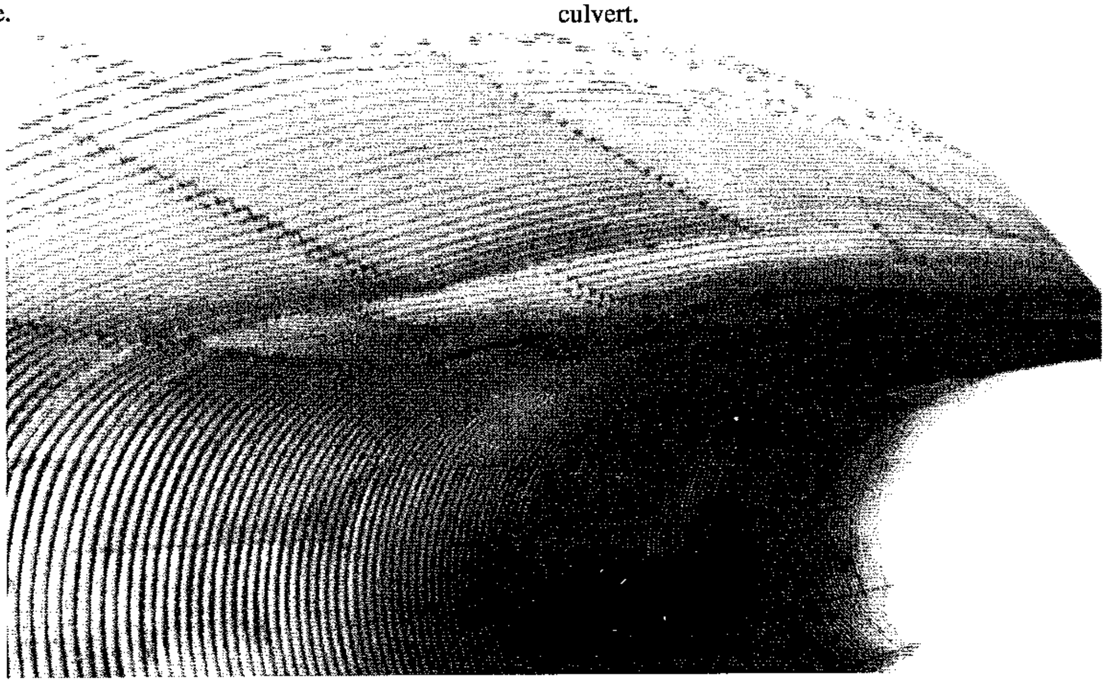


Figure 4. Big Mountain Creek culvert - deformation of crown of culvert prior to failure.

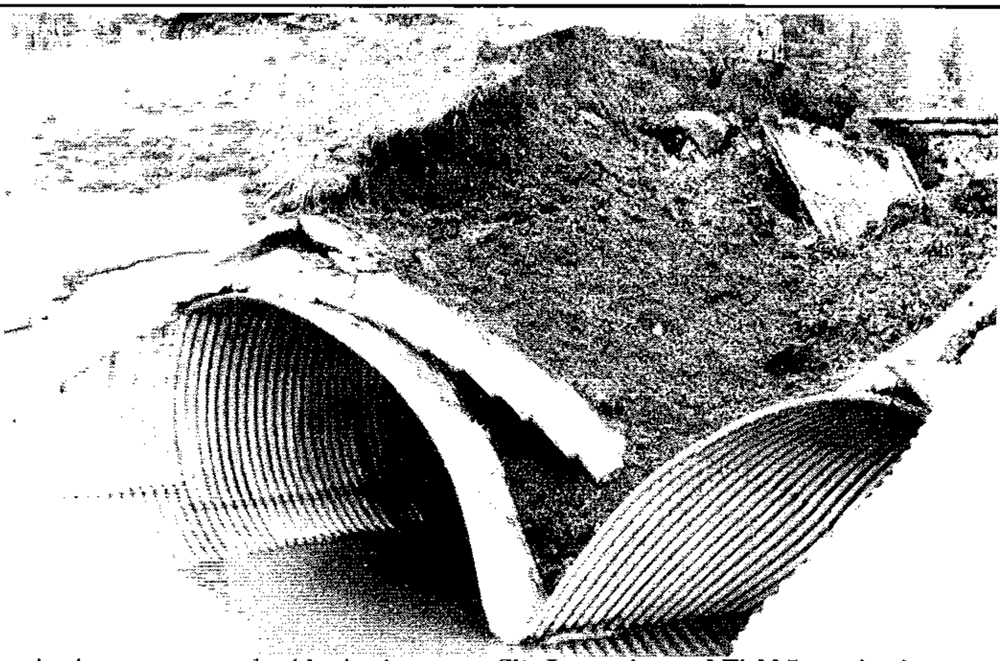


Figure 5. *Big Mountain Creek culvert - After failure.*

The investigations commenced with site inspections to examine the condition of both culverts, and the general condition of the site in the vicinity of each culvert. The overall situation at the site governed the specific direction of each site investigation. The approach was different for the Big Mountain Creek culvert compared to the Bald Mountain Creek culvert since the Big Mountain Creek culvert had failed. The approach to the site investigations evolved and was modified as it appeared that certain aspects seemed more likely to lead to the cause of the failure. Nevertheless the overall thrust was focused on the early observations that the deterioration of the compacted backfill was likely a major contributing cause of the failure of the culvert.

Site Inspection and Field Investigations

Big Mountain Creek Culvert

A site visit to the Big Mountain Creek culvert was undertaken on June 29, 1976 at the time that drilling of test holes had commenced. The site visit occurred about one month after the collapse of the culvert. The photograph given on Figure 5 illustrates that the culvert is in a complete state of collapse. The photograph also shows a broken portion of the concrete slab that was constructed above the culvert lying exposed on surface of the fill remaining on top of the collapsed culvert. Subsequent investigation revealed that the slab would not have acted monolithically, but instead as a number of small individual slabs because formwork was left in place during construction of the slab.

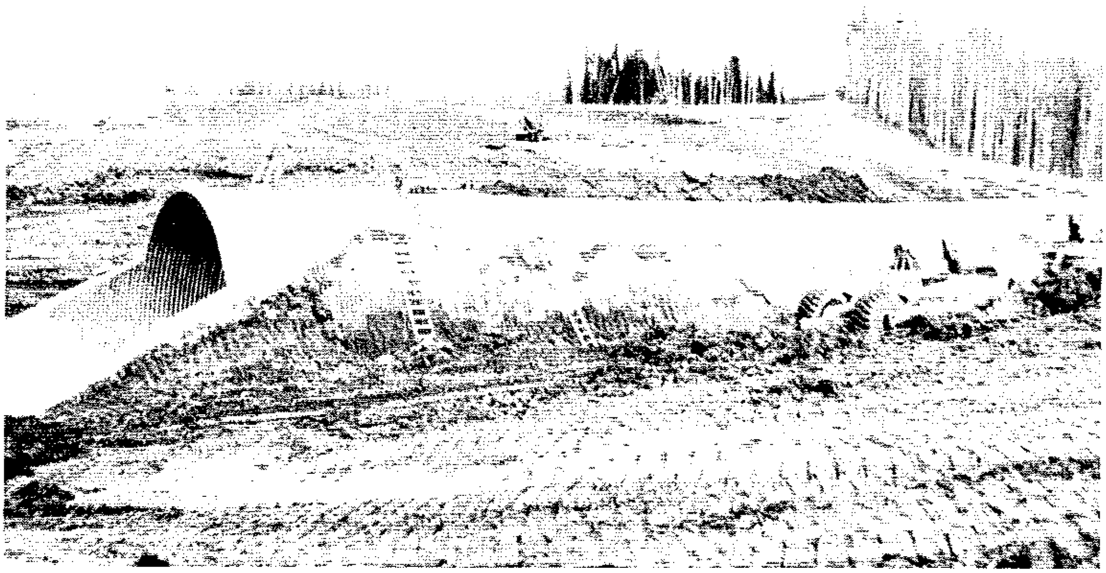


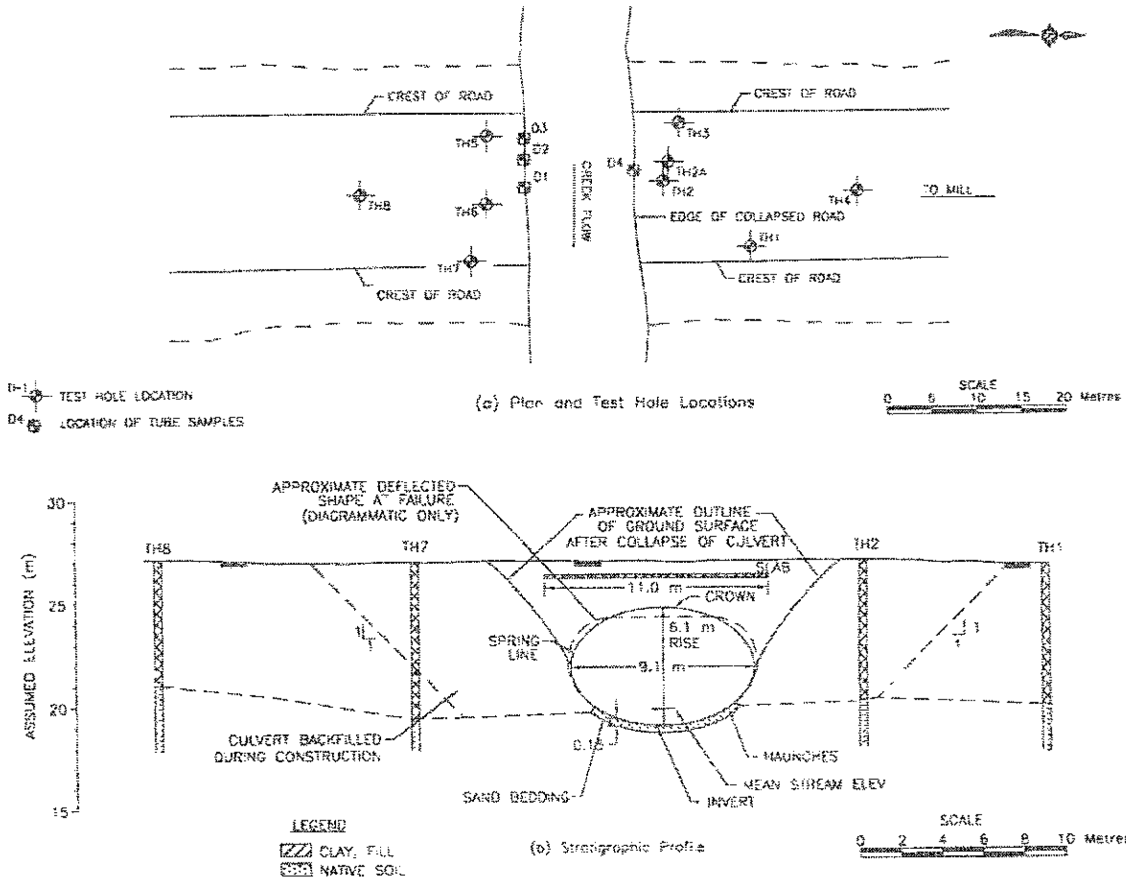
Figure 6. *Bald Mountain Creek culvert - excavation of clay backfill prior to placing granular backfill.*

One of the major items investigated during the site visit was the condition of the bolts which joined the individual culvert plate sections. Except where necking of the steel had occurred around a bolt opening along a line where the culvert steel had actually torn as a result of the failure, only one bolt was found which was not tightened completely. Based on the site inspection there was no evidence to suggest that the construction of the bolted seams led to failure of the culvert.

