lieve a design is safe if the bearing strength of the subgrade, p_o, is selected at 0 10 to 0.20 in and the bearing strengths of base and subbase materials on such basis that the total deflection of the whole pavement does not exceed Mr. Hubbard's figure The laboratory test on finite depths of soil for the base courses should supply the data

It is necessary in all bearing tests either to correct the load-deflection data for the effect of seating the plates and consolidation of the soil or use some method of repetitive loading suggested by Hubbard in Asphalt Institute Research Bulletin No. 9, or use the yield-value method suggested by M. Housel in the 1942 *Proceedings*, Association of Asphalt Paving Technologists.

These many thoughts have been discussed to emphasize that the approach to bearing tests should not be hampered by too many refinements. It will not affect the ultimate design a great deal to call a 20-psi. bearing value subgrade an 18 or 22-psi. subgrade. When the amount of deflection to be tolerated in the pavement has been decided upon, the bearing value measured will not be out of line by any careful technique as much as the variations to be encountered in soil conditions and construction in the field.

One last thought· bearing strength determined by other methods than bearing tests may be used in their proper place in the formulae for design. If the bearing value of a soil, determined by some other test such as the triaxial shear test, can be evaluated in terms of the size bearing area and load required in the design, such values may be taken for p_o, p'_o and p_s without changing the ultimate answer a great deal.

CULVERT DESIGN IN CALIFORNIA[1]

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SYNOPSIS

After a 20-year period of comparatively moderate storms in California, the heavy runoff of 1937–38 caused such expensive damage to highways throughout the state that the Division of Highways of California, Department of Public Works, authorized an exhaustive departmental study of the performance of culverts and survey of culvert practice.

It was found that conduit sizes were usually determined directly from formulae, such as Talbot's depending only on drainage area and a runoff factor, and that the runoff factor was modified to fit local conditions as determined by experience Consequently, there was a tendency toward under design after a period of drought and toward over design after a period of floods It was also evident that headwalls, endwalls, and other appurtenances had not been designed from any rational application of hydraulics

From this need there was developed a new concept for design of culvert conduits, combining two principles; (one) design to pass a 10-year flood without static head on crown of conduit at the entrance, (two) balanced design of barrel and appurtenances to pass a 100-year flood without serious damage to the highway

These two principles are applied successively for each problem For the typical case with free outfall, the outlet is far from full when the entrance is just full. If gradient of flow line exceeds a neutral slope (say 0 8 per cent for smooth and 3

[1] The conclusions and recommendations expressed are those of the writers and do not necessarily reflect policies of the California Department of Highways, Division of Highways

per cent for rough bores), the water surface drops sharply just inside the entrance; increasing the slope does not increase the capacity of the culvert. So the first principle determines only the minimum section at the culvert entrance and a minimum slope for the flow line.

Computation of entrance head and outfall velocity for a somewhat greater flow is too complicated for routine designs. But for the 100-year flood (40 to 80 per cent greater) the conduit will be full, or nearly full, so that hydraulic analysis may follow a simple procedure

From another viewpoint, this is "limit design" applied to culverts. Instead of arbitrary freeboards above 10-year floods, "balanced design" has zero freeboard above 100-year floods. Cost will be less than for the former practice; even if first cost is greater for some cases, annual cost will be reduced by the infrequency of damage.

This paper outlines the essentials of balanced design,—rational hydrology, selection of size and type of barrel, location, gradient, entrance and headwalls, outlet and endwalls and energy dissipators. Advantage of siphoning for low-head culverts is analyzed and exemplified. Experimental development of rough bores in concrete barrels introduces a means of controlling outfall velocities An arbitrary control is advocated for large culverts to avoid irrational extrapolation of small-culvert formulae,—for which the term "rated waterway" is defined and evaluated

The effects of trenching, foundation conditions, and back-filling operations on the effective earth load supported by a culvert are made plain for reference by construction engineers Debris control, which is a major problem on California streams is treated by definition of debris types, classification of remedial measures and recommendations for design, construction, and maintenance.

Insofar as State-wide policies are concerned, highway drainage in California began with the inauguration of the State highway system in 1912 At that time, rainfall and runoff data throughout California were meager and scattered and generally inadequate for predication of engineering design It was not until 26 years later, in 1937–8, that the first of several general storms of extreme intensity occurred to test the validity of a quarter-century of highway drainage practice In this series of storms, rainfall records were broken, resulting in unprecedented damage, State-wide in scope, exceeding $8,000,000 on State highways alone Although drainage structures in general functioned satisfactorily, there were enough inconsistencies in the performance of culverts and drainage facilities to warrant a comprehensive study and survey

Such a study was ordered and conducted over a two-year period by a joint departmental committee[2] of the Division of Highways, Department of Public Works

Field observations and studies covering the State indicated that culvert practice had long passed the stage where empirical determination of the size of a culvert opening was the all-important essential of design, emphasized to the point of disregard of other hydraulic and installation refinements.

This paper records observations and conclusions of the California survey along with corrective measures adopted to bring culvert practice abreast the field of experimentation and experience. Basic principles are outlined which, with development, it is hoped will place culvert design and practice on a sound engineering plane comparable to other phases of highway practice

COMPARATIVE HYDROLOGY

Early in the investigation need of greater uniformity of practice based on state-wide hydrologic studies became evident Preliminary analyses demonstrated the necessity for divergence of California practice from that developed for areas in the mid-western and eastern States—particularly small drainage areas.

Consideration of Frequency

If basic data are available, we can estimate statistically the extreme flood which will occur, on the average, once in (say) 100 yr Such a flood is just as likely to occur in 1943

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as in 2042, but the odds are 99 to 1 against such a flood being equalled or exceeded in any particular year.

The term "100-yr. flood" may be applied to any statistical phase of a flood—its stage, duration, mean daily discharge, momentary peak discharge, etc For culverts, the critical phase is the momentary peak. Hence in this paper we will define the "100-yr. flood" as the momentary peak discharge which will occur, on the average, once in 100 yr By the phrase "on the average", it is implied that all peaks could be observed for a series of centuries and an average struck of century peaks This is Fuller's concept(1)[3] applied to small streams

The Fuller concept applies to short records of culvert-size streams, as it appears to yield a simple exponential relationship between frequency and flood magnitude This may be illustrated by Figure 1

Fuller reasoned that flood records plotted in this way should yield flat curves which would straighten as the record lengthened. Hence the slope of the lower part of the curve would permit an estimate of the ultimate straightline variation, from which long-period floods could be predicted. The fact that curves of several streams in one climatic region had nearly the same slope led to the Fuller formula· $R=1+0.8 \log T$, where R is the ratio of the T-yr. flood to the 1-yr flood.

California Frequency. Also plotted on Figure 1 are flood data from six small California streams from two distinct climatic regions. The curves demonstrate an approximate law for each region, differing severely from each other and from eastern laws.

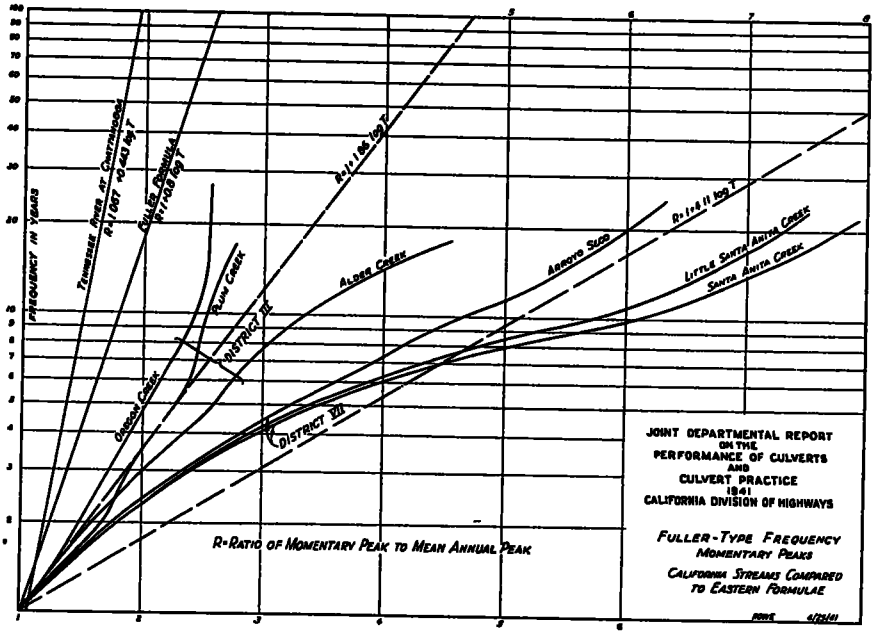


Figure 1. Comparison of Eastern Flood Frequency With That of Two Regions in California

These streams were selected as the smallest for which there is a fair record (17–28 yr.) and negligible regulation Basic data and deductions are shown in Table 1.

The diversity of frequency may be illustrated by applying Figure 1 to four hypothetical streams for which the mean annual peak is 100 sec -ft. On the first, in Tennessee, a flood of 200 sec -ft ($R=2$) would occur, on the average, once in 128 yr. In Fuller's region, the frequency would be 18 yr.; in District III[4] of California, 3 5 yr.; in District VII,[4] once in 1.75 yr.

[3] Numerals in parentheses refer to the list of references at the end of the paper.

[4] State Highway Districts.

If, for each stream, we compute the 100-yr. flood, we find. in Tennessee, 195 sec -ft ; in Fuller's region, 260 sec -ft , in District III, 472 sec -ft , in District VII, 922 sec.-ft

These deductions from Figure 1 should be considered illustrative, not quantitative There may be areas in California, such as the north coast, where the Fuller Formula would apply with slight adjustment There are other areas, such as the southern deserts, where the law must be more extreme than in District VII. As data are collected, hydrologic laws will be defined more precisely—both as to boundaries of regions and statistical constants. This has been started in District VII on a noteworthy scale(3)

On the average, 10 floods of 1,000-yr. frequency were experienced each year, somewhere in the group. But, of course, there were more than 10 in "wet years," less than 10 in "dry years." This is significant. Finding report or water marks of a recent flood of such infrequency, an engineer will be inclined toward over-design.

