Culvert Inlet Failures—A Case History

ROY C. EDGERTON, Research Engineer, Oregon State Highway Department

Bent-up ends have been experienced on three large structural plate culverts installed with the upstream ends square and projecting to the fill toe.

The paper describes the installations and failures and presents one explanation of the cause.

● INLET FAILURES on three structural plate culverts on new construction on the Oregon Coast Highway in Curry County, Oregon, occurred in January 1959. The inlet ends were bent up, apparently by the buoyant force resulting from the difference in water surface inside and outside the culverts.

Figure 1 shows the Burnt Hill Creek installation in plan and section. The two 96-in. structural plate culverts vary from 1 gage at the center to 10 gage at the ends with 1-gage inverts throughout. The length is 612 ft. The upstream ends are square and extend to the fill toe; the entrances are thus of the projecting type. The culverts outlet into a concrete flume. The culverts were staked according to the plan which called for a 1.2 percent over-all slope with 1-ft camber at the center. From profiles taken in the undisturbed portions of the culverts after failure, it seems probable that at the time of failure the slopes of the inlet sections were steeper than the over-all design slope and supercritical.

The fill slope was 2:1 normal to the centerline. Since the upstream leg of the culverts was skewed about 30° with the normal, the actual slope in the direction of the culverts was about 2.3:1. The culvert inverts were placed above the channel bottom and the channel upstream was raised to invert elevation by a random fill. The resulting approach channel was approximately level for about 300 ft upstream from the culverts.

The Whiskey Creek installation is a single 90-in. structural plate culvert. Again, the gage varies from 1 to 10 with 1-gage invert. The culvert is straight, 411 ft long, and has a concrete outlet flume. The culvert grade situation is similar to that at Burnt Hill Creek except that there is no question that the entrance section was on a supercritical slope at the time of failure. The fill slope is 2:1 and the culvert is so nearly normal that the same slope applies to the line of the culvert. A level random fill extends about 400 ft upstream from the culvert entrance.

On the morning of January 9, following an intense storm, the situation shown in Figure 2 was found at the Burnt Hill Creek site. Similar conditions were found at Whiskey Creek. In all cases, the bends were smooth, well formed elbows with straight, undistorted sections of pipe extending upward from the bends.

The Whiskey Creek culvert and the south culvert at Burnt Hill Creek were blown off at the bend with ring charges to allow the ponds to drain. Figure 3 shows the Burnt Hill Creek installation after the blast. Although the blasts destroyed or distorted most of the bend area of two culverts, there was an opportunity to examine the north culvert at Burnt Hill Creek before and after disassembly. The corrugations in the 1-gage invert plates that formed the outside of the bend were extended quite uniformly from the original 6 in. to about 6.6 in. The corrugations in the 10-gage plates that formed the inside of the bend vary from 1 to 3 in., apparently depending on the amount and nature of fill material trapped in the folds. The curvature was not uniform, being sharpest at the center of the bend. There was about 25 ft of straight, undistorted pipe extending upward from the bend. The bends in the Burnt Hill Creek culverts occupied