Another major item investigated during the field inspection was the condition of the clay backfill. A distinction should be made between the condition of the general backfill within the general vicinity of the culvert, and the condition of the backfill that is located immediately adjacent to and directly behind the culvert plates. The compaction of the clay backfill in the general vicinity of the culvert would not have been affected by the presence of the culvert, and this backfill will be referred to in the future as the general clay backfill. The compaction of the clay backfill immediately adjacent to the culvert would likely have been affected by the presence of the culvert during and after construction, and this backfill will be referred to in the future as the culvert clay backfill.

The field investigation included drilling of eight test holes at the locations shown on Figure 7a using a truck mounted hollow stem drill rig. Three test holes were drilled parallel to and as close as possible to the spring line on each side of the culvert. However, because the culvert had collapsed and the remaining collapsed fill sloped up from the culvert to the crest of the road as shown on Figure 7b, the location of the closest test hole was about 5 m from the spring line. Information available later established that these test holes were not close enough to provide information on the specific condition of the culvert clay backfill adjacent to the culvert. Three other test holes were drilled some distance from the culvert to provide information on the condition of the general clay backfill far enough away not to be influenced by the presence of the culvert. Continuous undisturbed sampling using Shelby tubes was undertaken in Test Holes 2 and 6. Undisturbed Shelby tube samples were obtained in the remaining test holes at 1.5 m intervals followed by a standard penetration test.

Figure 7. Big Mountain Creek culvert - plan, test hole locations and soil stratigraphic profile.



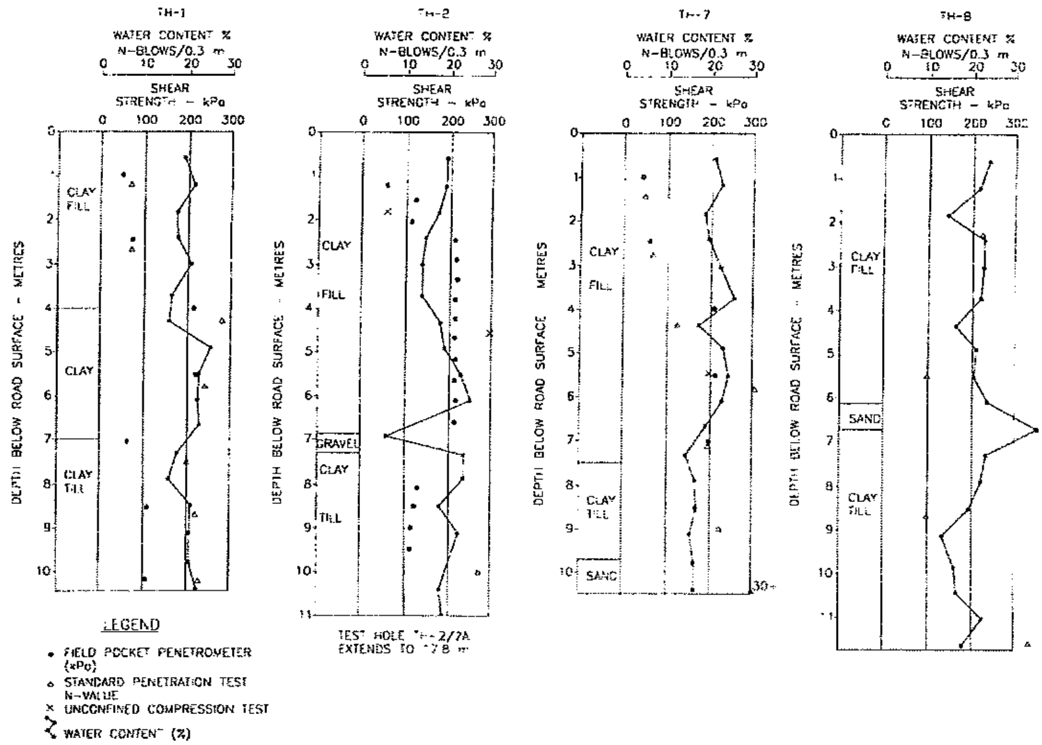


Figure 8. Big Mountain Creek culvert - test hole logs and test results.

The condition of the general clay backfill within the general vicinity of the culvert was examined in test pits which showed the clay backfill was of very stiff to hard consistency. A careful examination of the compacted clay backfill revealed that there was a noticeable blocky structure. It was evident that the compaction process had not completely pulverized the clay lumps of the excavated clay obtained from the clay borrow area. This is not unusual for stiff overconsolidated clay soils.

In addition to examining the condition of the general clay backfill in the vicinity of the culvert, several test pits were excavated immediately behind the spring line of the culvert. The test pits were specifically located in areas behind the culvert where the backfill appeared not to be disturbed by the failure of the culvert. Although the Big Mountain Creek backed-up and flooded the area after the culvert collapse, the clay soil examined in the test pits should not have been in direct contact with the backed-up water because it was either covered by at least 0.3 m of clay, or by the culvert plates.

The strength of the clay backfill exposed in the test pits was measured with a pocket penetrometer. In addition, four Shelby tube samples were obtained at the spring line in the soft zone of clay backfill directly behind the culvert walls at the approximate locations shown on Figure 7a as D1 to D4 inclusive. The clay soil immediately behind the walls of the culvert was too soft to allow obtaining tube samples, however it was possible to obtain tube samples within 300 mm to 600 mm behind the walls of the culvert.

Bald Mountain Creek Culvert

The Bald Mountain Creek culvert site was visited on April 15, 1977, about one year after the Big Mountain Creek culvert failure. The purpose of the site visit to examine the general condition of the culvert and the compacted clay backfill. Weyerhaeuser had decided that while there was no indication of imminent failure of the Bald Mountain Creek culvert, remedial measures were being undertaken since they could not risk the possibility of a future failure which could restrict access to their timber supply. The remedial measures included excavation of the clay backfill and replacement with compacted granular backfill. The field investigations for the Bald Mountain Creek culvert were undertaken concurrently with the backfill excavation operations. Consequently, it was necessary to plan and execute the field investigation program to minimize interference with the remediation work which had to be completed prior to the spring flood.

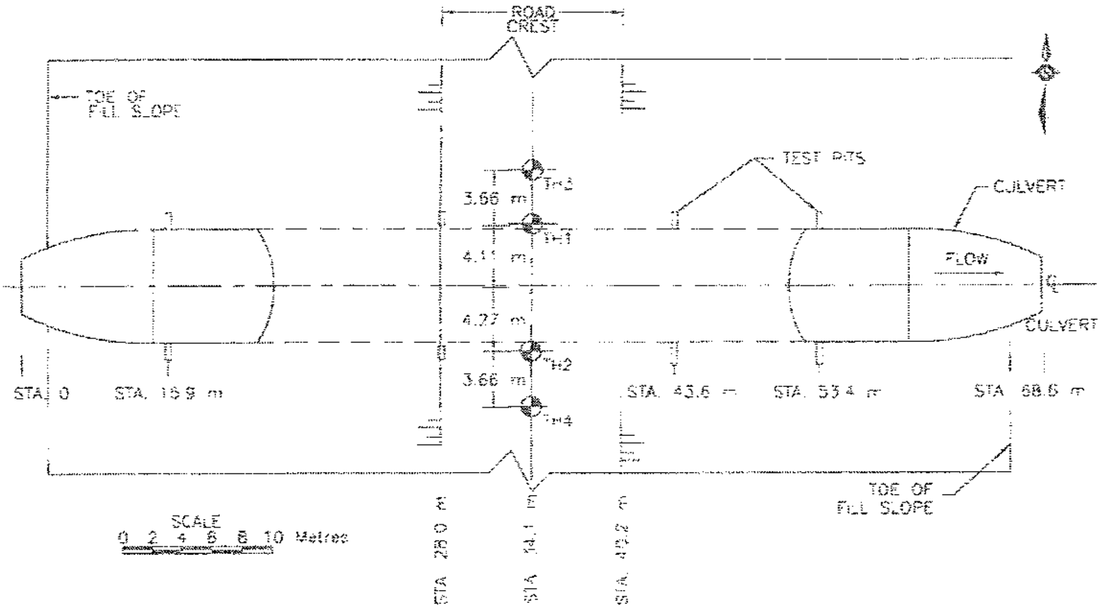


***Figure 9.** Bald Mountain Creek culvert - plan and test hole locations.*

The field investigation included drilling of four test holes from April 16 to 18, 1977 with a hollow stem auger on the road centerline at the locations shown on Figure 9. Two test holes were drilled as close as possible to the culvert, without penetrating the culvert, with one test hole on each side. The other two test holes were drilled 3.66 m further from the culvert. Test Holes 1 and 2 were drilled so that at the closest point, the test holes were 0.22 m and 0.38 m respectively from the culvert spring line. These two test holes were drilled adjacent to the culvert to provide information on the quality of the compacted culvert clay backfill adjacent to the culvert. The other two test holes were drilled further from the culvert to provide information on the general clay backfill.

Shelby tube samples were taken at 0.75 m intervals in Test Holes 1 and 2 for the upper 3 m and below 3 m Shelby tube samples were taken continuously. In Test Holes 3 and 4, Shelby tube samples were taken at about 1.5 m intervals followed by standard penetration tests. The shear strength was measured in the field using a pocket penetrometer on the ends of most of the Shelby tube samples.

Eight test pits were excavated adjacent to the culvert at the locations shown on Figure 9 while the culvert backfill was being excavated. Detailed soil sampling and testing was undertaken at various levels in the test pits. Pocket penetrometer tests were undertaken to determine the strength of the clay backfill and samples were taken for water content tests. The strength and water content tests were taken, starting directly behind the culvert walls, and extending from the culvert walls at about 150 mm intervals to a distance of about 900 mm. Based on the observations made during the site visit, it appeared that any significant softening of the clay backfill occurred within about 1.0 meter distance from the culvert walls. Some Shelby tube samples were also obtained by pushing or driving the tubes into the sides of the test pits. In addition, several bulk soil samples were taken at various locations to undertake compaction tests in the laboratory.

Laboratory Investigations

A range of laboratory tests were undertaken to provide information on the strength of the clay backfill, the compressibility, and various other parameters. The laboratory tests included the following.