On the contrary, there is a possibility that some basin just missed the big storms, so that the maximum flood experienced in 30 yr. is only a 5-yr. flood. Here the reports and water marks will lead to under-design

Weather cycles, whatever the cause, may also lead to erroneous conclusions as to frequency. Historical summaries (2b) recall that floods were general throughout California in the 10 climatic years 1862, 1868, 1879, 1881, 1890, 1907, 1909, 1915, 1916 and 1938. On the average, California experiences one general outstanding flood every 8 yr , but intervals range from one to 22 yr.

TABLE 1

FREQUENCY FACTORS OF SIX SMALL CALIFORNIA STREAMS COMPARED TO EASTERN TYPES

Stream	Basin Area	Length of Record Years	Gage Elevation	Mean Discharge	Annual Momentary Peak			Fuller's Constant $\frac{R-1}{\text{Log } T}$	Ratio of 100-yr Flood	
					Minimum	Mean	Maximum		To Annual Flood	To 10-Yr Flood
	sq mi		*ft*	*sec-ft*	*sec-ft*	*sec-ft.*	*sec-ft*			
Eastern U S										
Fuller Ideal		20			661	1,000	2,041	0 8	2 60	1 44
Tennessee River	21,400	57	620	38,300	85,900	208,600	361,000	0 5	1 95	1 29
District III										
Plum Creek	6 8	17	4,100	8 1	33	230	635	1 70	4 40	1 63
Alder Creek	22 8	18	4,000	30 0	92	387	1,760	2 43	5 86	1 71
Oregon Creek	35 1	28	1,500	72 5	295	1,596	4,080	1 45	3 90	1 59
District VII										
Little Santa Anita	1 9	22	2,200	1 0	3	107	800	4 31	9 62	1 81
Santa Anita Creek	10 5	22	1,400	5 6	34	672	5,330	4 46	9 92	1 82
Arroyo Seco	16 4	25	1,400	9 6	81	1,366	8,620	3 57	8 14	1 78

Misconceptions of Frequency. Before leaving the subject of frequency, there are a few points that require further explanation One, in particular, is the probable frequency of recent floods observed at highway bridges and culverts

Suppose, for illustration, we take a random group of 10,000 drainage basins in California, selected by some arbitrary rule—say those intercepted by the 10,000 largest cross-drainage highway culverts. Suppose also that we had observed the maximum annual flood at each of these culverts for 10 yr. Suppose further that weather was random and noncyclic Then it is probable that, of the 100,000 annual floods, 10,000 were 10-yr. floods or greater, 1,000 were 100-yr floods or greater, 100 were 1,000-yr floods or greater, etc.

Over-design probably followed the four floods in one decade, 1907–1916, just as under-design was the rule towards ends of the droughts of 1891–1906 and 1917–1937 To avoid generosity in current designs, following a new series of flood years, frequency studies must determine a normal expectancy

Fuller-type frequency has been criticized because there is no upper limit to floods. The curve for Santa Anita Creek can be extended to determine how often we can expect 50,000 sec -ft. to pass that way,—once in 26,000,000,000,000,000 yr. Both discharge

and time are so remote as to be meaningless. The obvious impropriety of designing for such indefinite extremes has led to the conception of the "design flood"

The "Design Flood". If, for a certain site, we can estimate reliably the magnitudes of floods which on the average should occur with certain frequencies, we can select the flood of some one frequency as the "design flood" from principles of economy. We can estimate the damage likely to be caused by greater floods and compute the annual cost of this contingency. This should just equal the allowable annual cost of additional capital investment to provide waterway for the greater flood

For large bridges, a separate study can be made for each site, to obtain maximum long-time economy For culverts and small bridges, some rule-of-thumb is required.

The most important conclusion reached involves two general rules. one, that a culvert just pass a 10-yr. flood without static head on crown of culvert at the entrance, two, that design of culvert and appurtenances be balanced to avoid serious damage from head and velocity obtaining in a 100-yr. flood. Of course there will be justifiable exceptions to any such rules, some of which will be discussed later.

Culvert Formulae

Designers use a wide variety of culvert formulae, it being common practice to apply at least two for a check. Since most formulae include an arbitrary coefficient, the value of a check depends upon the experience and judgment behind the selection of coefficients

The basic formulae were developed for railroad practice in eastern United States Figure 1 demonstrates need of special adaptation for California practice This has been done to a large extent, but not from the aspect of frequency.

The commonest culvert formulae are of the type $a = CA^n$ where a is the area of culvert section, C a coefficient depending upon climate, topography and units of area, A the

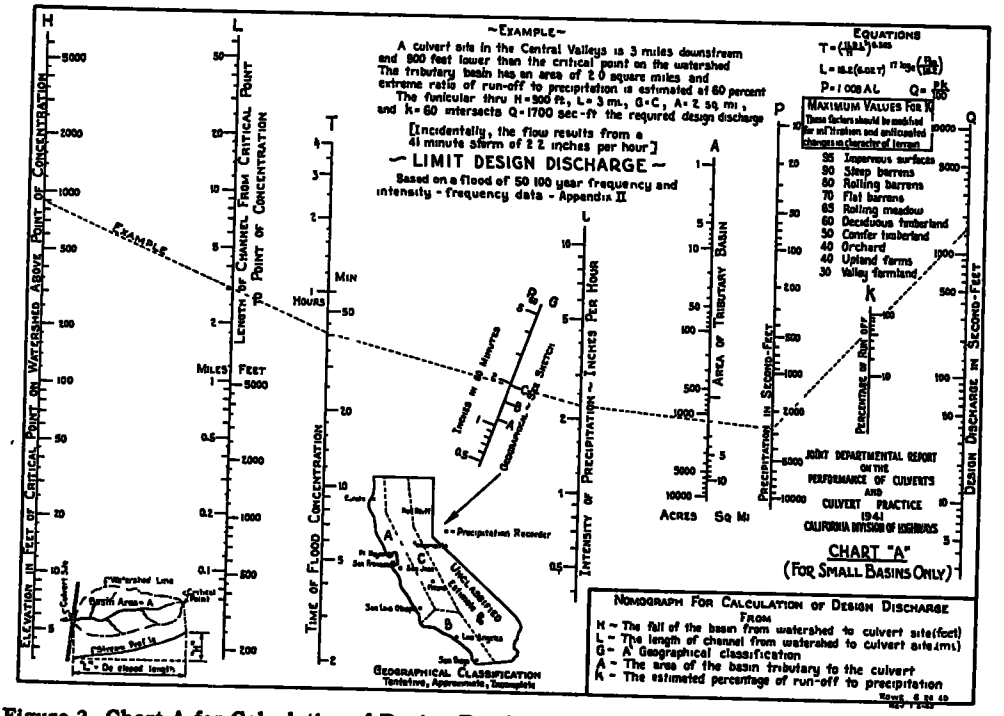


Figure 2. Chart A for Calculation of Design Discharge by a New Formula in Nomographic Form

area of watershed and n an exponent varying (by authors) from 0 5 to 0 8.

Talbot found $n=0\ 75$ and gave rules for estimating C, Wadsworth (4) confirmed this for Sierra streams in 1907 Recent studies by the bridge department of the California Division of Highways confirm it generally for northern California, but find $n=0\ 70$ for southern California From the data in Table 1 it appears that a 100-yr flood would overload a culvert designed by the Talbot formula for a 10-yr flood by 44 per cent in Fuller's region, 64 per cent in District III of California, and 80 per cent in District VII. Hence, knowledge of the magnitude of 100-yr. floods will permit balance of designs based on Talbot's formula.

Chart for Small Basins For small drainage areas in California, the Committee suggested use of Chart A, (Fig 2) to evaluate a judgment factor "K", which depends upon topography The chart is a formula in nomographic form depending upon four measurable items (length and fall of channel from critical point on divide to site, mean rainfall-intensity for 60 min. expected once in 100 yr and area of tributary basin) to yield a design discharge (momentary peak once in 100 yr).

Intermediate steps of the chart give the time of flood concentration for the basin and the mean intensity of precipitation for that concentration period. If hydrology for some particular region affords a more rational estimate of either of these factors, use of the chart may start from that point and continue through the other steps

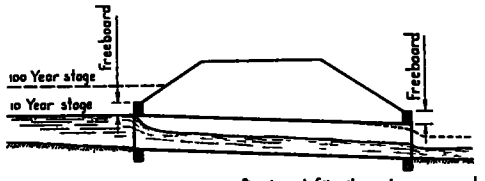
At the same time Figure 2, can be used as a check method. For "K", the chart lists probable maximum values for certain types of topography Minimum values may be smaller by 20 percentage points For rainfall-intensity, the data now available are approximated for coastal and valley areas designated by letters A B and C on a key map Similar points can be plotted for areas in other states by use of the P_{60} scale, representing the 100-yr. rainfall for 60-min.

The most comprehensive study of rainfall-intensity frequency available is that by Yarnell (5) For all of the United States except California, he prepared intensity-frequency charts which will serve to select tentative values for the 100-yr. frequency, 60-min intensity These should be modified for local use, where data are available, particularly if orographic variation is known.

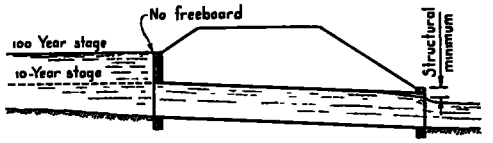
In California, the data are still meager, but a careful study (6) was made to locate Points A, B and C

CULVERT SIZE AND TYPE

Ordinary culvert formulae provide for passage of a 10-yr. flood without static head on the crown of the culvert at entrance. This is approximately equivalent to application of the Talbot Formula, which is used to some extent (7) by 36 states The AASHO (8) states "In general, culverts shall be proportioned to carry the maximum flood discharge without head "



(a) Current Practice Frequent flood just fills the entrance, equal freeboards, insufficient at entrance, excessive at outlet



(b) Balanced Design Infrequent flood submerges entrance, appurtenances fit this stage without freeboard

Figure 3. Comparison of Controls for Current Practice and Balanced Design for Free-outlet Culvert.