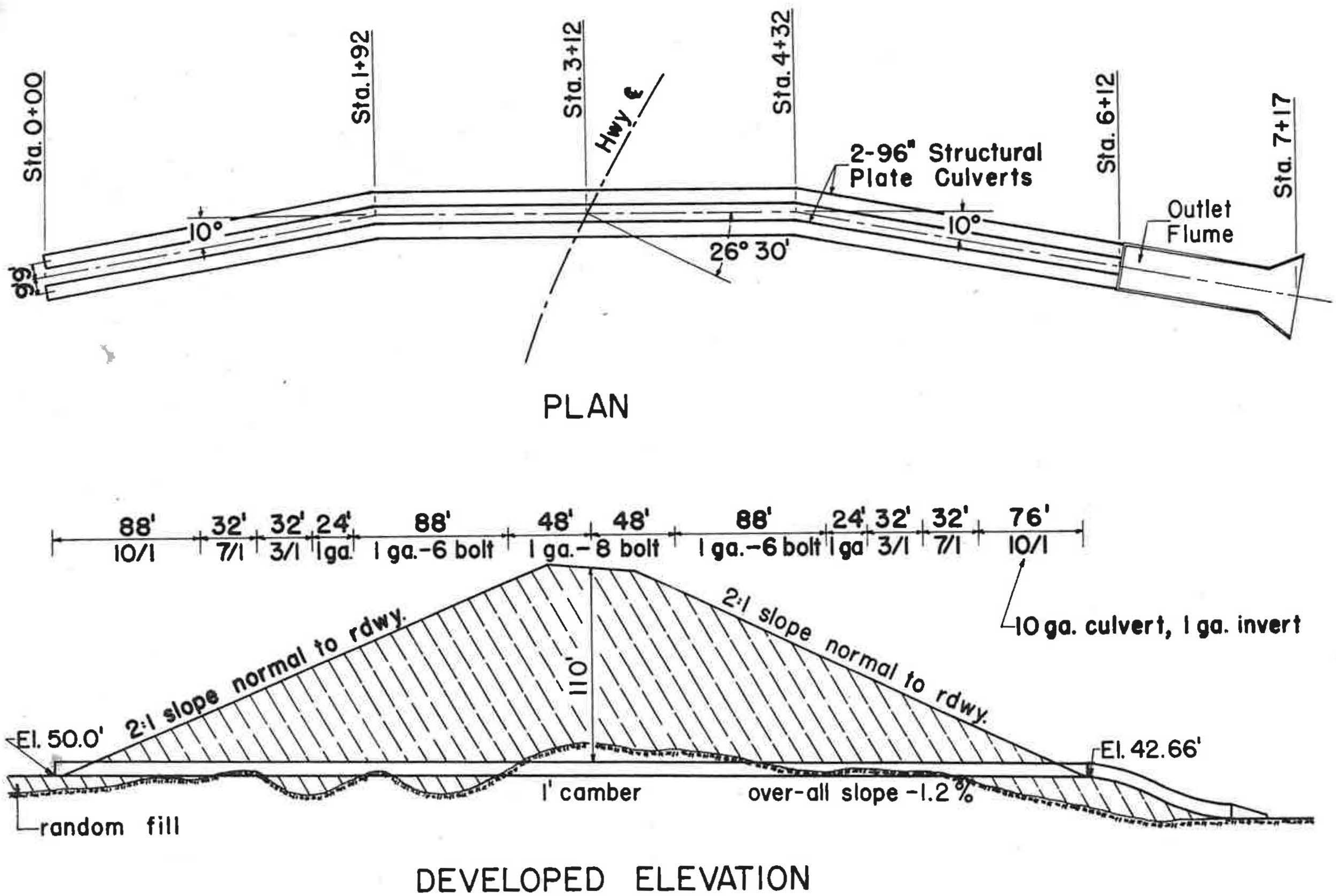

Figure 1. Burnt Hill Creek installation.

some 18 ft of pipe. The distances from the centers of the bends to the original upstream ends were about 44 ft for the south culvert and 34 ft for the north culvert.

There were no slides in either area. Other than minor fill settlement, the only displacement of fill material was that lifted out by the raising culverts. The amount of debris in the area could not have materially affected culvert operation. The random fill channel approaches eliminated any bed load problem at the culvert entrances.

The known facts point to buoyancy as the cause of the failures. The verification of this would require that the moment applied to a culvert by external forces be compared with the resisting moment of the culvert. This has not been possible since no data on the longitudinal bending properties of structural plate culverts have been found. External moment calculations were made for a 96-in. culvert of the type used at Burnt Hill Creek. A headwater depth of 96 percent of the diameter was used as preliminary calculations indicated that this produced the highest moment of any unsubmerged depth. The assumptions for the moment calculations are shown in Figure 4. A discharge of 340 cfs was used.

Moments were calculated for fill slopes from 0.5:1 to 2.5:1. The magnitudes and locations of the maximum moments are listed in Table 1. Within this range, the maximum moment more than doubles for each half unit flattening of the slope. This might explain how large structural plate culverts with projecting ends have been used successfully with steep fill slopes. The locations of the maximum moments can be considered to agree with the locations of the bends at Burnt Hill within the accuracy of the basic assumptions.

The culverts were replaced as originally installed except that the fill slopes in the vicinity of the entrances were steepened to about 0.5:1 with a facing of large rock riprap.