- Water content tests
- Atterberg Limit tests
- Pocket penetrometer tests
- Unconfined undrained compression strength tests
- Insitu dry density of block samples or tube samples
- Standard Proctor Compaction Density tests
- Cyclic triaxial compression tests to determine the modulus of elasticity of the compacted clay soil for the short term conditions

- One dimensional load deformation creep tests (consolidation tests) to provide the modulus of deformation of the compacted clay soil for the long term conditions

The results of these laboratory tests are summarized in various tables and figures which will be referred to in the subsequent discussions. It should be noted that to provide a common basis, the strength results obtained from either the pocket penetrometer tests or from the unconfined compression tests are given as the undrained shear strength. Also, the maximum undrained shear strength that can be measured with the pocket penetrometer is 216 kPa, and when the strength is larger than 216 kPa, the result is given as P.P. = 216+ kPa.

Evaluation of the Results of the Field and Laboratory Investigations

Big Mountain Creek Culvert

A simplified profile illustrating the general stratigraphy is shown on Figure 7b and simplified logs for four typical test holes are shown on Figure 8. The general soil stratigraphy shows that the soil consists of compacted clay fill overlying the insitu "till like" material. In Test Holes 4 and 8 a thin stratum of alluvial sand and gravel was encountered immediately beneath the fill. A thin layer consisting largely of crushed gravel was encountered in Test Hole 2 and this layer is likely a part of the gravel fill used as bedding material for the culvert. The natural soil at the base of the creek valley consisted of a dark grey till-like clay. This stratum extended to the maximum depth drilled of 17.8 m and is a medium to high plastic clay soil of stiff to very stiff consistency, with silt and sand lenses. This material is very similar to the soil excavated from the borrow area which was used as backfill around the culvert. The fill, consisting of a medium to high plastic clay to silty clay, was encountered in all the test holes.

Groundwater was encountered in Test Holes 2, 4 and 8 and the water level stabilized at approximately the elevation of the sand and gravel seams while the other test holes were effectively dry. It is believed that the stabilized groundwater elevation corresponds to about the mean creek surface elevation.

Compaction control during the original construction consisted of undertaking insitu density tests and the results are summarized in Table 1. The maximum standard Proctor compaction dry density is 18.84 kN/cu.m and the optimum water content corresponding to the maximum dry density is 17.1 percent.

The minimum specified compaction density was 100 percent of standard Proctor dry density at the optimum water content. The insitu densities measured during the construction that are given in Table 1 are based on three Shelby tube samples and three chunk samples taken from the compacted clay backfill. The location of these samples is not known except that they were in the vicinity of the culvert. The insitu densities from these six samples ranged from 95.4 to 100.1 percent of the maximum dry density, and the water content varied from -0.3 to +0.6 percent of the optimum water content. The undrained shear strength measured from the unconfined compression tests and the pocket penetrometer tests ranged from 192 to 240 kPa, which means that the compacted clay backfill was classified as being of hard consistency.

The condition of the general clay backfill observed from testing samples from the test holes drilled after the culvert failed is described next. The results of laboratory tests to determine the water content, the dry density, the shear strength, and the Atterberg Limits on Shelby tube samples are summarized in Table 2 and on the simplified test hole logs given on Figure 8 for four representative test holes. The results of Atterberg Limit Tests suggest that there are two basic clay soil types; a medium plastic soil with a plastic limit which ranges from about 15 to 19 percent and a liquid limit which ranges from about 43 to 49 percent, and a highly plastic soil with a plastic limit of about 22 percent and a liquid limit of about 69 percent.

The undrained shear strength measured from the unconfined compression tests and the pocket penetrometer tests are given in Table 2 and on Figure 8. The results show that the undrained shear strength of the compacted general clay backfill in the upper 1.5 m to 2.5 m below the top of the road fill ranged from 50 to 120 kPa, compared to 192 to 240 kPa at the time of construction. It is believed that the reduction in strength is due to weathering resulting from desiccation and from freeze-thaw cycles. The undrained shear strength below about the 2.5 m depth level was 216+ kPa, essentially the same as at the end of construction. These results indicate that general weathering did not affect the compacted clay soil at depths greater than 2.5 m.

Test pits were excavated next to the culvert and the results of pocket penetrometer tests undertaken in the test pits on the compacted culvert clay backfill adjacent to the culvert showed a significant reduction in soil strength. In the first 50 mm immediately behind the culvert wall, the clay was so soft that it was not possible to record a pocket penetrometer reading. Between 150 to 200 mm behind the culvert wall, the strength was in the order of 50 to 70 kPa, and at a distance of 300 to 380 mm, the strength was in the range of 120 to 140 kPa. The above results show that significant strength decrease and softening had occurred immediately adjacent to the culvert wall, with the effect being less pronounced as the distance from the culvert wall increased. Tube samples of the very soft clay soil next to the culvert wall could not be obtained but a few tube samples were obtained within 300 to 600 mm behind the culvert wall.

The results of the insitu dry density tests of the compacted general clay backfill determined from tube samples taken in Test Holes No. 2, 5, 6, and 7 are summarized in Table 2. These results indicate that the dry density ranged from 16.60 to 18.90 kN/cu.m which is about 88.2 to 100.3 percent of the maximum Proctor dry density. The two density values that were less that 16.60 kN/cu.m could be for the material with a high plasticity and as a result a lower standard Proctor density would apply. The dry density of the two samples of backfill, D2 and D4, located immediately behind the culvert wall at the spring line was lower, and ranged from 14.00 to 16.87 kN/cu.m, which is about 74.3 to 89.6 percent of the maximum Proctor dry density. These results indicate that there was a significant reduction of the density immediately adjacent to the walls of the culvert.

The water content tests on the tube samples of the general clay backfill from the Test Holes No. 2, 5, 6, and 7 are summarized in Table 2, and on Figure 8. These results show that the water content of the general clay backfill ranged from about 15 to 25 percent. The range is due to the variability of the water content during the construction, and partly reflects whether the clay backfill could either be a medium or high plastic clay.

The water content of one of the tube samples taken immediately adjacent to the wall of the culvert at the spring line at D4 was 30.8 percent which is considerably larger than the optimum value of 17.1 percent. The larger water content corresponds to a significant reduction in strength to a value of 56 kPa that was obtained during the confined cyclic test.

The overall conclusion of the tests on the soil strength, the density, and the water contents is that there was significant deterioration of the compacted general clay backfill within the depth of 2.5 m below the road surface due to weathering, but with essentially no change below 2.5 m. There was even a more significant deterioration of the compacted culvert clay backfill adjacent to the culvert walls which could be the result of weathering, or lower compaction during the original construction, or a combination of both factors.

The elastic properties of the compacted clay backfill soil were used in the determination of the soil-structure interaction and in assessing the original design of the culvert. The elastic properties were determined from laboratory tests and the results are summarized in Table 3.

***Table 1.** Big Mountain Creek Culvert - Summary of the Density and Strength of the Clay Backfill During Construction*

Sample No.	Undrained Shear Strength - kPa*	Dry Unit Weight - kN/m^3	Water Content %	Proctor Compaction † %
Shelby Tube				
1	230	18.82	16.8	100.0
2	240	18.87	17.4	100.1
3	202	17.91	17.7	95.4
Chunk Samples				
1	216	18.71	17.0	99.3
2	225	18.80	17.3	99.9
3	192	18.05	17.2	96.7

* From unconfined compression tests on Shelby Tube samples and pocket penetrometer tests on chunk sam ples.

† Maximum Proctor Density = 18.84 kN/m3 at an optimum water content of 17.1%.

Material	Test Hole Number	Sample Number	Depth (m)	Water Content (%)	Dry Unit Weight kN/m³	Plastic Limit (%)	Liquid Limit (%)	Specific Gravity	Undrained Shear Strength: Unconfined Compression (kPa)	Undrained Shear Strength: Confined Cyclic (kPa)
Test Holes										
Fill	2	U3	1.8	19.5	17.19				57	
Fill	2	U9	4.6	16.7	17.66				340	
Fill	2	U10	4.9	14.6	18.9	14.8	43.6	2.7		398
Fill	5	U4	5.5	20.4	16.6				312	
Fill	6	U4	2.4	15.5	18.1				249	
Fill	6	U9	5.5	17.6	16.83	19.1	48.6	2.68		307
Fill	6	U10	5.8	23.1	15.54				216	
Fill	7	U4	5.5	21.8	15.8				201	
Samples at Culvert Spring Line, (Test Pits)										
Fill	-	D2	5.2	19.7	16.87	15.9	43.2	2.66		79
Fill	-	D4	5.2	30.8	14	21.6	68.8	2.73		56
Foundation Soil Samples, (Test Holes)										
Till	2	U17	8.8	19.5	16.95				67	
Till	6	U18	9.5	18.7	-				110	

Table 2. *Big Mountain Creek culvert - after failure - Summary of Laboratory Testing on Tube Samples*

walls which could be the result of weathering, or lower compaction during the original construction, or a combination of both factors.
The elastic properties of the compacted clay backfill soil were used in the determination of the soil-structure interaction and in assessing the original design of the culvert. The elastic properties were determined from laboratory tests and the results are summarized in Table 3.

The live cyclic loading applied by the haul trucks was approximated by cyclic triaxial compression tests on the compacted clay backfill samples taken during drilling. The cyclic tests were undertaken at confining pressures approximately equivalent to the soil pressure at a depth between the crown and the spring line of the culvert. The confining pressures selected took into account the larger soil pressures adjacent to the sides of the culvert due to arching as observed by Lefebvre et al. (1976) and Byrne et al. (1990)

Cyclic live load stresses were applied at two or three axial compression stress levels to each of the triaxial samples. The corresponding cyclic modulus of elasticity values, E_c, are given in Table 3, and typically range from 150 to 200 MPa for the general fill, and about 35 MPa for the deteriorated back-

Table 3. *Big Mountain Creek culvert - after failure - Summary of Moduli of Elasticity and Deformation.*

Test Hole Number	Sample Number	Sample Depth (m)	Cyclic Triaxial**: Ec* (MPa)	Cyclic Triaxial**: Average Compressive Pressure (kPa)	Secant Modulus Triaxial Es* (MPa)	Deformation Modulus (D m)* Consolidometer: Constrained (MPa)	Deformation Modulus (D m)* Consolidometer: Corrected v = 1/3 (MPa)
General Fill							
2	U10	4.9	203	69	34.4		
			199	103			
			532	138			
6	U9	5.5	284	34	31	5.7	3.9
			150	69			
			153	103			
Deteriorated Culvert Fill							
-	D2	Spring Line	37.6	69	1.8	4.6	3.1
			52	103			
-	D4	Spring Line	38.7	69	1.6	4.1	2.8
			11.3	103			

* E c = Cyclic modulus of elasticity; E s = Secant modulus of elasticity; D m = Deformation modulus
**Confining Pressure = 96 kPa

Fill Condition	Cyclic Live Loading [illegible] MPa	Backfill Load on Culvert: End of Construction [illegible] MPa	Backfill Load on Culvert: Long Term Loading Modulus [illegible] MPa
[illegible]	[illegible]	[illegible]	
[illegible]	[illegible]	[illegible]	[illegible]

***Table 4.** Big Mountain Creek culvert - Suggested Design Values for Moduli of Elasticity and Deformation.*

ble for the culvert load applied by the backfill at the end of construction since the general fill has not changed much since construction. The same type of compression tests were undertaken on the deteriorated backfill adjacent to the culvert and the corresponding considerably lower secant modulus values of approximately 1.7 MPa are given in Table 3.