If the "maximum flood discharge" just reached the crown of the culvert entrance, so that flow would be "without head", there would be no need for headwalls above that level, and parapets would serve only to retain the highway embankment. However, a large proportion of culverts are subjected to considerable head on the entrance, so that parapets serve the further purpose of protecting the embankment from erosion.

Headwalls are overtopped much more often than endwalls and to a much greater degree (Fig. 3a), suggesting that parapet elevation should not be determined arbitarily, but from hydraulic study.

Strict adherence to the practice ignores the opportunity for reduction of the culvert section below the entrance if there is a fair slope available, for the outlet is far from full when the entrance is just full. Actually, the water surface drops rapidly just inside the entrance, and a large portion of the waterway is never utilized (Fig. 3a). In the case of a culvert so selected, increasing a slope beyond neutral slope does not increase capacity of the culvert since the size of entrance controls capacity.

Solutions have been proposed from time to time to correct the uneconomic practice, such as use of belled entrances to pipe culverts and tapered barrels in concrete boxes. For various reasons progress ended with the experimental installations.

Balanced Design Proposed

"Balanced Design" is proposed as one step in the improvement of practice. Instead of constructing headwalls, endwalls and other facilities to arbitrary freeboards, the combination of culvert barrel and all appurtenances should barely satisfy for the 100-yr. flood without any freeboard (see Fig 3b) The limiting flood, designated the "design discharge", has been given an approximate frequency of once in 100 yr It is an "ultimate capacity" of the system, beyond which there may occur still greater floods which will damage all parts of the system—perhaps destructively.

Balanced design is defined as that combination of conduit section, shape, texture and gradient with entrance and outlet appurtenances which will just pass a 100-yr. flood without interruption of traffic and without serious damage to structure, embankment or abutting property

To obtain such balance, the designer must know the stages and velocities at critical points of a trial layout and the durability of structure, embankment and natural channel where exposed thereto.

Computation of these items for a large number of culverts becomes a tremendous task Tables are available for certain kinds of pipe and for short culverts, but the committee was unable to find any compact combination of tables and charts applicable to the widely variable California conditions

The Iowa Formulae. The formulae developed at the University of Iowa (9) after tests in cooperation with the Bureau of Public Roads (now the Public Roads Administration) were found applicable to all designs of the California Division of Highways Since the tests in the Iowa experiments were limited to pipes up to 30 in. in diameter and concrete boxes up to 4 by 4 ft, 30 6 ft. long, any set of tables or charts to cover current practice requires extrapolation to six times the diameter and length of the test units, which is a reasonably small prototype-to-model scale ratio. The Iowa formulae are of the general form·

$$Q = AV = \frac{A\sqrt{2gH}}{\sqrt{1 + aR^m + \frac{bL}{R^n}}}$$

where g is the acceleration of gravity, A, L, Q, R and V are the culvert area, length, discharge, hydraulic radius and velocity, and H is the total culvert head in feet from headwater to tailwater,—all in foot-second units. The other letters represent constants determined experimentally for certain barrel textures and entrance conditions, varying as follows a from 0 05 to 0 62, b from 0 0045 to 0 0201, m from 0 to 1.9 and n from 1 0 to 1 25 These formulae are not suitable for field computations To facilitate application in California, a chart (19) was prepared to relate H, L and Q for types and sizes of culverts in common use Originally it had been designated as Chart B of the series A–D of the joint departmental report.

Procedure Recommended. The steps recommended for balanced design of culverts can be summarized briefly as follows

(a) Determine from maps, highway records and field study the basic data required for Chart A (Fig 1) and for application of at least one culvert formula, and for at least one field estimate of flood discharge.

(b) Compute and compare discharges. If culvert formula leads to a waterway area, take the design discharge in second-feet at 15 to 18 times the area in square feet. Anticipate that recent high water marks may represent a flood of anywhere from 30 to 120 per cent of the 100-yr. flood. If differences are reasonable, select a weighted mean, other-

wise, analyze the data and allow for the effect of unusual factors.

(c) By preliminary application of engineering factors (list follows) eliminate from further consideration any type which should not be used at the particular site

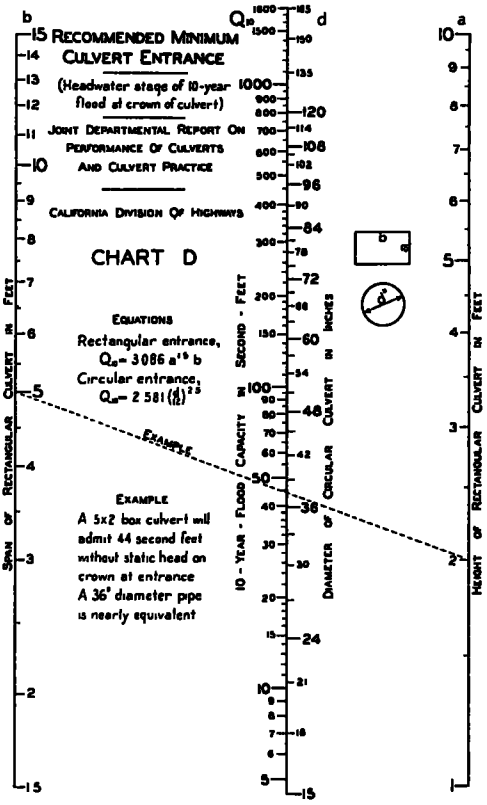


Figure 4. Chart D for Determining Minimum Culvert Entrance Section From Estimated 10-yr. Flood.

(d) Determine minimum sections by use of Chart D (Fig 4) for a 10-yr. flood, which may be taken at 55 to 65 per cent of the design discharge Modify this minimum if required by item d'.

(d') For large free-outlet culverts, the 10-yr. flood in sec -ft should not exceed 10 times the rated waterway of the section in square feet.

(e) Apply working charts (19) or pertinent formulae to the minimum sections to determine head on the culvert, and compute the headwater stage. If this stage is too high, use the allowable stage and determine the cheapest conduit which will satisfy this restriction.

(f) Study conditions at the entrance with the help of profile or site plan and consider the effect of headwater pool on natural channel, upstream property and the highway embankment. For each design still being considered (2 or 3 at most) estimate the cost of reasonable protection without freeboard As part of this step, it may be advisable to modify the tentative section selections.

(g) Establish grades or gradient of culvert flowline and estimate velocities at outlet for both the 10-yr. flood and the design discharge (100-yr flood)

(h) Study conditions at the outlet and consider the effects of high velocity, eddies or other turbulence on natural channel, downstream property and the highway embankment. For each design, estimate the cost of reasonable protection—against nominal loss in 10-yr floods and against serious loss in 100-yr. floods. Include energy-dissipator in estimate, unless it can be shown that maintenance charges after damage will be less expensive.

(i) Estimate the annual cost, including anticipated maintenance, of each tentative balanced design of culvert and appurtenances (for the larger structures, this may include a credit for displaced embankment, for some box culverts there may be a credit for displaced subgrade and slab).

(j) Select the most economic combination, giving reasonable weight to the following engineering factors and department policies.

There are five principal engineering factors which should always be appraised before final selection is made.

(a) Character and stability of underlying foundation material on which culvert is to be laid.

(b) Nature and extent of lateral forces acting in the covering embankment.

(c) Effect of earth loads from high embankments.

(d) Effect of impact under shallow earth cover.

(e) Accessibility of culvert site

In special cases, minor engineering considerations may be sufficient to determine type, such as

(f) Salvage value where installations are temporary.

(g) Facility of extending existing culverts.

(h) Adaptability to jacking under pavements

(i) Resistance to alkali, salts, and acids.

(j) Resistance to abrasive action of stone-laden stream flow on the invert

(k) Desirability of eliminating endwalls by cantilevering extensions.

(l) Advantage of using one type of culvert throughout the contract.

Foundation and Earth Loading. Commercial culverts of both the flexible and rigid types are limited as to safe height of fill. Above such limitations it becomes necessary to design monolithic types to support the weight of the high over-fill.

Pipe culverts laid on excessively yielding foundations and culverts laid under embankments on sidehill are subject to lateral movement and should be of the type that best resists disarticulation. Flexible pipes in long lengths of heavy gage, with extra long collars have proven more satisfactory in such cases than short length sections of rigid types.

Other Factors Influencing Selections. For pioneer roads in mountainous country and similar inaccessible locations where deep gulches are encountered, the deciding factor in selection of type may be the speed with which the installation can be made Long sections of the flexible type, light in weight, are readily adaptable to such locations.

Selection of size or type of culvert may be finally determined by departmental policy such as.

(a) Adopted minimum size of culvert for maintenance purposes—for instance, minimum 18-in. pipe culverts under shallow fills or minimum 24-in. under high fills

(b) Limitation of unproven types of culverts to experimental installations.

(c) Conformance with specification policies of participating governmental agencies.

(d) Conformance with national governmental dictates in critical periods

CULVERT LOCATION AND SLOPE

Modern traffic demands and development of earth-moving equipment justifies increasingly higher standards of alignment and grade. In consequence, cuts through ridges are deeper and embankments over depressions are higher. These higher embankments increase the burden on culverts so that stronger and more expensive conduit sections are required.

To offset the combination of greater length and higher unit cost, every expedient seems to have been tried Many cases were observed and studied in which expedients of first-cost economy had sacrificed hydraulic essentials. Judgment, rather than rule, must be the determinant, but some general rules should be helpful in avoiding wide use of expedients which should be applied only in limited fields.

Horizontal and vertical locations of a culvert are interdependent and generally are selected by "rule-of-thumb" practices Conforming to rational and orderly procedure in design, both alignment and slope should be analyzed hydraulically.

Bottom Location

To clarify the study, location practice has been classified as illustrated in Figure 5a–g. The bottom location is the natural position for the culvert with flow line generally coinciding with channel bed. With few exceptions, this is the best location hydraulically, even if alignment must be curved, or grade line broken Conduit cost will be a maximum, but alternatives should not be compared without estimating total culvert cost, including headwalls, spillways, channel changes, revetment, maintenance and probable damage if natural conditions are altered.