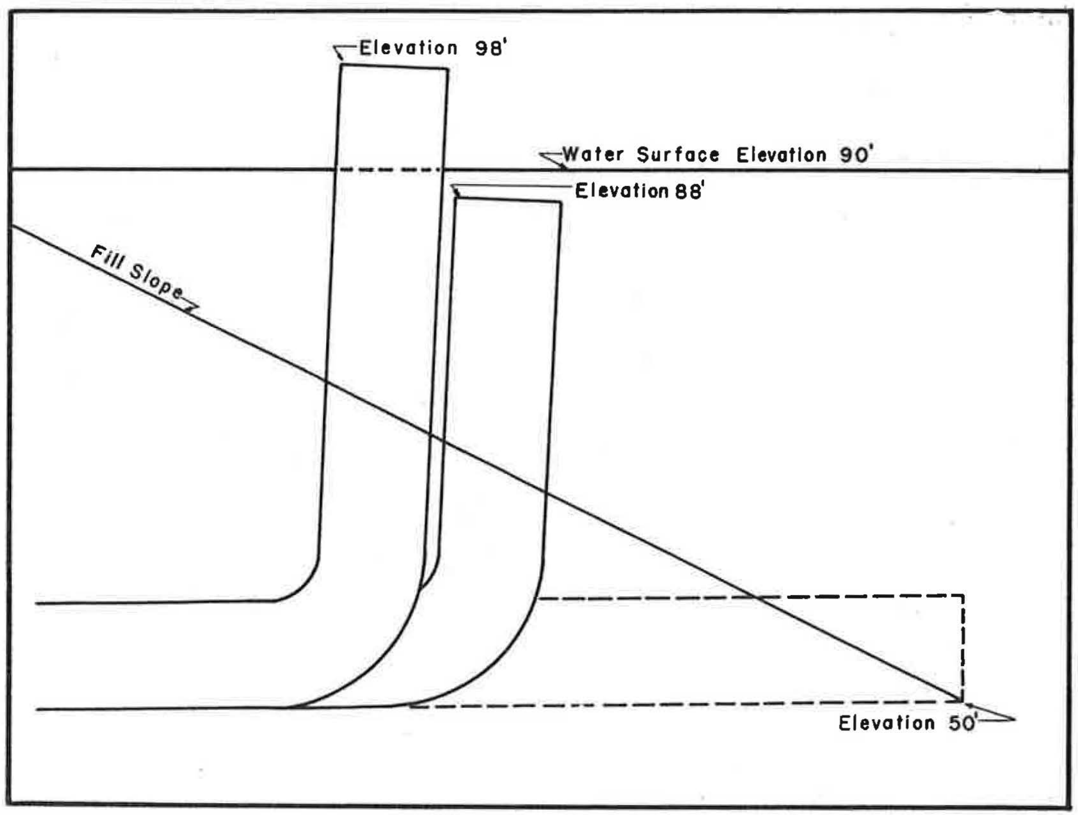

Figure 2. Schematic sketch of Burnt Hill culverts failure.

Figure 3. Burnt Hill Creek inlets.

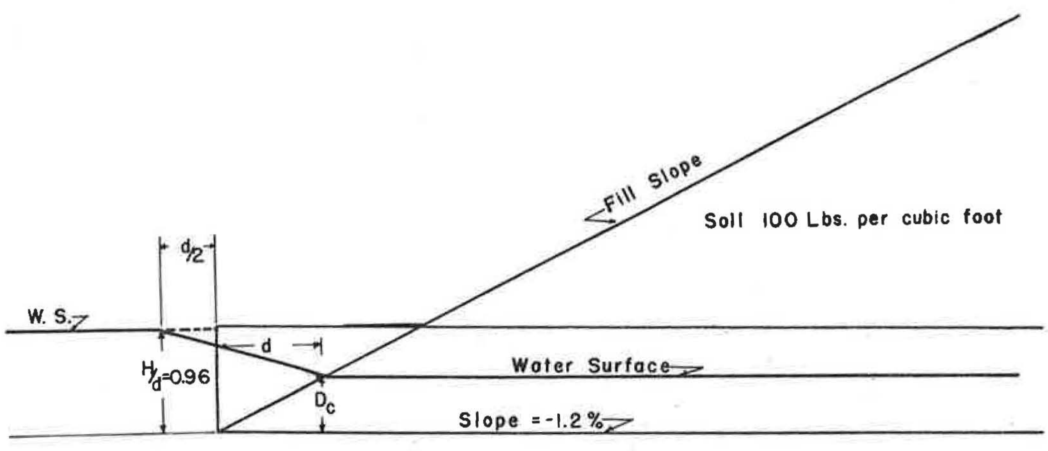

Figure 4. Assumed entrance conditions.