The long term time dependent soil deformation properties are more closely represented by the one-dimensional load creep deformation test (consolidation test). The consolidation test approximates the long term yielding of the clay backfill caused by the side pressure of the culvert on the backfill due to the culvert overburden pressure. This test also simulates, to some extent, the softening that can occur over time if water is allowed into the backfill. Typical values of the deformation modulus for the general fill and the deteriorated fill adjacent to the culvert are given in Table 3. Representative suggested design values for the modulus of elasticity and the long-term loading modulus of deformation are given in Table 4. The use of these values is illustrated later.

Bald Mountain Creek Culvert

The soil stratigraphy at the Bald Mountain Creek site was very similar to the Big Mountain Creek site. The compacted clay backfill extended to the invert of the culvert as shown on Figure 10. The backfill consisted of a medium to high plastic clay to silty clay. The foundation soil below the invert consisted of sand and gravely sand which may have been the bedding material for the culvert placed during construction. Below the sand bedding the foundation soil consisted of a very stiff natural clay till. The groundwater levels measured in the standpipes that were installed during the site investigation are shown on Figure 10.

On the basis of some of the earlier test results it became apparent that there was some variability in the clay soil used for the backfill. As a result a number of Atterberg Limit Tests and compaction density tests were undertaken. The results of all the Atterberg Limit Tests on samples from the test pits and the test holes are summarized in Table 5. The results of the Atterberg Limit Tests on the test hole samples are also summarized on the test hole logs on Figure 10. The results of the Proctor compaction density tests are summarized in Table 6 and on Figure 11. The Atterberg Limit Tests which specifically correspond to the compaction tests are also summarized in Table 6.

The Atterberg Limit Test results and the Proctor compaction tests indicate that there are basically two main soil types; a medium plastic clay with liquid limits in the 40 to 50 percent range and plastic limits in the range of 15 to 18 percent. The other group is a highly plastic clay with liquid limits in the range of 50 to 70 percent and plastic limits in the range of 19 to 24 percent. The results of compaction tests corresponding to the two main soil types indicate that the maximum dry density and the optimum water content for the medium plastic clay was in the range of 16.8 kN/cu.m and 18.4 percent respectively, and for the highly plastic clay, 15.6 kN/cu.m and 22.5 percent respectively.

The variability in the backfill soil type creates some difficulty in assessing whether there is a change in the quality of the clay backfill if the assessment is made on the basis of comparison of water contents and backfill density values, or whether the difference is due to different soil types. For example, an apparently higher water content which implies a lower strength may not necessarily be a correct correlation if the clay is highly plastic, since a higher water content can be associated with the same strength that would be obtained for a lower plastic clay with a lower water content. Consequently, comparison of the backfill density to the standard Proctor density values can only be made if similar soil types are compared. This means that backfill densities must be compared with Proctor tests undertaken on the same samples, or that other classification tests such as Atterberg Limit Tests indicate that the soil types are similar so that comparisons are valid.

Some comparisons of the change in quality of the backfill soil can be made on the basis of indirect evidence obtained from soil strength tests such as

Sample Location*	Sample Number	Depth (m)	Liquid Limit (%)	Plastic Limit (%)	Plasticity Index (%)
Test Holes					
TH-1	U3	2.1	48.2	17	31.2
TH-1	U5	3.4	38.5	19.9	18.6
TH-1	U17	7	43.8	19.6	24.2
TH-1	U29	10.7	55.7	20.3	35.4
TH-2	U24	9.1	70	24.2	45.8
Test Pits*					
28-3.66N		5.8**	61.3	20.4	40.9
34.1-0.9S		1.8 (above culve crown)	41.6	15	26.6
27.7-14.0S		0.6-1.2**	40.5	17.3	23.2
43.6-3.66N		2.9**	50.8	18.1	32.7
43.6-2.13N		0.8**	47	17.8	29.2
43.6-0		0.3-0.9 (above culvert crown)	47.8	19.2	28.6

Table 5. Bald Mountain Creek culvert - Atterberg Limits.

* Sample location - 28-3.66N indicates that the soil sample was taken at S 28m, 3.66m north of the centreline of the culvert. Depths are below culv crown, unless otherwise indicated.

** Depth below crown of culvert for samples in test pits.

means that backfill densities must be compared with Proctor tests undertaken on the same samples, or that other classification tests such as Atterberg Limit Tests indicate that the soil types are similar so that comparisons are valid.

Some comparisons of the change in quality of the backfill soil can be made on the basis of indirect evidence obtained from soil strength tests such as the unconfined compression tests and pocket penetrometer tests. The unconfined compression tests were undertaken in the laboratory only. The pocket penetrometer tests were undertaken in both the field and the laboratory on the ends of the tube samples.

The results of the pocket penetrometer tests (PP) taken in the field, and the unconfined compression tests and the pocket penetrometer tests done in the laboratory, are summarized in Table 7 and on Figure 10 for the test holes. The results of pocket penetrometer tests undertaken on the soil samples adjacent to the culvert are shown on Figures 12 and 13. Since the investigation was undertaken in mid-April, the soil adjacent to the inside of the culvert could be frozen and this could also apply for portions of the test holes near the culvert, and the test pits adjacent to the culvert. Consequently, the pocket penetrometer tests results on the frozen soil will be larger than for unfrozen soil. Pocket penetrometer tests done in the laboratory and the unconfined compression tests are not affected since the tests were done on thawed samples.

The maximum extent of frost penetration in Grande Prairie typically occurs in March, and thawing typically commences in April. The specific condition in mid-April at the time of drilling is uncertain, but very likely much of the backfill extending about 1.5 to 2 m behind the culvert wall was still frozen. Some thawing may have commenced for a relatively thin zone of backfill immediately behind the

Table 6. Bald Mountain Creek culvert -Summary of Proctor Compaction Tests and Atterberg Limits.

Sample* Location	Depth**(m)	Max. Dry Density kN/m³	Optimum Moisture Content (%)	Liquid Limit (%)	Plastic Limit (%)	Plasticity Index (%)
43.6-3.66N	2.9 m	16.73	18.5	50.8	18.1	32.7
43.6-2.13N	0.8 m	16.81	18.8	47	17.8	29.2
43.6 - 0	0.3-0.9 m above culvert crown	16.73	18	47.8	19.2	28.6
28.0-3.66N	5.8 m	15.52	23	61.3	20.4	40.9
15.9-3.66S	4.3 m	15.82	21.7			
34.1-0.9S	1.8 m above culvert crown	17.08	16.5	41.6	15	26.6
27.7-14.0S	0.6-1.2m	17.38	15.5	40.5	17.3	23.2

*Sample Location - 43.6 - 3.66N indicates that the soil sample was taken at Station 43.6 m, 3.66 m north of the centreline of culvert, at a depth of 2.9 m below the crown of the culvert.

**Depth below crown at culvert.

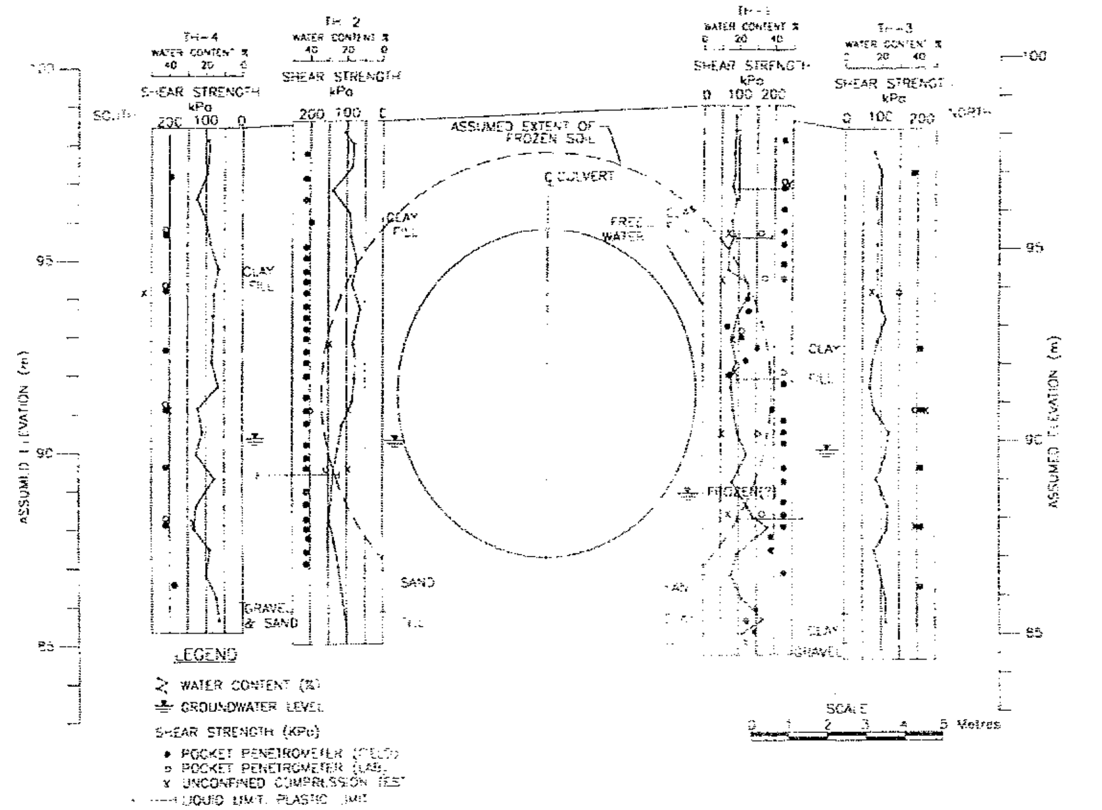


***Figure 10.** Bald Mountain Creek culvert - Soil stratigraphy, location of culvert and test results.*

these test holes measured with the pocket penetrometer in the field were mostly larger than the maximum measurable strength of 216 kPa, given in Table 7 and on Figure 10. The shear strength measured from the unconfined compression tests and pocket penetrometer tests in the laboratory were generally similar, and typically were about 150 to 200 kPa. It was concluded that the general clay backfill located far enough away from the culvert was compacted adequately during construction, and at the time of drilling had not changed much since construction.

The condition of the compacted clay backfill closest to the culvert is of interest since it is the condition of this backfill which governs the performance of the culvert. Test Holes 1 and 2 are located 0.22 and 0.38 m respectively from the spring line of the culvert at the closest point. The central portions of these test holes which are directly opposite the culvert can be considered as being the culvert clay backfill adjacent to the culvert, although the test holes are not as close as the test pits. In view of the close proximity of these test holes to the culvert, probably most, if not all, of the soil samples within the height of the culvert were frozen. Consequently the results illustrated on Figure 10 of pocket penetrometer tests taken in the field on the ends of the frozen tube samples will overestimate the shear strength. Within the height of the culvert, the only shear strength values that can be accepted with confidence are the values determined from unconfined compression tests on the thawed tube samples in the laboratory, and the corresponding laboratory pocket penetrometer test results.