In the modified bottom location, the outlet grade is raised above natural grade, shortening the distance between embankment slopes This has been successful for small pipe culverts in steep channels, as the conduit can project and cantilever from the bank so that outfall will clear the toe of the embankment slope. Even for medium-size culverts of pipe or box section, the modification has been applied by aligning the conduit with the gradetour (grade contour), so that the culvert is built throughout on solid material and outfalls to one side of the natural channel

Sidehill Locations

If both entrance and outlet are set well above natural grade, the alignment usually ignores the natural channel, following a gradetour on that side of the ravine where highway grade is low This has been designated a sidehill location (or a top location if conduit is laid just below the roadway grade). For the latter, hazard of flood overtopping the roadway should be investigated. In

Culvert Locations Defined

Bottom Location

Culvert located in approximate line and profile of natural channel

Modified Bottom Location

Culvert entrance in bottom of natural channel and outlet projecting from embankment slope.

Sidehill Location

Culvert entrance and outlet well above bottom. Wrong On an unstable berm. Right In trench or on solid berm.

Top Location

Culvert entrance and outlet near roadway grade, well above bottom

Diversion Location, Normal

Culvert normal to roadway, diverting a skewed stream at entrance or outlet or both.

Diversion Location, Preferred

Small skews eliminated, moderate skews retained, large skews reduced

Transverse Relief Location

Culvert located transverse to roadway to intercept gutter flow at regular intervals

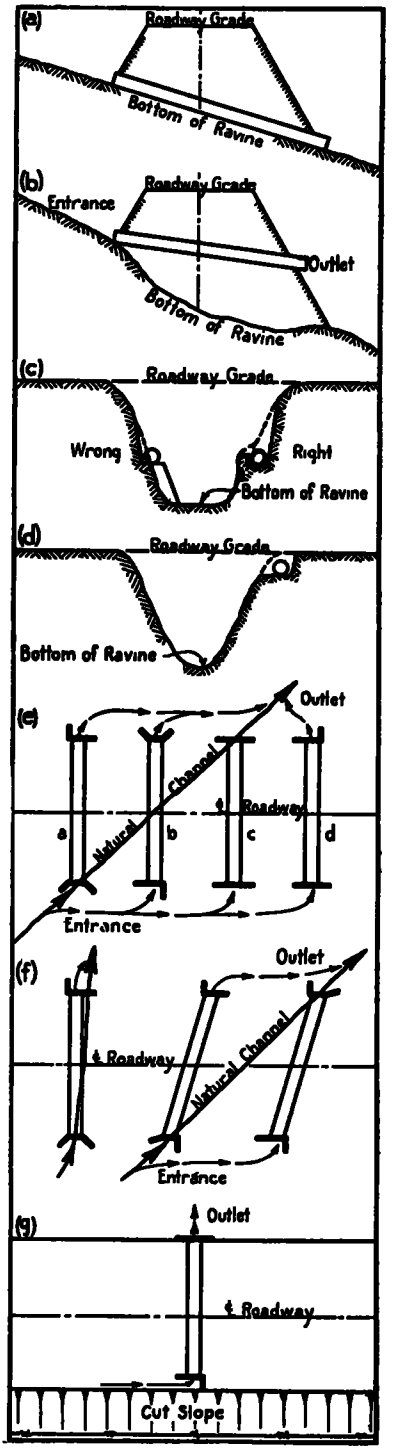


Figure 5. Culvert Locations Defined

either case, the embankment must serve as a dam and may require special attention, such as use of impervious borrow or greater compaction on the upstream slope and provision for drainage of the rest of the prism

Such a location increases stage for some distance upstream, aggrades the bed with mineral debris and leaves a stagnant pond after each flow Selection of this type presumes that these changes are tolerable In some cases, however, aggradation filled the channel rapidly, so that approach channels had to be maintained, but bottom locations would have passed debris through the culverts indefinitely with little maintenence.

Also, for sidehill locations, conduits may be subjected to unusual stress by the shearing force induced by weight of a settling embankment Unbalanced pressures at such points have caused total collapse of pipes. This can be avoided by adjusting alignment and grade of the culvert until the conduit is entrenched throughout in firm material

Diversion Alignment

The Diversion Location (Fig. 5e) is the substitution of a normal (or nearly so) alignment for the natural skew of the channel Four atlernatives are sketched as typical of installations observed in the study. All require that the channel be positively changed or that the embankment toe resist erosion by flood flows For high velocity flow, this means revetment of earth slopes and careful design of headwalls and endwalls

As a general rule, flood waters should be conducted under the highway at first opportunity, as for alternative (a), minimizing scour of embankment and entrapment of debris. If the culvert gradient will be steep or if free drop is planned at outfall, then alternative (c) may be justified, but estimates should include annual cost of maintaining channel and embankment toe above the culvert entrance. Alternative (b) is a compromise that can be justified only when local conditions limit channel changes at both ends. Alternative (d) will usually be a sidehill location and should be studied as such. Rarely, in a U-shaped valley, this alignment will resemble a bottom location, with the advantage of dry trench or structure excavation, but with all the disadvantages of alternatives (a) and (c) combined.

One point should be emphasized as a general criticism of alignment in diversion location Reduction of skew should be considerate of extra cost of skew construction and of channel change and maintennce At one extreme, consider a box culvert 100 ft long on a 3-deg skew Extra cost of framing the skew structure would probably exceed the alternative channel adjustment (5 ft) for a normal alignment At the other extreme, suppose the channel skew was 45 deg, a normal culvert would appear to save 41 ft of conduit by adding 100 ft of channel work. But an 8-deg skew culvert would have saved 40 ft of conduit by adding only 86 ft of channel.

Hence, as a general rule, small skews should be eliminated, moderate skews retained and large skews reduced—the limits being determinable for each site by cost comparison. Accordingly, the second illustration of diversion location (Fig 5f) indicates the preferred locations.

Slopes Generally

It has already been recommended that culverts be designed to carry a 10-yr. flood without static head on crown at entrance and that balanced design of culvert and appurtenances be based on the 100-yr. flood Either of these rules may (for some particular site) determine culvert size or gradient. Generally, however, the first rule will determine only a minimum waterway area and the second will yield balanced combinations of area, gradient and conduit texture for cost comparison.

Since gradient assumes its greatest importance when considering the rule for a 100-yr. flood, we have defined neutral slope as the gradient which will just carry the 100-yr. flood with a full waterway.

Computation Charts

Chart C (Fig 6) has been prepared to show the relation between full capacity and neutral slope for culverts of concrete or corrugated metal The relationships were derived from the Iowa formulae (9) and studies of spun concrete pipe by Scobey (17) and depend upon size, shape and texture of the conduit, but not its length For circular sections, the slope is obtained in one step with the isopleth intercepting the 100-yr flood on

the discharge scale and the diameter scale corresponding to the pipe material For non-circular sections (box or arch) of cast-in-place concrete, there are two steps, the first isopleth intercepting perimeter and area scales to obtain a turning point on the diameter scale, whence a second isopleth through the discharge obtains the neutral slope.

Before applying this chart, a minimum conduit section should be determined from the 10-yr. flood criterion, remembering that the 10-yr flood is only 55 to 65 per cent of the 100-yr. flood Chart D (Fig 4) gives this directly for circular conduits and a series of isopleths rotated through the 10-yr. flood on the discharge scale will give a series of equivalent rectangular box dimensions.

For example, suppose for a certain site that

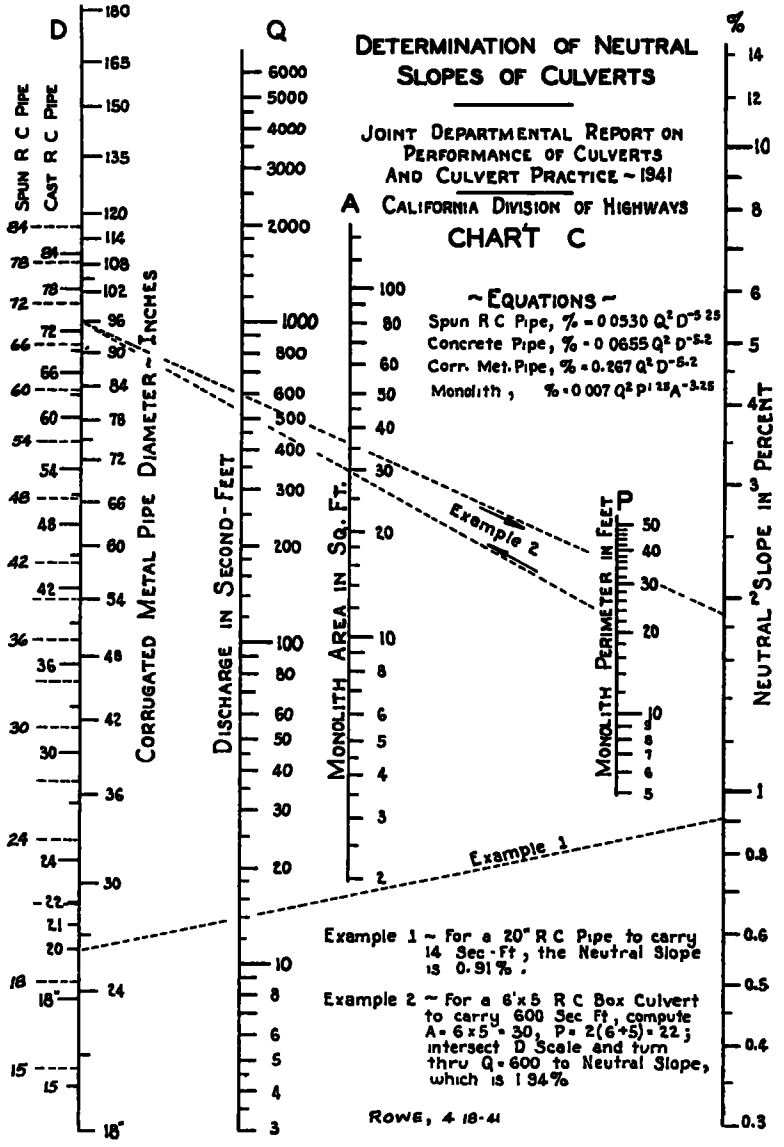


Figure 6. Chart C for Computing Neutral Slope from Estimated 100-yr. Flood

the 10-yr. flood is 145 sec-ft From Figure 4, a 60-in. pipe or a 6 by 3 9 ft. box will provide the minimum entrance section. For this frequent flood, capacity is governed by size and shape of entrance, regardless of slope and texture of conduit.