TABLE 1

RELATION OF BUOYANT MOMENT TO FILL SLOPE

Fill Slope	Maximum Moment (ft-lb)	Location (ft from entrance)
0.5:1	3,200	6.3
1:1	25,300	12.5
1.5:1	81,800	19.5
2:1	172,500	26.5
2.5:1	398,000	40.1

There are two general approaches to the prevention of future failures of this kind. One of these is to determine the longitudinal bending properties of the culvert and design within safe limits. The other is to adopt designs that avoid unbalanced uplift moment.

The first approach would require a series of full-scale tests of culverts for bending moment. The second would involve the employment of culvert end treatments now commonly used such as beveled or step-beveled ends or concrete head walls. Concrete head walls offer an advantageous solution in that they eliminate pipe projection, furnish weight to the pipe end, and improve the entrance condition.

In conclusion, the culverts are considered to have failed by buoyancy. Future failures of this kind can be avoided by determining the longitudinal bending properties of culverts and designing within these limits or by avoiding unbalanced uplift moments.

Discussion

R. ROBINSON ROWE, Principal Bridge Engineer, California Division of Highways, Sacramento, Calif.—Edgerton's contribution is noteworthy because of the dearth of reliable data on failures, particularly of small structures which are so frequently replaced or covered up quickly to salve the reputation of the designer of the reliability of the material. When faithfully reported, failures are better teachers than successes and the lessons are more eloquent.

The writer will present brief accounts of other failures due to buoyancy of culverts, comment on the determinacy of uplift and ballast, and express his views on economy of alternative safeguards.

The most common failure due to buoyancy in California has been due to installation of small metal pipes in pervious embankment with shallow overfill, particularly in low-standard highways and detours during construction. A typical case is a 24-in. pipe installed in a pervious gravel embankment covered by a 6-in. overfill and a 3-in. oil-cake slab. Water rises to the culvert crown, saturating the gravel but flowing only 18 in. deep in the pipe. Per foot of pipe, uplift is 200 lb, but surcharge is 340 (20 for pipe, 140 for water, 120 for fill and 60 for slab). If drift reaches and clogs the culvert reducing the surcharge to 200, just balancing the uplift, failure is imminent—the proximate cause being one of the following:

1. The uplift is greater at the downstream (lower) end of the pipe, which floats first.
2. Ponding behind blocked pipe increases uplift at upstream end of the pipe, which floats first.
3. Shear resistance of slab prevents flotation of pipe, but ponding overtops road, floats the slab, and the pipe rises throughout.

All three types of failure have been observed. No doubt many complete washouts followed this pattern, judging from stranded location of pipe and slab, but were not reported as such for lack of conclusive evidence. Elastic bending of pipe may have contributed to failure, but recovered pipes appeared straight except for those swept broadside against a tree or power pole.

Next in frequency is the flotation-flexure of downstream ends of tide-gated metal pipe. Unlike flexure of the upstream end which backs water higher so as to increase uplift, flexure of the downstream end reduces uplift until pressures are balanced. Hence flexure may not produce failure, the deflection being (a) elastic, (b) tolerably plastic, or (c) correctable instead of (d) beyond repair.

An instance of the latter was reported by William A. White who had been Resident Engineer for the Corps of Engineers on the Deer Island Project along the Columbia River in Oregon. A series of CM pipe culverts 24 to 48 in. in diameter carried runoff from the hills under a project dike into a canal. Passing the site in 1942 he observed that all the gated outfall ends had been bent vertical by high water in the canal. He estimated the head could not have exceeded 3 ft from the canal itself, but considered it possible that the river had overflowed the site at a higher stage. There were 6 to 8 projecting above the canal and possibly many others bent below the water and hence not visible.