The shear strength values in Test Holes 1 and 2 are evaluated, bearing in mind the preceding consider-

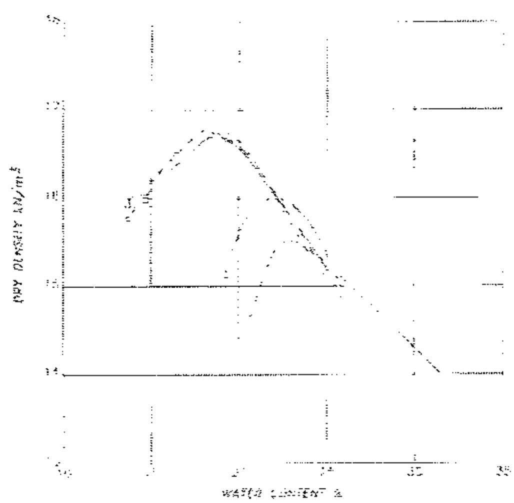


***Figure 11.** Bald Mountain Creek culvert - Standard Proctor Compaction test results.*

Sample Location	Sample Number	Depth** m	Soil Type	SHEAR STRENGTH Pocket Penetrometer kPa	Unconfined kPa	Water Content %	Dry Unit Weight kN/m³	Wet Unit Weight kN/m³
Test Holes								
TH-1	U3	2.1	Clay	216+	230	16.8	18.03	21.07
	U5	3.4	Clay	216+	69	17.1	15.80	18.51
	U9	4.6	Clay	158	58	23.0	15.44	18.99
	U14	6.1	Clay	105	81	24.3	16.46	20.47
	U17	7.0	Clay	216+	88	15.7	18.69	21.63
	U22	8.5	Clay	154	52	17.5	15.80	18.57
	U29	10.7	Clay	168	67	28.3	14.89	19.10
TH-2	U13	5.8	Clay	—	145	16.1	17.25	20.03
	U19	. 7.6	Clay	202	96	16.8	16.67	19.46
	U24	9.1	Clay	154	88	26.4	15.25	19.28
TH-3	U3	4.3	Clay	154	76	20.3	17.07	20.55
	U5	7.3	Clay	197	221	18.6	17.36	20.58
	U7	10.4	Clay	—	192	24.4	15.46	19.23
TH-4	U3	4.3	Clay	216+	268	17.0	17.78	20.80
	U5	7.3	Clay	216+	196	15.4	18.08	20.88
	U7	10.4	Clay	216	132	21.9	15.99	19.50
Test Pits Adjacent to Culvert								
15.9-3.66S*		4.3*** (0-0.3)	Clay	48	26	30.2	14.28	18.58
53.4-3.66S*		4.3*** (0-0.3)	Clay	34	29	26.0	15.40	19.39

*Sample Location - 15.9 - 3.66S indicates that the soil sample was taken at Station 15.9 m, 3.66 m south of the centreline of the culvert, at a depth of 4.3 m below the crown of the culvert.
**Depth below top of ground surface
***Depth below crown of culvert

***Table** 7. Bald Mountain Creek culvert - Summary of undrained shear strength from unconfined compression tests and pocket penetrometer tests.*

pocket penetrometer test results.

The shear strength values in Test Holes 1 and 2 are evaluated, bearing in mind the preceding considerations. The undrained shear strength results measured with a pocket penetrometer in Test Holes 1 and 2 are shown on Figure 10 and these were generally larger than the maximum measurable value of 216 kPa, but most of these tests were likely on frozen soil within the zone extending from the crown to the invert of the culvert. The major exception occurred in Test Hole 1 at a depth of approximately 5 m below the ground surface and opposite the 2:00 to 3:00 o'clock position. In that location the undrained shear strength values from the pocket penetrometer ranged from 70 to 120 kPa. The lower results mean that either the ground was not frozen or that the amount of freezing did not affect the strength of the soil significantly.

During drilling of Test Hole 1, free water was observed at the 5.2 m depth which essentially corresponds to the 2:00 o'clock position, and as a result the ground was not frozen in this area.

The shear strength obtained from unconfined compression tests in the laboratory on thawed tube samples from Test Holes 1 and 2 is given in Table 7 and also shown on Figure 10. The shear strength in Test Hole 2 was about 100 kPa and generally ranged from 52 to 88 kPa in Test Hole 1. These shear strength results are significantly smaller than that obtained from the pocket penetrometer tests on the frozen soil. The shear strength measured in Test Holes 1 and 2 is also significantly smaller than in Test Holes 3 and 4, and this provides evidence of a reduction in the quality of the compacted backfill adjacent to the culvert.

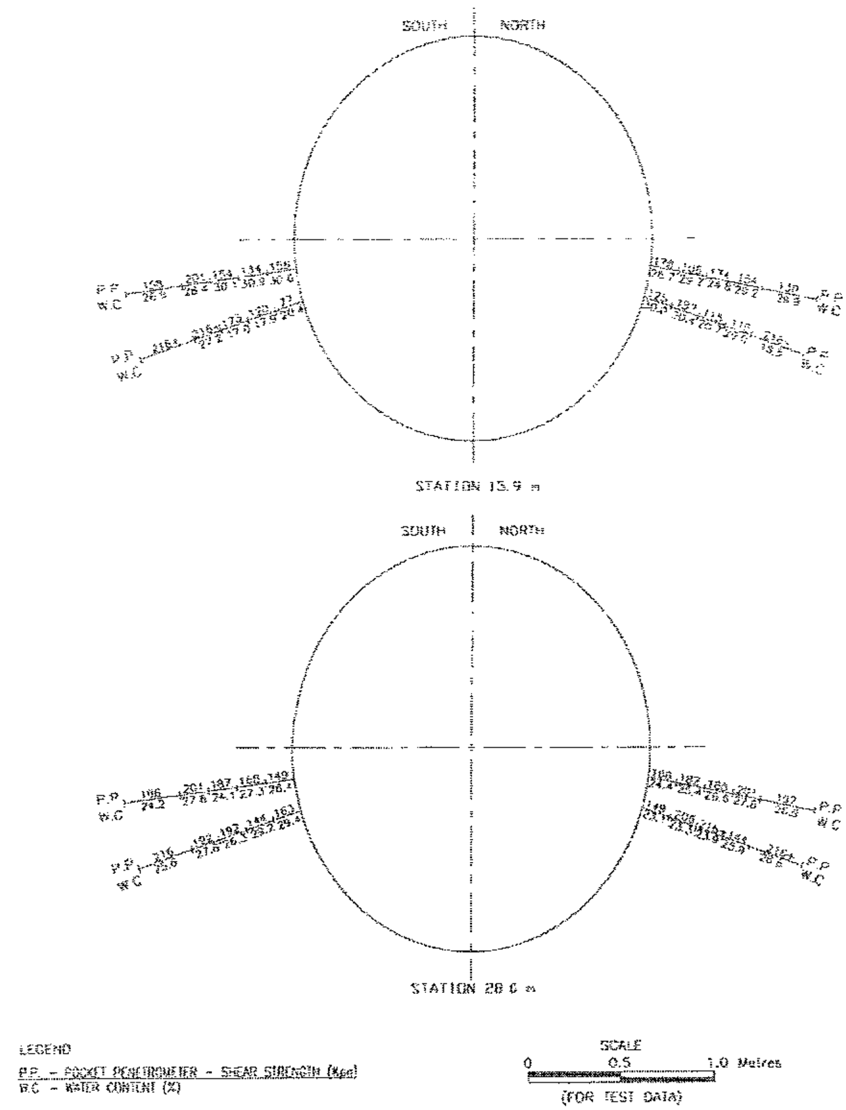


Figure 12. Bald Mountain Creek culvert - Summary of strength and water contents adjacent to the culvert for Sta. 15.9 m and Sta. 28.0 m.

location of the test pits is shown on Figure 9 and each test pit is identified by the distance (Station) from the western end of the culvert. Test pits were excavated at Stations 15.9 m, 28.0 m, 43.6 m, and 53.4 m. A larger emphasis is placed on the test pits at Stations 28.0 m and 43.6 m since these test pits had the greatest depth of fill on the culvert.

Testing in the test pits consisted of taking pocket penetrometer tests and water content samples, starting directly behind the culvert wall, and along radial lines from the culvert walls at intervals of about 150 mm from the culvert wall to a maximum distance of 900 mm. The shear strength results observed from the pocket penetrometer tests and the water content results are summarized on Figures 12 and 13. Since there is some variability of the type of clay backfill, samples were taken from several locations adjacent to the culvert to determine the Proctor density results and the results are given in Table 6. The location of the Proctor test samples is given in Table 6.

The compacted clay backfill adjacent to the inside of the culvert walls was frozen since the test pits were excavated in mid-April. Some thawing of the frozen ground could have been occurring during test pit excavation, depending on the length of time that the test pit was open. The test pits could not be left open long enough to allow complete thawing because of the need to complete the remedial work prior to the spring flood. Consequently, the pocket penetrometer tests were undertaken on backfill which could range from being frozen, partially frozen, or thawed. Unfortunately, unless there is specific evidence to the contrary, it is necessary to

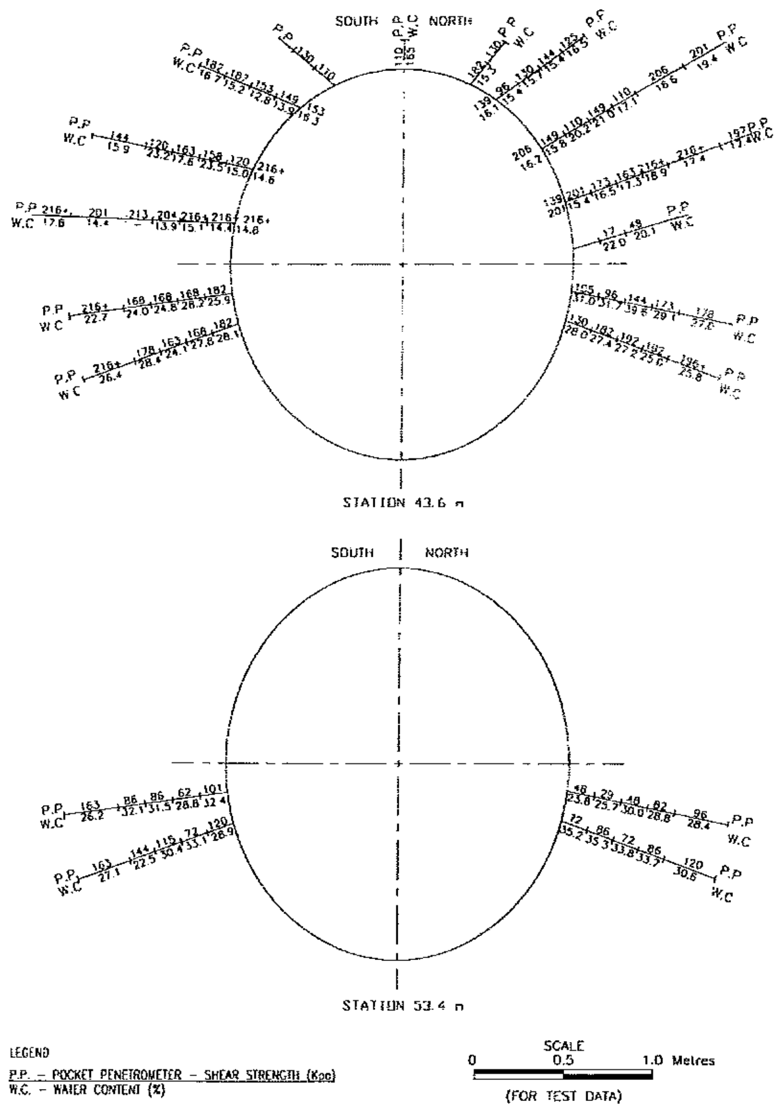


***Figure 13.** Bald Mountain Creek culvert - Summary of strength and water contents adjacent to the culvert for Sta. 43.6 m and Sta. 53.4 m.*

assume that the pocket penetrometer tests overestimate the shear strength. The exceptions to this assumption are considered below.