Suppose further that the 100-yr flood is 250 sec -ft (Fig. 6), showing the importance of slope and texture, gives this assortment of neutral slopes Corrugated metal pipe, 0 0385, Concrete pipe, cast, 0 0095, Concrete pipe, spun, 0 0071, R C. Box, 6 by 3 9 ft , 0 0071

If any of these conduits is laid on its neutral slope the depth of headwater pool above crown of culvert will just equal the entrance head,—that is, entrance loss plus velocity head. The culvert will flow full without pressure on the crown line, except for the draw-down near the outlet Outfall velocity will be at a minimum.

Accommodation of Extreme Slopes

Diminishing the slope below the neutral has little effect on outfall velocity, but increases headwater stage by an amount equal to the product of the slope decrement and the culvert length Thus, laying a 200-ft. metal pipe on the neutral slope for a cast concrete pipe would increase the headwater stage by 5 8 ft.

Increasing the slope above the neutral will not reduce the headwater stage, but outfall energy and velocity will be materially augmented

Usually the culvert slope is determined closely by the mean gradient of the existing channel, so that there will be an advantage in using the conduit for which this is nearly the neutral slope. Failing in this, the effect of the slope must be offset by appropriate means

For very mild gradients there are two alternatives, a larger opening or protection against backwater at entrance. For very steep channels there are at least four alternatives. (a) the multiple culvert, (b) an extended culvert discharging at high velocity well beyond the toe of embankment, (c) a projecting culvert with free fall at outlet; or (d) a broken-grade flow line

For the intermediate range, modifications of either metal pipe or box culvert are practical. For the metal pipe, the backwater stage can be computed and headwalls designed accordingly, or a larger pipe may be specified. The box (or arch) culvert can be roughened, or its grade broken, or its invert stepped, or high velocity anticipated at outlet

Experimental Designs

The combination of a stepped invert with roughened invert and walls was introduced in an experimental arch culvert recently built at McNamee Creek in California (Fig 7).

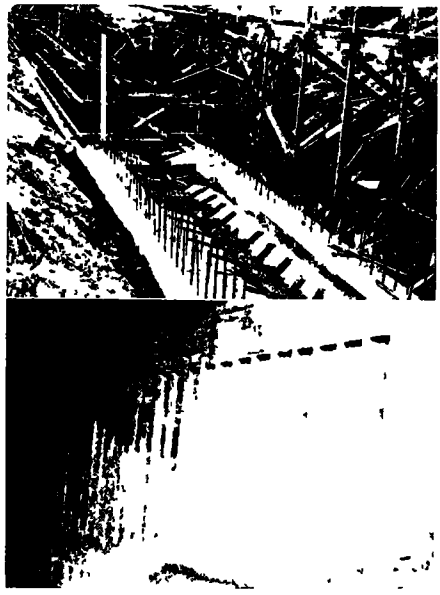


Figure 7. Upper: Invert Steps and Herring-bone Roughening to Create Turbulence and Reduce Outlet Velocity.Lower: Fluted Side-walls to Further Reduce Velocity (McNamee Creek).

Here the natural channel slope was 3 8 per cent, 50-ft steps with 0 9-ft risers reduced the effective gradient on the steps to 2 0 per cent. The barrel was the standard 73-sq -ft. arch 252 ft long under a 60-ft embankment. Its invert was roughened in a herring-bone pattern by pressing two by sixes in the fresh concrete The walls below springing were fluted by nailing scabs to the forms.

Several other types were suggested, but disapproved because of hazard of blocking by drift One, which would be very economical for clear water on a steep slope, would provide a normal entrance waterway, contracting at once to increase velocity and friction, then

expanding near outlet to regain low-velocity outfall. Others provide baffles or weirs, as for canal chutes and drops

The hazard of silt deposit in culverts laid on flat gradients is well known. To avoid such deposit, culvert slope must assure a velocity throughout its barrel greater than the ruling velocity of the stream—not only at flood stage, but at all stages of roily flow. For some sites, this principle will warrant laying culverts on gradients somewhat steeper than critical This is particularly applicable to box culverts, but sufficiently high velocity at low stage may be gained by shaping the invert like a broad flat V.

ENTRANCES AND HEADWALLS

For the past 30 yr. in California, empirical determination of the size of a culvert opening has been the first essential of design. The ability of a culvert to carry drainage water under the highway has been measured by the size of entrance opening, with too little attention to other hydraulic and installation refinements There was some justification in the fact that run-off records were few and short, so that floods could not be estimated closely This practice was no doubt widespread and recognized as such at the time of Iowa's experiments (9) on the flow of water through culverts in 1926.

With accumulation of longer-period rainfall records enabling increasingly accurate determination of the intensity-frequency of run-off, our earlier knowledge of culvert hydraulics can be applied in culvert design. Such refinement must start at the culvert entrance.

Rounded-lip and Beveled-lip Entrances

Rounding, beveling or expanding the entrance in almost any way will increase capacity for every design condition—whether outlet is free or submerged; whether slope is above or below neutral; whether 10-yr. flood just fills the entrance (current practice) or the 100-yr. flood fills the pipe throughout with head at entrance (balanced design).

Under conditions of balanced design, for a given discharge the rounded-lip entrance will maintain the same mean velocity at less head, the effect of which is to harmonize velocity distribution within the section by accelerating the flow along the wetted perimeter.

Figure 8 defines the common forms of expanded entrances, for all of which the advantage increases with culvert diameter and decreases with culvert length The Iowa tests (9) produced formulae from which the advantage may be computed in terms of equivalent sections, or of relative efficiency, or of entrance head.

For example, a 39-in beveled-lip concrete pipe 30 ft long has the same capacity as a 42-in square-ended concrete pipe. On the other hand, a 51-in beveled-lip concrete pipe

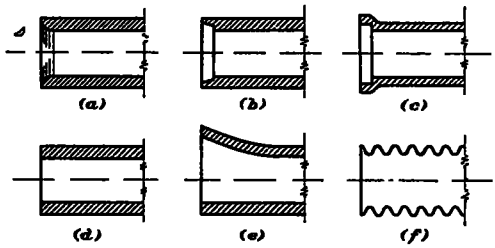


Figure 8. Types of Culvert Entrances. (a) Rounded-lip (concrete box). (b) Beveled-lip (concrete pipe), (c) Belled-lip (concrete or vitrified pipe), (d) Square-ended (concrete box), (e) Belled or Throated (concrete), (f) Affected Rounded-lip (corrugated metal).

TABLE 2

Diameter Concrete Pipe	*E* in Percent for Culvert Length		
	30 ft	100 ft	200 ft
in			
18	85	91	94
48	71	75	80
72	64	68	72

200 ft. long is just equal in capacity to a 54-in. square-ended pipe.

The saving in entrance loss for concrete pipes is shown briefly in Table 2 in which *E* represents the percentage of total head lost in beveled-lip pipe to that lost in a square-ended pipe of the same diameter. In other words *E* is the efficiency of the square-ended pipe relative to the beveled-lip pipe.

It should be noted that beveled-lips are commercially available without premium. For a rounded lip, cost of special forms may offset any saving, so that the practice is not recommended.

The same principle affects concrete box culverts to a greater degree, because the entrance can be rounded to the optimum shape.

Hydraulics Affecting Culvert Entrances

The head required for a culvert has at least four components (a) velocity head, (b) entrance loss, (c) friction loss, (d) eddy loss

The first two are effective at the entrance, unless the culvert becomes a siphon The elevation of the headwater pool above the crown of the culvert cannot be safely estimated below their sum, although the Iowa tests showed some contradiction Hence, any saving in entrance loss will be reflected in a lowering of headwater pool, which means a decrease in backwater above the culvert.

Experiment (9) has determined that this saving is independent of culvert length and is proportional to the velocity head, in accordance with the following formulae.

For concrete pipe, $\Delta h_e = (0\ 31\ D^{0\ 5} - 0\ 10)\ h_v$

For box culvert, $\Delta h_e = (0\ 40\ R^{0\ 3} - 0\ 05)\ h_v$

Table 3 gives values for these expressions for several sizes of culverts and three entrance velocities.

TABLE 3

Culvert Size and Type	Saving in Entrance Loss (head in ft) by Beveled-lip or Rounded-lip over Square-ended Entrance for Velocity (ft per sec) of		
	5	10	15
18 in concrete pipe	0 11	0 44	0 98
48 in. concrete pipe	0 20	0 81	1 82
72 in concrete pipe	0 26	1 03	2 31
2 ft. by 1 5 ft concrete box	0 10	0 40	0 91
4 ft by 3 ft concrete box	0 13	0 52	1 16
8 ft by 6 ft concrete box	0 16	0 65	1 47
12 ft by 9 ft concrete box	0 19	0 75	1 68

Recommendations Summarized

Since use of beveled-lip entrances on concrete box culverts will either (a) reduce the size of culvert, or (b) reduce materially the back-water caused by the culvert, it is recommended

That beveled-lip entrances be adopted as standard practice for concrete pipe and rounded-lip entrances for concrete box culverts, with a radius of rounding equal to approximately 10 per cent of the greatest culvert dimension; and that square-ended entrances be considered as exceptions which need to be justified for either type of culvert.

That entrances to arch culverts, whether part-circle, multiple, or concrete, be rounded only below the spring line. The cost of forming a rounded-lip on the crown portion of a concrete arch will offset any saving in head.

That rounding of the concrete headwall to corrugated metal culverts be discouraged since the manufactured product already effects a partially rounded-entrance

That in special cases where it becomes economical or practicable to reduce the culvert section, as in the case of a very long culvert, throated-entrances (Fig 9) be considered. In waterways carrying heavy debris, some type of debris control must be installed to prevent large stumps from entering and choking the throat.

Figure 9. Forms of Throated Entrance to Concrete Arch Culvert. Note gradual flattening of grade towards outlet.