Flotation is not limited to metal pipes. During the flood of December 1955, an 8- by 8- by 48-ft RC box culvert, built in 1926 at the mouth of Cold Canyon along Merced River, jammed with drift and floated out intact, wingwalls included. The structure weighed 90 tons displaced 132 tons and carried an overfill of 34 tons so that net buoyancy was 8 tons less what little water was still inside. Overlapping the highway increased the uplift. It came to rest 15 ft nearer the river and 5 ft downstream (Fig. 5).

Also, on a number of occasions the downstream end of RC pipe culverts projecting into a large channel below high water have been displaced in a manner strongly suggesting flotation rather than erosion. Runoff from such small tributaries may be far down on the falling stage before the main stream peaks, at which time flow past the projecting pipe entrains water from the pipe to create uplift.

Recognizing situations which might induce uplift failures is important in design, but predicting uplift pressures is quite speculative. In dam design authorities are in wide disagreement. For example, some speculate that intensity of pore pressure varies uniformly from full hydrostatic at the heel to none at the toe and combine that pressure with a coefficient proportioned to the ratio of pore area to base. This latter coefficient has been set at 0.2 to 0.4 by some who then use 0.5 or 0.67 for safety. Others set it as high as 0.9 and allow for lifting of the heel so that full pressure penetrates under the dam for some distance.

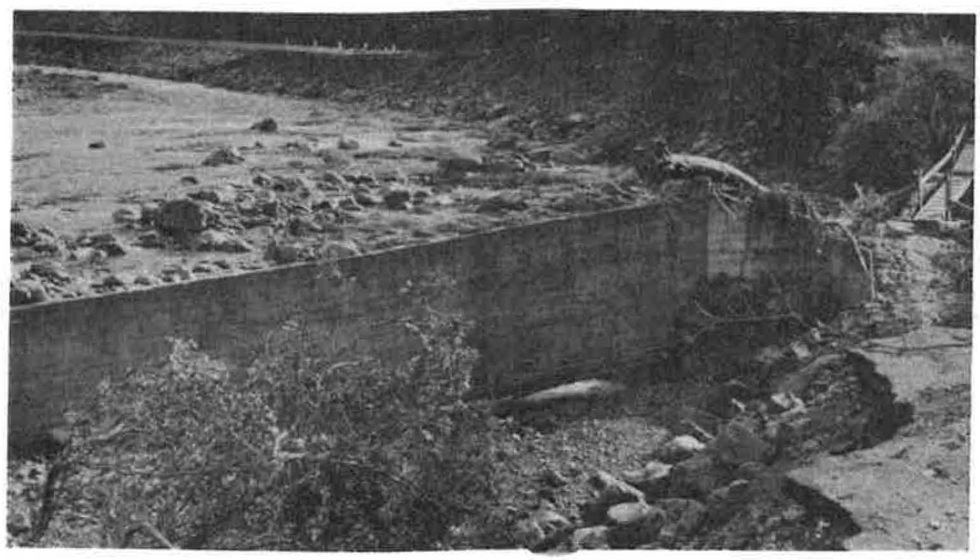

Figure 5. Flotation failure of 8x8x48-ft RC box culvert at mouth of Cold Canyon, Merced River, California.

Pipe bedding is generally much more pervious than dam foundations, so that coefficients should be higher. Elastic deflection of the projecting end will admit a wedge of water at full pressure. Some saturated embankments may become semi-fluid so that uplift is magnified by the specific gravity of the soil-water mixture.

The hydrostatic head is just as speculative. Edgerton has computed a critical value for a clear entrance and rapid percolation. The head may be much greater for culverts blocked by drift at the entrance or by a gate at the outfall. Percolation may be much less for a tight fill or a flashing stream. Deflection of the culvert may be progressive, each increment adding to the head as water is impounded above the entrance.

Just as elusive is the weight of soil over the pipe effective in ballast. At first the pressure is active, but as the pipe begins to rise it becomes passive and much greater, at least until the soil shears to the surface. The weight, passive pressure and shear are sensitive to saturation when stage rises above the pipe; probably for granular fills the unit weight increases but the passive pressure decreases.