The results given on Figures 12 and 13 for the test pits adjacent to the culvert indicate that even if the soil is frozen or partly frozen, the undrained shear strength obtained from the pocket penetrometer was less than for the unfrozen soil in Test Holes 3 and 4. The undrained shear strength from the pocket penetrometer tests on Test Holes 3 and 4 was generally 200 kPa or larger, while only a few results in the test pits reached 200 kPa. Most of the results in the test pits were in the 100 to 200 kPa range, but there are localized areas where the shear strength was less than 100 kPa. There are other examples when the shear strength determined from the pocket penetrometer tests was so low that the clay could not be frozen, or if so, only slightly frozen. For example at Station 43.6 m, north side, just above the spring line and at Station 53.4 m, north side, just below the spring line the shear strength values measured with the pocket penetrometer at a distance of 150 mm to 450 mm from the culvert wall were in the 17 and 48 kPa range.

Unconfined compression tests were undertaken on two thawed Shelby tube samples that were taken at Stations 15.9 and 53.4 m, south side, at the spring line within 0 to 300 mm of the culvert wall. The results are given in Table 7. The undrained shear strength at Station 15.9 m was 26 kPa, at a water content of 30 percent, and the shear strength obtained from the pocket penetrometer test was 48 kPa. The shear strength for the Shelby tube sample taken at Station 53.4 m was 29 kPa, at a water content of 26 percent, and the shear strength obtained with the pocket penetrometer was 34 kPa. These results indicate that there has been a significant localized deterioration and reduction in strength.

High water content values given on Figures 12 and 13 may be an indication that the actual shear strength after thawing will be less. The term high water content is a relative term. A water content of about 25 percent for a medium plastic clay with an optimum water content of about 17 percent may be relatively high, but for a high plastic clay with an optimum water content of 23 percent, a water content of 25 percent is not high. However, within the range of the type of clay soil used for the backfill, it is almost certain that regardless of the soil type, that when the water content is larger than 30 percent, the shear strength is likely to be low. Either the water content was initially too high to allow suitable compaction during construction, or there has been deterioration of the backfill since construction, and the water content has increased. As can be seen from Figures 12 and 13, water content values higher than 30 percent were found at several localized areas.

Summary of the Conclusions Reached from the Field and Laboratory Investigations

The field and laboratory results for both the Big Mountain Creek and the Bald Mountain Creek culverts showed that the compaction of the clay backfill constructed in the general vicinity of the culverts was adequate at the time of construction, and there did not appear to be any significant deterioration four or five years later. The backfill in the general vicinity of the culvert was far enough away so that the presence of the culvert did not affect the compaction or cause deterioration of the backfill. The only location where there was some indication of deterioration was within the upper 2.5 m below the road surface and this was due to the normal process of weathering and frost action. In contrast, the results have also shown that the quality of the backfill adjacent to the culvert had deteriorated considerably in comparison to the general backfill. A comparison of the general clay backfill and the culvert clay backfill properties is given in Table 8. The results obtained from the Big Mountain Creek culvert indicated that the density of compacted clay backfill at the end of construction ranged from about 95 to 100 percent of Proctor density. There is a good possibility that the density immediately adjacent to the culvert was less. The density of the general backfill measured after failure indicated

Big Mountain Creek Culvert	General Fill	Culvert Fill**
Density*	9[illegible] - 100%	[illegible] 90%
Shear Strength	200 - 240 kPa	0 - 50 mm [illegible] kPa 150 - 200 mm 20 - 70 kPa 300 - 380 mm 120 - 140 kPa

Bald Mountain Creek Culvert	General Fill	Culvert Fill***
Shear Strength	200 kPa	test holes - Cu 50 - 90 kPa
	200 kPa	test pits - pp 17 - 45 kPa Cu 20 - 29 kPa

* Compacted density as a percentage of Standard Proctor density

** The undrained shear strength for the Big Mountain Creek culvert fill was measured at the distances shown from the culvert.

*** The undrained shear strength for the Bald Mountain Creek culvert was measured in test holes or test pits as indicated. The undrained shear strength measured from the unconfined compression tests and the pocket penetrometer tests is designated as Cu and pp respectively.

***Table 8.** Comparison of the general clay backfill and deteriorated culvert clay backfill for both the Big Mountain Creek and Bald Mountain Creek culverts.*

that the density ranged from about 90 to 100 percent of the standard Proctor density, as given in Table 8. The density of the compacted backfill immediately adjacent to the culvert for two tests ranged from 74 to 89 percent of Proctor density, and this is a substantial decrease. The undrained shear strength of the general clay backfill was 200 kPa or larger, while the strength of the backfill immediately adjacent to the culvert ranged from 50 to 70 kPa, again indicating a substantial decrease.

Similar results were obtained for the Bald Mountain Creek culvert. There are no records available for the conditions applicable at the end of construction. The results obtained five years after construction indicate that the undrained shear strength of the general clay backfill was adequate with a value of about 200 kPa. There was marked reduction, however, for the backfill near the culvert where the shear strength values ranged from 50 to 100 kPa, and values as low as 25 kPa were observed (Table 8).

In a general sense it appeared that a larger portion of the backfill in the immediate vicinity of the Big Mountain Creek culvert walls had undergone a greater reduction in strength and stiffness, compared to the Bald Mountain Creek backfill. This general observation should not be surprising since the Big Mountain Creek culvert failed while the Bald Mountain Creek culvert did not fail. The cause for this is not known but there can be several factors at work.

Firstly, although the backfill for both culverts was intended to be constructed to the same quality, it is possible the construction for the Bald Mountain Creek culvert resulted in better compacted backfill adjacent to the culvert. Secondly, as discussed later, the side pressure on the backfill for the Big Mountain Creek horizontal elliptical culvert is larger than for a circular, or a vertical elliptical culvert, and as result, the Big Mountain Creek culvert is more sensitive to deterioration of the backfill quality. Thirdly, the exposure conditions for the Big Mountain Creek culvert were more severe. Because the thickness of fill on the crown of the culvert was only 2.2 m it is likely that the backfill over the crown of the culvert was frozen the full depth since freezing would occur from the road surface and also from the inside of the culvert. The depth of backfill was larger over the crown of the Bald Mountain Creek culvert.

Deformation of the Culvert

Allowable Culvert Deformation

The maximum deformation that circular culverts can sustain prior to collapse has been determined experimentally to be about 20 percent of the diameter of the culvert. The normal design procedure used by the culvert manufacturer at the time that the two culverts were designed was to use a factor of safety of four to limit the maximum allowable deformation of circular culverts to 5 percent of the diameter. For the Big Mountain Creek culvert, the maximum allowable 5 percent deformation is 380 mm for an equivalent circular culvert with a diameter of 7.6 m.

A few years after the Big Mountain Creek culvert was installed it was recognized by culvert manufacturers that the 5 percent deformation criteria was not valid for horizontal elliptical culverts, and also that the deformation should be calculated on the basis of the minor axis. The culvert manufacturer recognized this design limitation and, at some time before the culvert had failed, their design method was modified. The culvert manufacturer arbitrarily limited the allowable deformation to 1.5 percent of the minor axis, which, for the Big Mountain Creek culvert, is 92 mm.

Estimated Culvert Deformation From Compacted Backfill Properties

The principal design approach adopted by the culvert manufacturer for the Big Mountain Creek and the Bald Mountain Creek culverts was largely a semi-empirical but a recognized method at the time that the culverts were constructed. The culvert manufacturer used this method to design the culverts and to estimate deformations. The critical backfill property for the empirical design approach used by the culvert manufacturer was the modulus of subgrade reaction which is related to the modulus of elasticity of the soil in a manner as suggested by Meyerhof and Baikie (1963). The modulus of elasticity assumed in the culvert manufacturer's design was E = 62 MPa and this resulted in an estimated deformation of 58 mm. The method used for calculating the deformation of a culvert with a shallow soil cover was that proposed by Meyerhof (1966).

As described previously, laboratory tests were undertaken on samples of the Big Mountain Creek culvert clay backfill to determine values of the modulus of elasticity, and these results are given in Table 4. The estimated culvert deformation was recalculated, using the manufacturers design

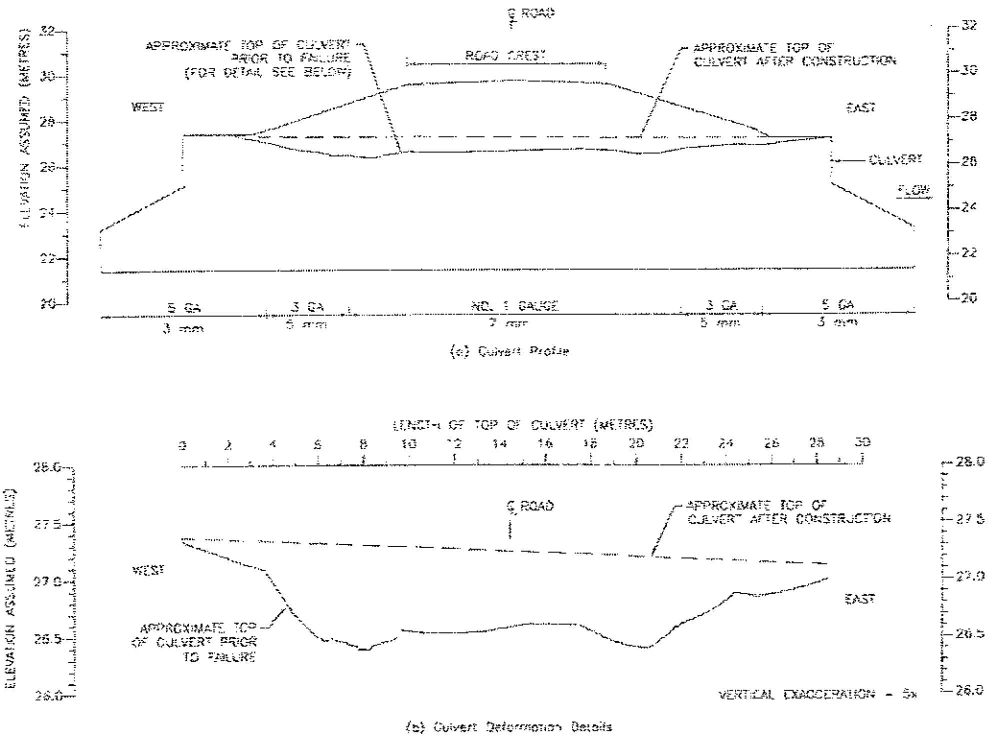


Figure 14. Big Mountain Creek culvert - Deformation of top of culvert prior to failure.

approach, but for values of the modulus of elasticity and the modulus of deformation given in Table 9.