CULVERT WALLS GENERALLY

Widespread field inspection indicates an arbitrary and non-uniform practice in the selection and adaptation of headwalls and endwalls There is a tendency towards selection of the same standardized type (Fig. 10) for both the upstream headwall and the downstream endwall in disregard of the different stream flow conditions

At culvert entrances, wingwalls are frequently given an arbitrary flare of 45 deg, which generally proves unsatisfactory for high approach velocities or oblique approach flow Flaring wingwalls from the axis of the approaching stream flow instead of from the culvert axis, is advisable. Warped-wing headwalls (Fig. 11) costing little more than vertical wingwalls, reduce scour as well as loss of head at the culvert entrance

When combined with an apron on a drop-

down slope from the approach channel, an increase in velocity is induced by the contracting water section so as to materially increase efficiency of the culvert entrance. At the outlet, wing endwalls are often flared to lines that water cannot follow at high velocities. This results in scour of embankments from eddy action at the ends of the wingwalls each side of high velocity effluent.

Recent experiments (9) indicate that "the 45-deg. wingwalls used with a corrugated-metal-pipe culvert increase the capacity from 1 to 10 per cent over that obtained with straight endwalls. They are more efficient when set flush with the edge of the pipe than when set 6 in. back from the edge of the pipe, and when built full height to the top of the headwall than when constructed only to the standard height.

The 'U' type wingwalls used with a vitrified-clay-pipe culvert produce a carrying capacity slightly less than that with the straight headwall.

Recommended Adaptations

Straight Headwall. For low approach velocity or headpool, light floating debris, undefined approach channel; or small defined channel entering culvert without change of direction; or limited right of way, or small culvert near grade.

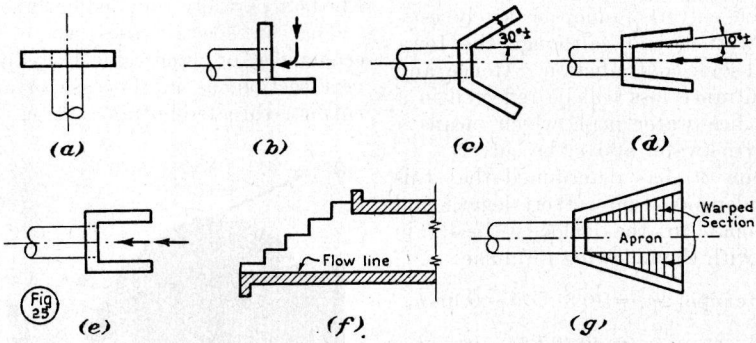


Figure 10. Types of Endwalls and Headwalls. (a) Straight Headwall or Endwall, (b) L-headwall (c) Wing Headwalls or Wing Endwall (30 deg. ± flare), (d) Flared Headwall (10 deg. ± flare), (e) U-type Headwall or Endwall, (f) Stepped U-type Headwall or Endwall, (g) Warped Wingwall or Endwall.

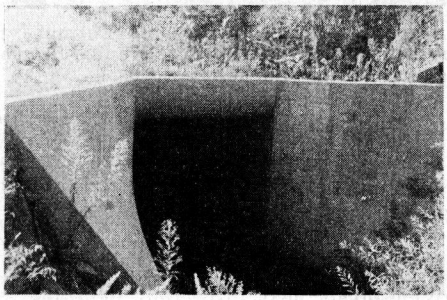


Figure 11. Warped-wing Headwall With Rounded-lip Crown at Entrance to Concrete Box Culvert.

Straight Endwall. For low velocity effluent not requiring outlet protection against eddy action.

L-Headwall. For gutter drainage (traverse relief culvert) where necessary to change abruptly course of water; natural defined channel where abrupt change of line cannot be avoided.

Wing Headwall or Endwall. For well-defined channel, moderate velocity, medium drift.

Warped-Wing Headwall. For well-defined channel, moderate-to-high approach flow, medium drift; is considered the most efficient type of practical headwall. Warped-wing headwalls provide a transition which narrows the channel and increases velocity from the natural channel to the culvert entrance—particularly effective with drop-down apron (Fig. 11).

Warped-Wing Endwall. For moderate-to-high velocity discharge, is considered the most efficient type of endwall at outlet because it reduces the drop down curve (free outlet) and minimizes velocity at the end of the apron.

U-type Walls. Most inefficient (9) type of

headwall or endwall, insofar as hydraulic considerations are concerned; advantage lies in ease and economy of extension (Fig. 12); recommended only for cattle passes where drainage is a minor factor.

Flared Headwall. For well defined channel with moderate approach velocity, medium drift at high velocity, and heavy drift at moderate velocity. Advantage lies in ability of slightly flared walls to align drift so that it passes the culvert endwise.

No Endwall. Endwalls are not required at the downstream end of pipe culverts where there is little likelihood of damage from erosion. Cantilever extensions of metal pipe are generally cheaper than endwalls. Experiments (9) indicate that a headwall at the upstream end of a culvert increases entrance efficiency over a culvert with no headwall.

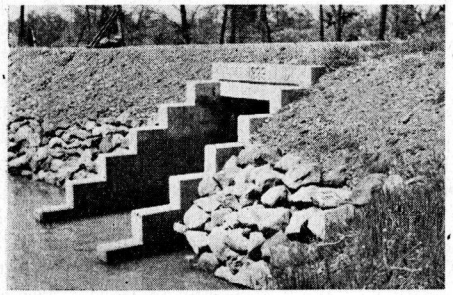


Figure 12. Stepped U-Type Endwall. Most inefficient type—not recommended except for cattlepass where drainage is a minor factor.

Miter and Skew End-Cuts

Where culverts are laid oblique to the roadway alignment, miter and skew end-cuts for the sole purpose of fitting embankment slopes or paralleling the roadway centerline are not recommended (Fig. 13). Uncut projections are not generally unsightly. Miter cuts make future extensions expensive, saving in cost is negligible; structurally the pipe may be weakened unless mitered end is adequately reinforced with rods.

OUTLETS AND ENDWALLS

The outfall of a culvert is functionally the antithesis of its approach. One is an accelerating transition channel, the other a decelerating transition. One accumulates potential energy in its forebay and transforms it to kinetic energy; the other must dispose of the excess energy by dissipation or transformation. The entrance is usually an artificial, permanent control for the stream channel; hydraulics at the outlet depends upon downstream controls, usually natural and often unstable.

Symmetry Unnecessary

Constructing the outfall identically the same as the approach does not satisfy the reversed conditions, but it was found that a large proportion of culvert appurtenances were designed in just this way. Occasionally the symmetry of upstream and downstream protection appears as ridiculous as would a debris barrier at outlet to match one at entrance.

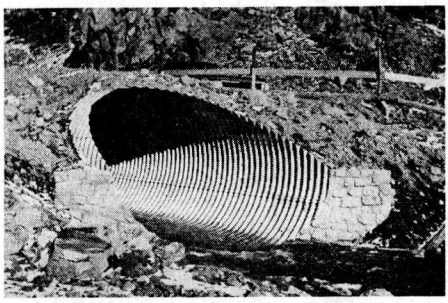


Figure 13. Entrance to Multiplate Culvert With Skew and Miter Cut. There is no general justification for either skew or miter cuts for metal pipe or multiplate culvert.

However, more recent construction showed a rational trend in the design of outfall works. Performance of these newer designs was observed for this paper, although many have not yet experienced extreme floods.

Design Variables

At many sites, there will be natural security of culvert outfall because of some combination of small discharge, low velocity, maturity of channel or durability of channel perimeter. For others, the design of outfall works must consider the following variables:

1. The energy of effluent. This may be expressed as energy head above mean elevation of invert. If more than one-third of this energy head is kinetic, then velocity is supercritical and the momentum equation will

govern in the outfall transition. On the sketch of typical profiles (Fig. 14) the plus signs above vectors indicate supercritical (shooting) and minus signs, subcritical (streaming) flow. These terms derive from the accepted conception of "critical flow,"

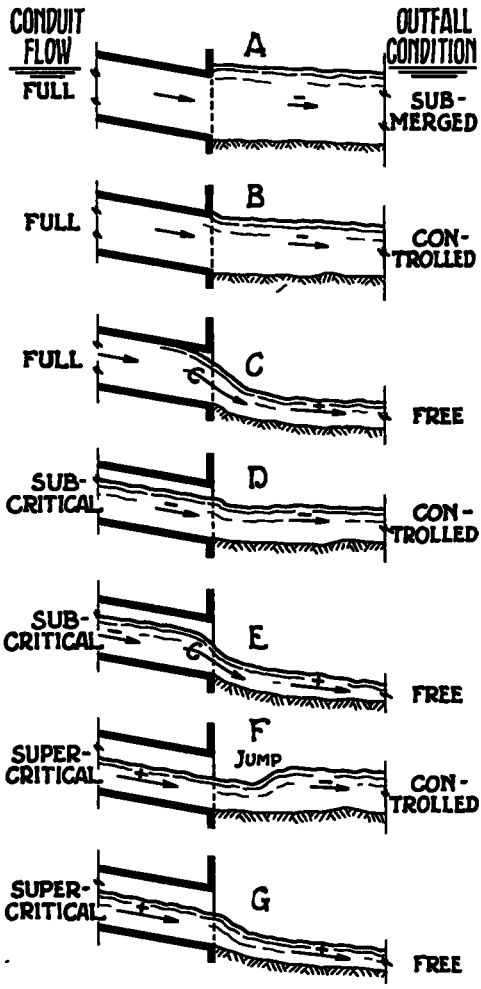


Figure 14. Typical Combinations of Conduit Flow and Get-away

when total energy is a minimum and mean depth is double the velocity head.

2. The get-away of downstream channel. In Figure 14 three conditions are recognized—submerged, controlled, and free—and the influence of these conditions is shown qualitatively. If get-away is poor, the outlet will be submerged and flow full, so transition velocities will be slow or moderate If get-away is good, the outlet will be free and high velocity will be maintained or developed For intermediate get-away, action of the transition must be studied to distinguish between accelerating flow (Case B) and the hydraulic jump (Case F)

3. The security of bed against scour Bed scour may be expected at any free outlet, but not necessarily at ends of all steep culverts. In Case F, for instance, the jump may occur on the apron or within the culvert conduit; downstream from the jump, velocity should be lower than the ruling velocity of the stream—hence endurable by the bed. Durability of the bed depends on the hardness and consolidation of an erosion surface or on coarseness of overlying products of erosion.