Once a pipe has floated, these factors affecting uplift and ballast can be estimated with some confidence, since the combined effect is known. Prediction for a new installation is futile.

In considering the economy of safeguards against flotation, the infrequency of failure must be given great weight. A good general rule might be: use cheap safeguards for ordinary hazards and expensive measures when the risk or consequence of failure is very great.

As an example, interpolating Table 1 for a slope of 2.3:1 the buoyant moment on the Burnt Hill Culvert would have been 280,000 ft-lb, 35 ft from the entrance. This could have been ballasted by 8,000 lb at the entrance or 16,000 lb distributed along the pipe. Even a small headwall or cutoff at the entrance or paving of the pipe invert would have sufficed at a nominal cost.

Comparable moments for larger pipe would be much greater, varying about as the cube of the diameter. Even for a 15-ft pipe, entrance ballast could be provided with wingwalls or other transition amounting to 13 cu yd of concrete at a cost of $1,000. Good hydraulic design ordinarily warrants a much heavier structure.

Summarizing, the failure cited by Edgerton is a spectacular example eloquently reminding engineers that culverts are light structures for which buoyancy cannot be neglected. The hazard is greatest for metal pipes, increasing rapidly with diameter, but concrete structures are not immune. The hazard has been increased by modern standards of milder slopes for highway embankments. Ordinary entrance transition structures will provide assurance of stability. Special ballast may be required for gated outlets and for culverts under low fills subject to overflow; if strategically located such ballast would be relatively light and inexpensive.

B. LE MÉHAUTÉ and J.F. FULTON, Queen's University, Kingston, Ontario— Edgerton has reported a very interesting occurrence which graphically illustrates the damage which can result due to neglecting uplift pressures, which will develop under certain conditions dependent upon how culverts are laid.

From calculations based on the assumed conditions, it is felt that the culverts would not have failed had the fill slope existed as shown in Figure 4. It is obvious that the pressure created by the headpond level acts around the culvert's circumference near its entrance and throughout its length by infiltration into the bank, the seepage line decreasing very slowly downstream almost parallel to the gradient line inside the culvert (Fig. 6). The uplift force is, therefore, equal to the weight of the volume of water which would occupy that volume of the culvert between the outside water level and the inside water level. This difference in levels is greater than, but roughly equal to the velocity head of the flow in the culvert $V^2/2g$. This uplift force, although appreciable, is not sufficient to lift the culvert, a wedge of the embankment ballast, and simultaneously overcome the culvert's resistance to bending properties.

If, on the other hand, the culvert was laid so that its end projected upstream of the embankment heel, or if scour effects around the culvert entrance were extensive enough to cause an embankment slope failure and thereby uncover an appreciable length of

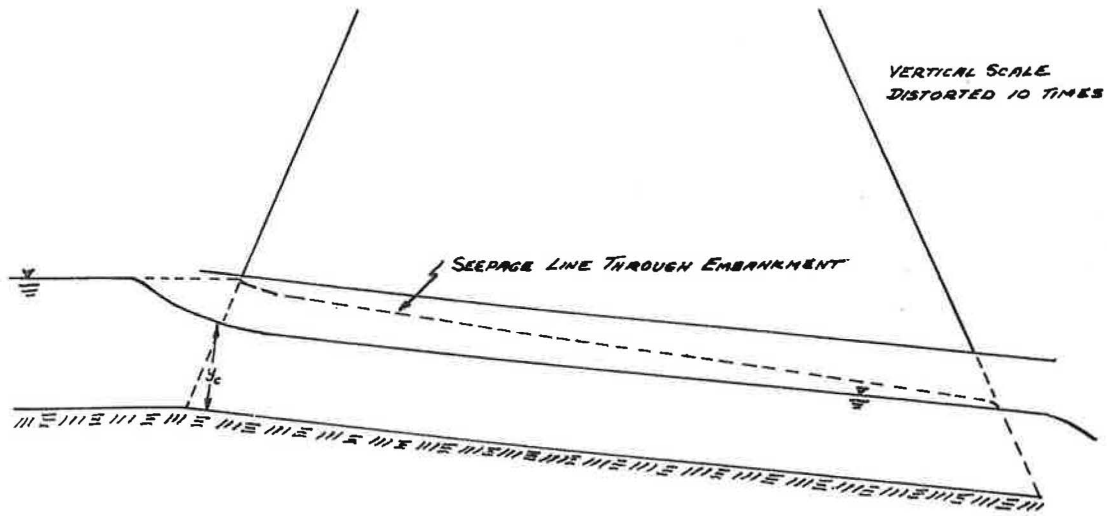

Figure 6.