For the cyclic live load caused by the trucks travelling over the culvert and for E_c = 170 MPa, the estimated deformation was 4 mm for the general backfill that would have been present at the end of construction (Table 9). However, for the deteriorated backfill adjacent to the culvert at the time of culvert failure, E_c = 35 MPa, and the corresponding estimated deformation was 22 mm (Table 9).

The estimated deformation of the culvert caused by the weight of the culvert backfill on the culvert at the end of construction, based on the measured secant modulus of 35 MPa was 82 mm (Table 9). The deformation of 82 mm is 1.3 percent of the minor axis of the culvert, and is less than the allowable value of 1.5 percent. The 82 mm deformation is in the same range as the 58 mm deformation estimated using the manufacturers design.

The estimated long term deformation based on the long term modulus of deformation of 5.3 MPa for the general backfill is about 500 mm, which repre-

Table 9. Big Mountain Creek culvert - Estimated deformation of top of culvert for suggested design values for moduli of elasticity and deformation of the clay backfill.

Fill Condition	Cyclic Live Loading		Backfill Load on Culvert: End of Construction		Backfill Load on Culvert: Long Term Loading Modulus	
	E_c* MPa	Δ* mm	Es* MPa	Δ* mm	Dm MPa	Δ* mm
General Fill	170	4	35	82**	5.3	500
Culvert Fill	35	22	35	82**	3.4	1000

* Ec Modulus of elasticity for cyclic loading
Es Secant modulus of elasticity
Dm Modulus of deformation
Δ Deformation
** For this case the culvert manufacturer estimated a deformation of 58 mm for an Es = 35 MPa

sents 8.2 percent of the minor axis of the culvert. The estimated long term deformation based on the long term modulus of deformation of 2.4 MPa is about 1,100 mm, which is 18 percent of the minor axis of the culvert.

It must be recognized that the use of a semi-empirical elastic analysis to estimate the culvert deformation can give relatively reasonable values for small deformations providing that realistic values of E are used. The use of the same analysis that results in estimating large deformations of 500 and 1100 mm is not valid, and the estimated deformations are not meaningful except as an indication that large deformations will occur. The estimated large deformations do serve the purpose of demonstrating that the allowable culvert deformation of 1.5 percent (92 mm) will be exceeded because of the deterioration of the clay backfill.

It should be recognized that the estimated deformations are based on the assumption that the modulus of elasticity selected is applicable for the full extent of the zone of backfill stressed by the culvert. The lower values of the modulus are applicable for the backfill near the culvert, and since the stressed zone of backfill further from the culvert is not as weak, the corresponding modulus will be larger. Consequently, deformations estimated on the basis of the lower modulus for the backfill zone immediately adjacent to the culvert will likely overestimate the actual deformation.

Actual Measured Culvert Deformation

The deformation of the Big Mountain Creek culvert was measured for four consecutive days before the culvert collapsed, with the last set of observations taken on the same day the culvert collapsed. The measurements were made inside the culvert by surveyors using a boat and the elevation of the inside of the crown of the culvert was measured. The observations were not taken exactly at the crown but instead were made on the sets of the bolts along the seams located close to the crown. Since the crown of the culvert had become almost flat just before failure, as indicated on the photographs in Figures 3 and 4, the error in assuming that the bolts located near the crown were at the same elevation as the crown is small relative to the total deformation that was observed.

The deformation of the crown of the Big Mountain Creek culvert is illustrated on Figure 14. The culvert deformation was measured inside the culvert at the crown at 34 locations. Figure 14a shows the approximate profile of the culvert at the end of construction and the profile of the deformed culvert just before failure. The deformation profile of the culvert is shown to more detail on Figure 14b. As can be seen, the maximum deformation occurred approximately at the mid-point of the road side slope, and the deformation was larger at the upstream end of the culvert. The maximum deformation also corresponds closely to the location where the gauge of the culvert structural multi-plates changes from No. 1 Gauge (7 mm thick) to a thinner No. 3 Gauge (6.2 mm thick).

More specifically, the maximum deformation occurs about 1 to 2 metres just before the transition from the heavier No. 1 Gauge multi-plate sections to the slightly lighter No. 3 Gauge sections, and this occurs at a location where the soil cover on the culvert is less. The increased deformation near the ends of the culvert where the soil cover is less may be an indication of the "end or face effects" as noted by Byrne (1990). Near the ends of culverts, the confining soil restraint is low but the thrust can still be large, and these circumstances may lead to buckling near the end of the culvert.

The measured deformation of the top of the culvert just before failure as shown on Figure 14 ranges from a maximum of 860 mm near the upstream end, to about 650 mm along the middle section of the culvert, and then to another maximum of about 750 mm near the downstream end. The deformations of 650 mm and 850 mm are 10.7 and 13.9 percent respectively of the 6.1 m height of the culvert. These deformations far exceed the original estimated deformation of 58 mm, and also exceed the allowable deformation.

The deformation estimated for the heavy gauge multi-plates in the middle section of the culvert using the original design method, but using the long term loading modulus corresponding to the weak clay adjacent to the culvert, was 1,100 mm (Table 9). As stated previously, the elastic method of analysis for estimating the deformation is not valid for such large deformations except as an indication that the deformation will be large. In fact, the actual measured deformations of 650 to 850 mm are large, and are similar to the estimated values.

After the failure of the Big Mountain Creek culvert, surveyed observations of the possible deformation inside the Bald Mountain Creek culvert were undertaken. Observations were taken in September of the year that the Big Mountain Creek culvert failed, and again in March of the following spring. The results of the observations were not conclusive but the results did not appear to indicate that large deformations were occurring during that period. The backfill would have been frozen in March so that it should be expected that most of the deformation would occur following spring thaw when the backfill is probably at its weakest. The owner, however, made the decision in the spring that remedial measures, which included removing the clay backfill, would be undertaken during the same spring. As result no further deformation observations were taken.

Possible Cause of the Culvert Failure and the Method of Failure

Method of Failure

The manner in which a culvert fails by deformation has been described clearly by Spangler and Handy (1973). The deformation failure description is given below and is illustrated on Figure 15.

A flexible culvert will deform under the vertical backfill load, with the vertical diameter decreasing and the horizontal diameter increasing. The outward movement of the sides of the culvert against the backfill develops the passive resistance of the backfill soil which acts horizontally against the culvert. The horizontal passive backfill resistance keeps the actual deformation of the culvert considerably below the amount the culvert would deform if loaded by the backfill, but without lateral support.

The culvert deformation described above continues as the backfill is increased until the top of the culvert becomes approximately flat. (The vertical deformation of the culvert can also continue if the passive lateral resistance decreases due to deterioration of the backfill, even if the amount of vertical backfill on top of the culvert remains constant). Additional backfill may then cause the curvature of the top portion of the culvert to reverse direction, and the top may become concave upward.

When reverse curvature occurs, the sides of the culvert will pull inward since the total circumferential length of the culvert is constant. As the sides deform inwards, the side support of the culvert will be eliminated, since the passive forces cannot follow the inward movement. The deformation of the culvert will proceed as rapidly as the soil backfill can follow the downward movement of the top of the culvert and exert pressure on the top of the culvert. Finally, complete collapse and failure may occur. The entire large deformation change is accompanied by high bending moments in the culvert wall.

The stage at which the vertical deformation is sufficient to cause flattening of the top of culvert and reverse curvature begins may be very unstable, and the subsequent deformation resulting in failure may occur very rapidly. This type of failure is called a "snap-through" failure. The failure of the Big Mountain Creek culvert was probably of this type since failure occurred between about 5:00 p.m. to 10:00 p.m. on the same day that the final set of survey observations were taken.

The information available from the photographs, such as those shown on Figures 3, 4, and 5, taken of the Big Mountain Creek culvert before and after failure, and the measured deformations given on Figure 14 indicated that the culvert probably failed by excessive deformation similar to the manner illustrated on Figure 15. High horizontal pressures were transmitted to the compacted clay backfill along the sides of the culvert as the culvert deformed vertically. Compacted clay backfill material tends to deform with time under these horizontal pressures, and because of the deterioration of the quality of the backfill with time, the amount of deformation increased. This behavior resulted in downward deflection of the crown of the culvert. Ultimately, plastic hinges developed in the steel walls of the culvert due to the excessive deformation and complete collapse occurred. Providing that the deformation is symmetrical about the centreline of the culvert, plastic hinges will form near the 10:00 and the 2:00 o'clock positions on the culvert.

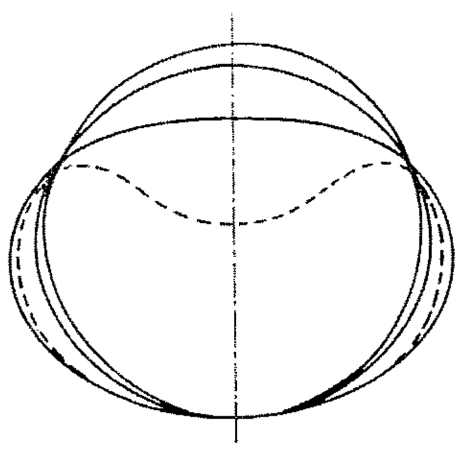


***Figure 15.** Stages during the deformation of a flexible culvert.*

Lateral Soil Bearing Pressures for Elliptical Culverts

Horizontal elliptical culverts develop high soil pressures at the sides, higher than circular culverts, and the side pressures can be significantly higher than the vertical backfill pressure on top of the culvert. Measurements of the soil pressures on a large diameter culvert by Lefebvre et al. (1976) demonstrated that the pressures at the spring line were 2 to 3 times larger than the pressure on the top of the culvert. For the dimensions of the Big Mountain Creek culvert, it was estimated that the side pressure at the spring line due to the dead load of the backfill was 200 kPa, and the side pressure due to the dead load plus the live load was 240 kPa.