4. The security of banks against scour. Banks may expose strata of diverse durability. Scour is most serious if the softer layers are near mid-depth, when upper layers will be undercut. Soft layers near bed level will be intermittently protected by talus from above Scour may progress from direct attack of oblique flow in the expanding transition, or by eddy action if the transition is too rapid.

5 The future control of stream flow. For a channel of nearly constant width, elevations of rock ledges or coarse bars will control stages for some distance upstream If such are lacking, the bed level is probably unstable and the trend (scour or deposit) should be determined Submerged outlets may become free, and vice versa. If width of channel is far from constant, the control may be a constricted section Riparian vegetation is a potent factor, but permanence should be questioned.

Free Outlet Transition

The typical free-outlet culvert is about half as wide as the natural channel As shown in Figure 14, C, E, and G, the water surface must drop because of freedom This drop will increase velocity and reduce area of wetted section, hence depth must be less than half that in the culvert Since the energy equation (Bernoulli's theorem) governs accelerating flow, the reduction in depth and potential energy must be compensated by an increase in kinetic energy and velocity.

The compensation is not complete because

of turbulence, eddies, and boundary friction—amounting ordinarily to 20 per cent of the change in velocity head. (In efficient transitions, this can be greatly reduced, but an inefficient transition is more desirable for culverts.)

Figure 15 has been drawn as a guide for estimating apron depth and velocity from outlet depth and velocity, the relations depending upon the cubic equation shown. The curves at the left give the depth over the apron for three depths at outlet and the three curves at the right the corresponding velocity. Each set is divided by a line representing critical flow at the culvert outlet. Above this line, all flow is shooting and the curves are reliable. Below, the flow is streaming as it leaves the culvert, contracts to critical flow with little loss of energy, and then becomes shooting. Probably the aggregate loss of energy will be less than assumed, so that velocity will be greater and depth less than indicated by the dashed portion of the curves.

As an example of the use of this guide, suppose that culvert effluent has been computed at 15 ft. per sec. for a depth of 3 ft. Then on the apron (if twice as wide as culvert) velocity will increase to 17.8 ft. per sec. and depth will be only 1.26 ft.

In particular, if the effluent is just critical, the velocity will increase 44.6 per cent. Percentage increase will be more for streaming flow and less for shooting flow. Not shown on the curves but deductible from the same premises is the fact that critical effluents minimize apron velocities. In the example, had the same discharge been streaming, say 5 ft. deep at 9 ft. per sec., apron velocity would have been 16.6 ft. per sec. But if just critical (effluent velocity 11.36) apron velocity would have been 16.4 ft. per sec.

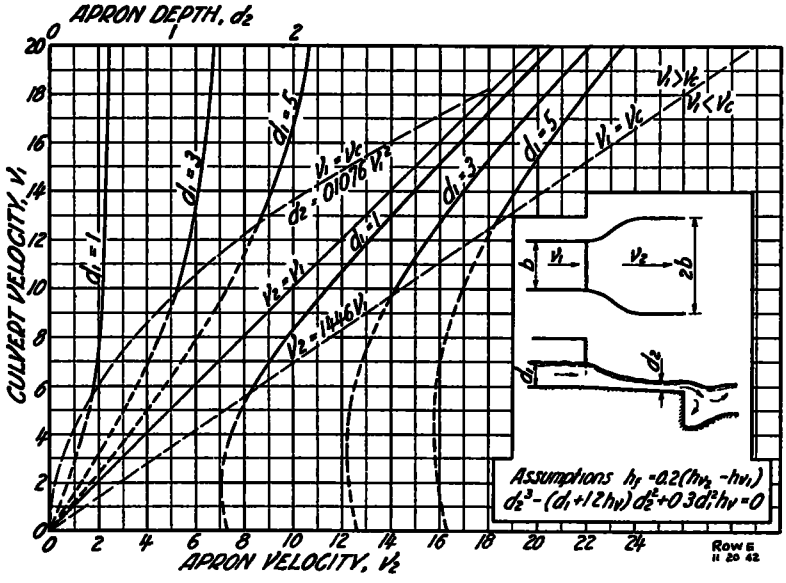


Figure 15. Guide Chart For Estimating Free-outfall Depth and Velocity on Apron from Depth and Velocity at Culvert Outlet

Free-Drop Outfall

As an extreme, the free outfall may be a free drop, as was illustrated (Fig. 5) for modified bottom, sidehill, and top locations; for these, the foregoing transition computation is not applicable. The scouring power of free drops is well understood, but experience (Fig. 16) has taught that the effluent trajectory must clear the embankment slope by a safe margin or the embankment will slough into the tailwater pool. The critical trajectory will be that for a small discharge.

Also, an ordinary free outfall may become a free drop by degradation of an unstable channel. Even rock sections may be eroded rapidly by boulder-loaded streams accelerated

by long culverts While it may be economy to add outfall works after such conditions develop it would be prudent to allow for this contingency in cost comparisons of alternative designs

Controlled Outfall Transitions

Future conditions are most uncertain if the downstream channel is naturally controlled at some point beyond the right-of-way line The control may have been naturally unstable, or a physiographical balance may be upset by the culvert. Such an upset may

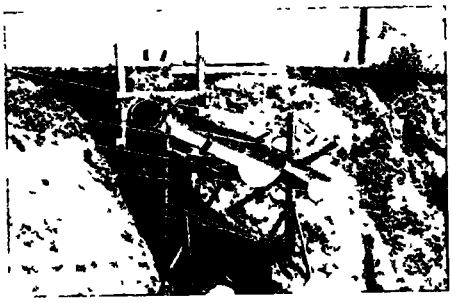


Figure 16. Projecting Culvert with Temporary Extension Added After Serious Loss of Embankment.

result from the acceleration of flow, change in channel alignment or grade, or modification of detrital loads

Since future change is difficult of prediction, it is not sound economy to provide expensive works to guard against all contingencies However, initial works or maintenance should assure against damaging alteration through property of others, or sudden loss of highway structure or embankment

Obviously the best assurance is construction of a transition which will discharge the flow at all stages just as the former channel did Granting that such provision is rarely possible or economical, it is a matter of judgment to determine the tolerable departure As a rule, bed scour is less serious than lateral erosion The former leads to a free outfall and can be corrected by maintenance betterment of the outlet structure, but lateral erosion may be progressive downstream, so that modification of outlet works can not provide a remedy

Endwalls Influence Lateral Scour

Endwalls serve the dual purpose of retaining the embankment and limiting the transition Older designs, such as the straight or flared walls (Fig 17 B, C) were economical retainers but poor transitions. Use of these

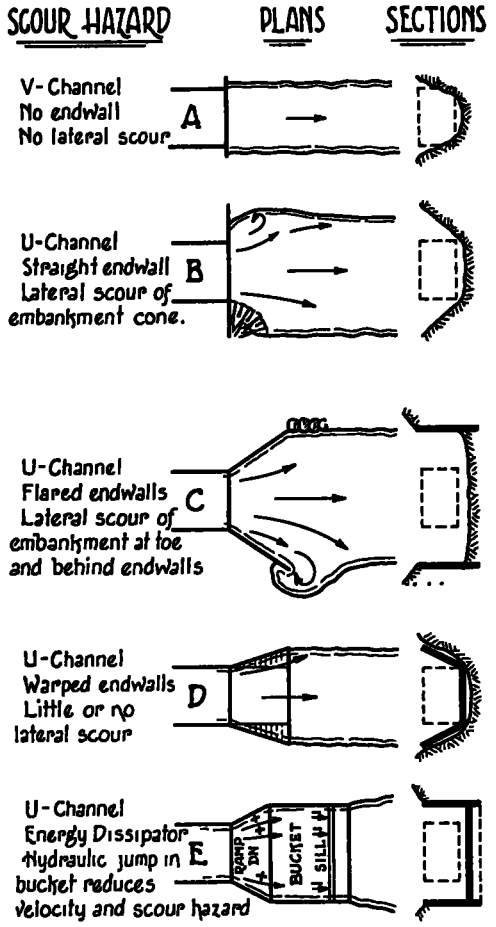


Figure 17. Influence of Transition Shape on Hazard of Lateral Scour

below small or submerged outlets is satisfactory, but embankment cones at ends of these walls are frequently cut at controlled outlets

Erosion of embankment toes can usually be traced to eddy action, as sketched Figure 18, shows an extreme case, where the angle of flare was too great and transition too short. Severe damage has been recurrent, during

floods of less than half the design discharge. At this site, the design discharge was 1,200 sec.-ft., anticipating moderate effluent velocity, as outfall channel seemed stable with fair controls and culvert gradient was only 0.8 per cent. Prior to 1942, the old-style high wingwalls were lost and apron was

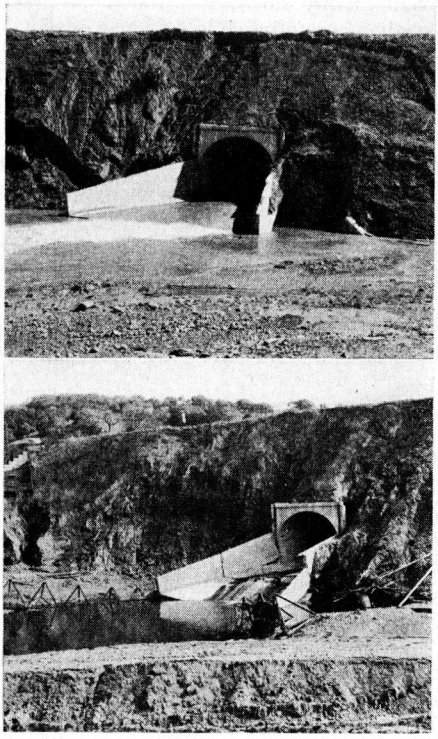


Figure 18. Upper: Nojoqui Creek Culvert in 1940 after Discharge of 640 sec.-ft.; Lower: After 2000 sec.-ft. Flood of December 1941.

undercut. Although discharge did not exceed 640 sec.-ft., effluent velocity reached 14.3 ft. per sec. The later pictures show further damage to wingwalls, apron, and uncompleted jetties in the 1,000-year storm of 1942, when discharge of 2,000 sec.-ft. created effluent velocity of 18.6 ft. per sec. At end of apron, velocity probably reached 25 ft. per sec., producing underscour and powerful lateral eddies.