culvert (at Burnt Hill Creek this length would appear to have been in the order of 25 ft), then the uplift force is sufficient to raise the culvert slightly. Once the pipe has lifted slightly, this phenomenon increases in magnitude. This is due to the fact that the headpond level is forced to rise (in order to discharge into the culvert), while the depth inside the bent section tends to decrease (since the velocity tends to increase with the increased slope); these two effects thereby increasing the difference between the inside and outside water levels.

It is the opinion of the writers that the failure was caused due to the combined facts that the culvert projected too far upstream of the fill slope and that the culvert was laid to too steep a grade.

The grade shown causes the critical depth to develop very close to the entrance, but if it developed farther down the culvert the intervening depths would form a backwater curve thereby reducing the magnitude of the uplift force.

One solution to this uplift problem would be to start the fill slope at the entrance of the culvert. A simple headwall (constructed of any material from hand placed rock to a complete concrete header) should also be included to prevent scour around the culvert entrance and guard against a slope failure of the embankment. A second solution would be to lay the culvert to a much gentler slope. Indeed, if the culvert were laid so that the critical depth developed at the downstream end, the difference between inside and outside water levels near the culvert entrance would be very small. In the case of flood the culvert would flow full (thereby causing buoyancy effects to disappear), the discharge of the culvert depending only on the difference between the upstream and downstream water levels. In installations where this head difference is large, additional factors such as air entrainment, cavitation, and stability pose further difficulties. At Burnt Hill Creek this head difference was only of the order of 40 ft when the culverts were out of operation. It is felt that under normal operation this head difference would not be duplicated, but even if it were these additional factors would not likely create serious problems. If it is found that stability difficulties are encountered, these may easily be corrected by simply pushing down the top of the downstream end of the culvert until the diameter is reduced to 0.9 D. Under flood conditions, the critical depth forming downstream will increase until this bent lip is encountered. The culvert will then fill up from the downstream end and will force any trapped air out the upstream end thereby eliminating the possibility for serious instability to develop. This bent section is shown in Figure 7.

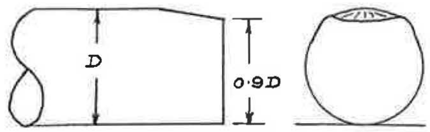

Figure 7.

The writers feel that either one of these solutions, or a combination of them, would provide a more economical answer than fixing the embankment slope at 0.5:1.

ROY C. EDGERTON, Closure—This paper was presented to call attention to the need for consideration of buoyancy in culvert installation and to encourage the release of reports of similar failures. Rowe's discussion furthers the second of these aims, contributing more to the subject than did the original paper.

Most of the points raised by Le Méhauté and Fulton are considered to have been covered in sufficient detail in the paper; however, a few points might warrant amplification.

As with photographs in general, Figure 3 was not intended for quantitative interpretation. The skew of the culvert with respect to the fill slope and the level of water in the pond makes this impossible. Figures 1, 2, and 4 were expected to serve the quantitative function. It should be noted that with a 96-in. culvert and an effective fill slope of 2.3:1 the slope line would intersect the top of the culvert about 18 ft from the end of the culvert.

The method of calculation of the water surface inside the culvert incorporated an entrance head loss of about $0.7\ Vc^2/2g$. It was assumed that by the time of failure, the full upstream head would be effective on the outside of the culvert to the area of the bend.

In the mind of the author there is not "too steep a grade" for a culvert except as it affects culvert wear and the control of outlet velocity. In general, culverts are placed where and as required. There is no particular interest in solutions that materially reduce culvert capacity.

The description of the restoration seems to have been misread. The culverts were replaced as originally installed. Large rock riprap was then placed around the culvert entrances. The slope of the riprap face was about 0.5:1. Since then a combination of step-bevel culvert ends and riprap slope facing have been adopted as standard for future construction.