The bearing capacity factor for the clay soil at a depth corresponding to the spring line is approximately 7. When the applied soil pressure just balances the ultimate bearing capacity, the factor of safety against bearing failure is 1.0. For a factor of safety of 1.0, the corresponding undrained shear strength for the applied bearing pressure of dead load, and the combination of dead load plus live load, is 200/7 = 29 kPa and 240/7 = 34 kPa respectively. This means if the soil shear strength is 29 or 34 kPa, bearing capacity failure will occur, and the culvert side walls will deform readily outward because of the poor soil resistance.

In fact, even before the shear strength decreases towards the above low values, deformation of the culvert will increase as the soil yields even though bearing failure does not occur. It has been shown previously that the undrained strength of the clay backfill was as low as 25 kPa, and strength values in the range of 50 kPa occurred in localized areas. Clay backfill with these low strength values would have caused lateral yielding and this provides further confirmation of the culvert failure by excessive deformation as a result of the deterioration of the clay backfill.

Possible Cause of Culvert Failure

It is believed that the cause of the Big Mountain Creek culvert failure was due to the use of a clay backfill around the culvert which deteriorated with time. It is also believed that if a compacted granular backfill was used, failure probably would not have occurred. A possible explanation of the manner in which the clay backfill deteriorated is described in the following paragraphs.

The general clay backfill that was initially compacted with heavy construction equipment had sufficient strength and stiffness to adequately support the culvert. The clay backfill immediately adjacent to the culvert wall was probably not compacted as well, but the compaction was initially adequate since the culvert functioned adequately for four years. While theoretically it is possible to compact the clay backfill immediately adjacent to the culvert as well as the general fill, it is not practical to achieve the same density under normal construction practice in an economically acceptable manner. The cost of achieving the same degree of compaction adjacent to the culvert as the general backfill could exceed the cost of an alternative solution such as using granular material.

Even if the clay backfill was adequately compacted adjacent to the culvert, weakening and deterioration would probably have still occurred for the Big Mountain Creek culvert, and failure may still have occurred, but would have taken longer.

The clay used for the backfill is heavily over-consolidated insitu and can be very dense. One of the consequences of such overconsolidated clay is after the clay is pulverized as part of the process of undertaking a laboratory compaction test, the amount of compactive effort used during a standard Proctor compaction test may not be adequate to obtain a density as large as the insitu density of the naturally occurring material. This is not necessarily a major concern, provided that the strength and the compressibility of the compacted clay are adequate for the proposed engineering application.

One of the concerns with the clay backfill has been described previously for the Big Mountain Creek culvert. Test pits excavated in the general clay backfill revealed that the compacted clay backfill had a noticeably blocky structure. It was evident that the compaction process had not completely pulverized the very dense lumps of clay obtained from the clay borrow area. The consequence of the blocky structure is that the compacted clay backfill is not homogenous and is composed of a matrix of hard clay lumps with less compacted clay soil between the hard lumps, and may even contain some voids. The blocky clay structure is not unusual for stiff heavily overconsolidated clay soil.

The problem that can arise is that the compacted clay backfill with a blocky structure may not be detected during the normal field density testing, and that the nature of this type of backfill can change with time. This condition may not be detected during normal field density testing because the average density of the matrix of very dense hard lumps and the less compacted clay between the lumps can still be comparable to the maximum density obtained in the laboratory on the pulverized soil, resulting in the conclusion that the backfill is well compacted.

The conclusion that the compacted backfill with a blocky structure is adequately compacted was initially justified since the compacted backfill with a matrix of dense clay lumps can have adequate strength and low compressibility. The problem that can arise is that the strong matrix of dense clay lumps can deteriorate with time under certain circumstances. Furthermore, if the general backfill which was compacted with normal heavy compaction equipment has a blocky structure, it should be expected that the backfill immediately adjacent to the culvert will likely have even a more blocky structure because of the greater difficulty of compaction near the culvert.

The deterioration of the compacted clay backfill with a blocky structure can occur for several reasons. The matrix of the dense clay lumps will tend to yield with time towards pockets of the less compact clay when subjected to load, either from self-weight of the soil or to due to applied pressure. If the less compacted clay has a higher water content than the dense clay lumps, the overconsolidated clay lumps will tend to absorb some of the water, and become softer and yield. The above yielding process is accelerated and enhanced if the compacted soil is subjected to freeze-thaw cycles. If free water seeps into the compacted clay backfill with a blocky structure, the additional water combined with exposure to freeze-thaw cycles, will increase the degree and rate of softening significantly.

Both the Big Mountain Creek and the Bald Mountain Creek culverts were subjected to high floods after construction with the culverts running full of water. Water ponded upstream of the access roads during the floods and almost over-topped the roads. As a result, water was probably forced through the culvert seams into the clay backfill behind the culvert walls, and into the backfill above the crown of the culvert. The water filled any voids between the culvert walls and the backfill, as well as voids in the backfill. Much of this water remained in the backfill after the flood waters receded.

The excess water in the backfill would cause softening of the clay and closing-up of voids. The real damage would occur during the subsequent freeze-thaw cycles as a result of the frost penetration from inside the culvert. Because of the thin soil cover over the crown of the Big Mountain Creek culvert, freezing would occur from both the inside of the culvert and from the top of the road surface. The net result of the freeze-thaw cycles is that much of the backfill adjacent to the culvert walls which initially supported the culvert walls adequately became weak and yielded.

Sufficient evidence has been provided in this article which demonstrates that the compacted clay backfill adjacent to the culvert had weakened considerably and the water content had increased significantly. Laboratory tests also demonstrated that the compacted clay stiffness property is time dependent, and with sufficient time and the appropriate circumstances, the stiffness can be reduced very significantly. Eventually, the backfill yielded sufficiently, and the Big Mountain Creek culvert failed.

It is believed that the problem of the backfill yielding would not have occurred if the backfill adjacent to the culvert consisted of a well compacted free-draining granular material. Placement and compaction of granular backfill immediately adjacent to the culvert walls would have been easier to achieve than placing and compacting a clay soil. The properties of a well compacted granular backfill will not change much when saturated with water, unlike a clay backfill. Furthermore, free-draining granular backfill within the zone of frost penetration will not deteriorate much with time, or during exposure to freeze-thaw cycles. Although the granular material can become saturated during flooding, the water will drain so that loosening due to water expansion upon freezing will not likely be problem. Granular material with fines should not be used since ice lenses and heave could develop.

The clay backfill for the Bald Mountain Creek culvert was replaced 20 years ago with granular backfill, together with some other remedial measures, and the culvert has performed well since.

Summary and Conclusions

1. Two large diameter multi-plate culverts were constructed in 1972 beneath a forestry haul road using clay backfill.

2. About four years after construction, the Big Mountain Creek culvert deformed severely and

collapsed in a snap-through manner.

3. Deformation of the crown of the culvert was measured just before the collapse of the culvert. The upper range of the measured deformations was between 650 and 850 mm which are 10.7 and 13.9 percent respectively of the 6.1 m height of the culvert. These deformations far exceed the original estimated design deformation of 58 mm, and exceed the allowable deformation.

4. The owner of the forestry road also became concerned about the safety of the Bald Mountain Creek culvert. Although the measured culvert deformations were not sufficient to suggest that the culvert might collapse in the near future, the owner decided to undertake remedial measures since the forestry haul road crossing the culvert was critical to the owner's operations. The remedial measures consisted of excavating the clay backfill and replacing with a compacted granular backfill, together with some structural strengthening of the top of the culvert. The culvert has performed well for the past 20 years.

5. The failure of the Big Mountain Creek culvert is believed to be caused by the weakening and softening of the clay backfill with time. Data obtained from the clay backfill of both culverts indicated that a significant deterioration of the clay backfill had occurred subsequent to construction. Unfortunately, information on the actual condition of the upper backfill for the failed Big Mountain Creek culvert could not be obtained since the backfill was washed away after the failure of the culvert.

6. The deterioration of the clay backfill is believed to be caused by a combination of several factors. The compacted clay backfill was found to have blocky structure, with a matrix of hard dense lumps of clay, surrounded by soft clay or possibly even voids between the hard lumps. The combination of water seeping through the culvert seams during spring flood conditions and saturating the backfill behind the culvert, together with freezing of the saturated clay backfill during the winter seasons resulted in severe deterioration of the compacted clay, causing sufficient yielding to result in the culvert collapse.

7. The amount and the rate of deterioration of a clay backfill will depend on the circumstances applicable at each culvert location. In some cases the clay backfill may perform adequately for a long time while in other cases deterioration may be quite rapid. Clay material is inherently more likely to change with time when exposed to deteriorating conditions compared to granular material for the same conditions.

8. It is believed that if a well compacted free-draining granular backfill was used as backfill around the Big Mountain Creek culvert, failure would not likely have occurred.

Acknowledgements
The investigations summarized in this article involve several people and organizations, and presentation of this article would not be possible without their assistance and cooperation.

The author would like to acknowledge and thank Mr. Bruce Schneider, Woodlands Manager, Weyerhaeuser Canada Ltd. for permission to publish this paper on their culverts. The author would also like to thank Mr. Paul Leroux of Weyerhaeuser Canada Ltd. for his assistance in reviewing the company's records, and for providing some of the photographs. The author would like to thank Mr. Peter Hodgson, formerly of Procter and Gamble who was the company's representative during the original culvert construction and remedial measures, and who was helpful recently in providing background information.

The author would also like to thank Agra Earth and Environmental, the authors' former employer, and specifically Dr. E.C.McRoberts, P. Eng., for permission to publish this paper. The author would like to thank and acknowledge Mr. N.S. Luxford in particular, also a former employee of Agra Earth and Environmental who undertook the original investigation of the Big Mountain Creek culvert failure. The author undertook some additional investigations and takes responsibility for that work.

Finally the author would like to thank his current employer, SRK-Robinson Inc., and in particular Mr. Mike Eivemark, P. Eng. for support during the preparation of this paper.

REFERENCES

Byrne, P.M. 1990. Design Procedures For Buried Flexible Metal Structures. Report to the Ministry of Transportation and Highways, Province of British Columbia.

Byrne, P.M., Srithar, T., and Kern, C.B. 1990. Field Measurements and Analysis of a Large Diameter Flexible Culvert. Conference on Flexible Pipes, Columbus, Ohio, Center for Geotechnical and Groundwater Research, Ohio Department of Transport,

Lefebvre, G., Laliberte, M., Lefebvre, L., Lafleur, J., and Fisher,C.L. 1976. Measurement of Soil Arching Above a Large Diameter Flexible Culvert. Canadian Geotechnical Journal, Vol. 13, p. 58-71.

Meyerhof, G.G. 1966. Composite Design of Shallow Buried Steel Structures. Canadian Good Roads Association, Halifax.

Meyerhof, G.G., and Baikie, L.D. 1963. Strength of Steel Culvert Sheets Bearing Against Compacted Sand Backfill. Proc. H.R.B.

Spangler, M.G., and Handy, R.L. 1973. Soil Engineering, Third Edition. Harper & Row.