Warped Endwalls

An ideal transition is a complicated ogee expansion, fitted to the variable momentum, dissipating very little energy. This is not satisfactory for a culvert outlet, where it is advantageous to reduce energy. A flared transition is very effective, if proportioned so that eddies induced by the effluent jet do not continue beyond the end of the wing or overtop a sloped wing. As a guide, it is suggested that product of velocity and flare angle should not exceed 150. That is, if effluent velocity is 5 ft. per sec., each wing may flare at 30 deg. from the thread of the stream; but if velocity is 15 ft. per sec., the flare should not exceed 10 deg.

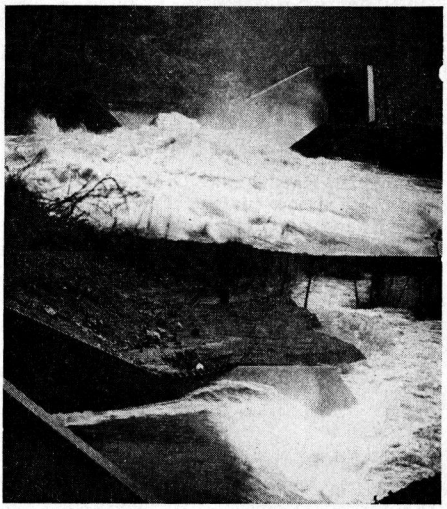


Figure 19. Upper: Bucket Outlet Deflects Salt Creek Through Angle of 70 deg.; Lower: Close-up of Salt Creek Outlet, Shooting at 22 ft. per sec.

The warped endwall (Fig. 17D) has been very successful as a transition, because it releases the flow to a trapezoidal section. Wider use is recommended, especially if the apron must be paved anyway, but it should be designed to greater length and less flare (at top) than similar walls used at culvert entrance. Even for free outlets, there will be little acceleration in this type of transition, so Figure 15 should not be used to compute apron velocity.

The energy dissipator is still in the experimental stage. Apparently successful for a free outfall followed by a sharp bend is the hydraulic bucket (Fig. 17E) adapted to the outfall of Salt Creek (Fig. 19). A free drop

qualifies in this respect; as a surge pool must follow, there is advantage in locating the drop ahead of the outlet (Fig. 20).

Cost of transition structures may be reduced by suiting the material to the velocity. Figure 21 shows warped wings of reinforced concrete followed by broken-slab riprap to a bend in the channel. On the bend, the riprap is continued on the outside only, the inside

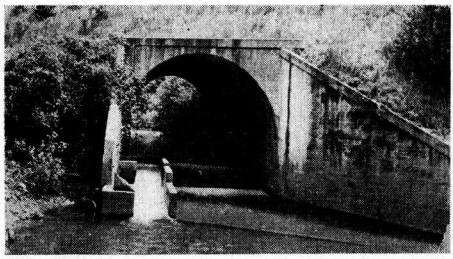


Figure 20. Rattlesnake Creek—Energy-dissipator with Drop Inside the Arch Culvert. Note fish ladder at left.

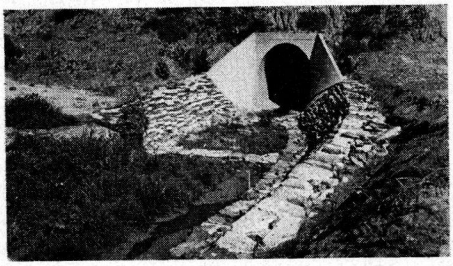


Figure 21. Warped Endwall Proved Too Short. Transition Extended by Supplemental Design.

bank being stable. This transition was developed by progressive maintenance.

Maintenance and Supplemental Design

Such developments are economic. In so many cases, the uncertainty of future channel controls will make a safe design very expensive. Rather than to anticipate the worst possible conditions, the designer may take ordinary precautions to provide reasonable security.

Subsequently, following a critical test under storm conditions, the design of the outlet works may be revised intelligently and reconstructed by maintenance forces. This procedure should be considered "supplemental design" and is not "maintenance" in the usual sense. However, the maintenance personnel are the first to discover the damage after floods, and should properly initiate the corrective work.

At such times, patchwork should be the minimum necessary to restore roadway and prevent furture damages. Conditions should be studied carefully while evidence of scour is clear. After the study, a report with recommendations should be forwarded to the designer for review, so that a supplemental design may be prepared.

For example, scoured fill cones along banks at outlet may be a warning that a large volume of embankment is threatened. If restoration and protection is confined to the slightly damaged area, the repair may prove temporary. Careful analysis of other evidence of scour might have predicted impending damage to the highway, structure, or downstream property.

CULVERT PRACTICE

Conditions encountered in culvert installation are often different from those assumed in the design, which requires the field engineer to make quick decisions and exercise independent judgment to avoid unnecessary delay. On this account presentation of the basic principles of earth pressures transmitted to culverts and the effect of various earth loadings on either flexible or rigid structures upon yielding and unyielding foundations should prove helpful to the field engineer.

Basic Principles of Earth Pressures On Culverts

The laws of mechanics and experiments on culvert pipes indicate (11, 12) that the vertical earth pressure on a culvert varies according to the relative deflections of the top of the culvert and the adjacent soil each side—ratio of e to E (Fig. 22).

Figure 22 (left) illustrates a rigid culvert on an unyielding foundation. In this case (assuming a fairly high fill) the earth alongside the culvert moves downward relative to the material in the prism over the culvert. This action causes part of the weight of the outside material to be transferred to the prism over the culvert, and E is greater than e. The pressure on the culvert will exceed the weight of the earth prism over the culvert.

Figure 22 (center) illustrates a culvert which settles or deflects downward, an amount just equal to the settlement of the plane of material originally level with the top of the culvert. The load is materially reduced, and E may approximate or equal e. The pressure on the culvert will approximate or just equal the weight of the vertical earth prism above the culvert

Figure 22 (right) illustrates a flexible culvert on a yielding foundation In this case, again assuming a fairly high fill, the earth prism directly over the structure settles downward relative to the material alongside due to appreciable shortening of the vertical diameter of the flexible culvert and by settlement of the bottom into a yielding subgrade E is less than e The pressure will be less than the weight of the earth prism over the culvert

The qualities "flexible" and "rigid" have been used in a general sense, but Marston (10) has defined a flexible culvert as one "whose cross-sectional shape can be distorted sufficiently to change its vertical or horizontal dimensions more than 3 per cent before causing materially injurious cracks", and a rigid culvert as one whose cross-sectional shape cannot be distorted enough to change its vertical or horizontal dimenstions more than 0 1 per cent without causing materially injurious cracks.

In every embankment sufficiently high, there is a horizontal plane at and above which the compression of the prism over the culvert just equals the compression of the materials alongside (see Fig. 22) This plane is known as the "plane of equal settlement" (12) and its height above the top of the culvert is the "height of equal settlement" The earth loading on a culvert is greatly affected by the "height of equal settlement" which may be above or below the roadway surface (11).

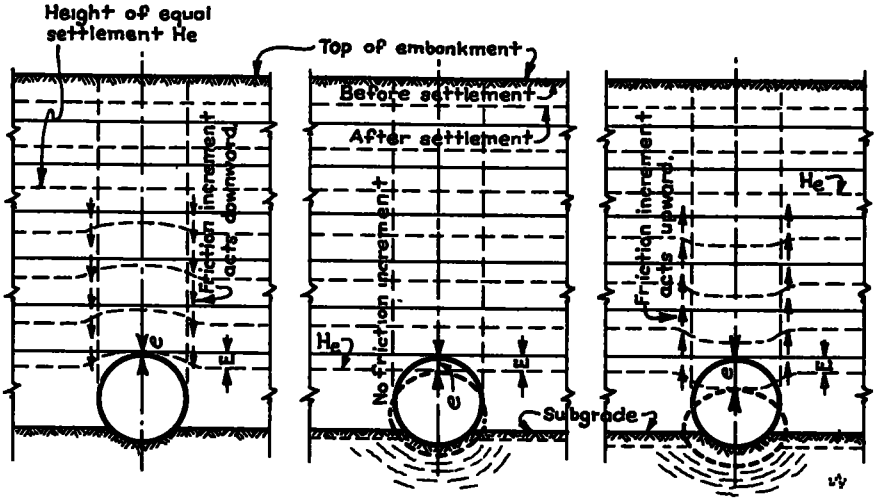


Figure 22. Three Commonly Encountered Culvert Installations Each Carrying Widely Different Loads From Same Height of Cover Material. Solid horizontal lines represent imaginary planes of embankment before settlement. Dashed lines represent the same planes after settlement has taken place.

Basic Principles Applied to Practical Installations

Eight standard cases and two special cases commonly encountered in field installations are presented for illustrative purposes

Case I (Fig 23a), Flexible Culvert in Trench on an Unyielding Foundation The backfill prism over the top of the culvert in trench tends to move downward relative to the earth alongside, and through frictional resistance and cohesion transfers part of its weight to the soil adjacent to the pipe In this case the frictional resistance increment acts upward relieving part of the weight of the vertical earth prism over the culvert

If the culvert is of the flexible type further relief of load occurs due to the ability of a flexible pipe to shorten its vertical diameter and lengthen its horizontal diameter.

Case II (Fig 23b), Rigid Culvert in Trench on an Unyielding Foundation Action of the backfill prism over the culvert is the same as in Case I, but appreciable distortion of the vertical diameter does not occur, resulting in a greater load being carried by the pipe.

As in the case of a flexible culvert on excessively yielding subgrades, a highly compacted cushion may be desirable to insure against excessive displacement

Case V (Fig 24e), Flexible Culvert on Unyielding Embankment Subgrade In opposition to Case I, there is a tendency for the earth alongside the culvert to move downward relative to the vertical prism over the culvert, causing the frictional increment to act downward and transfer load to the vertical earth prism over the structure This is based on the assumption that the materials are homogeneous and that the column of earth adjacent to the culvert moves through a greater height than the column directly

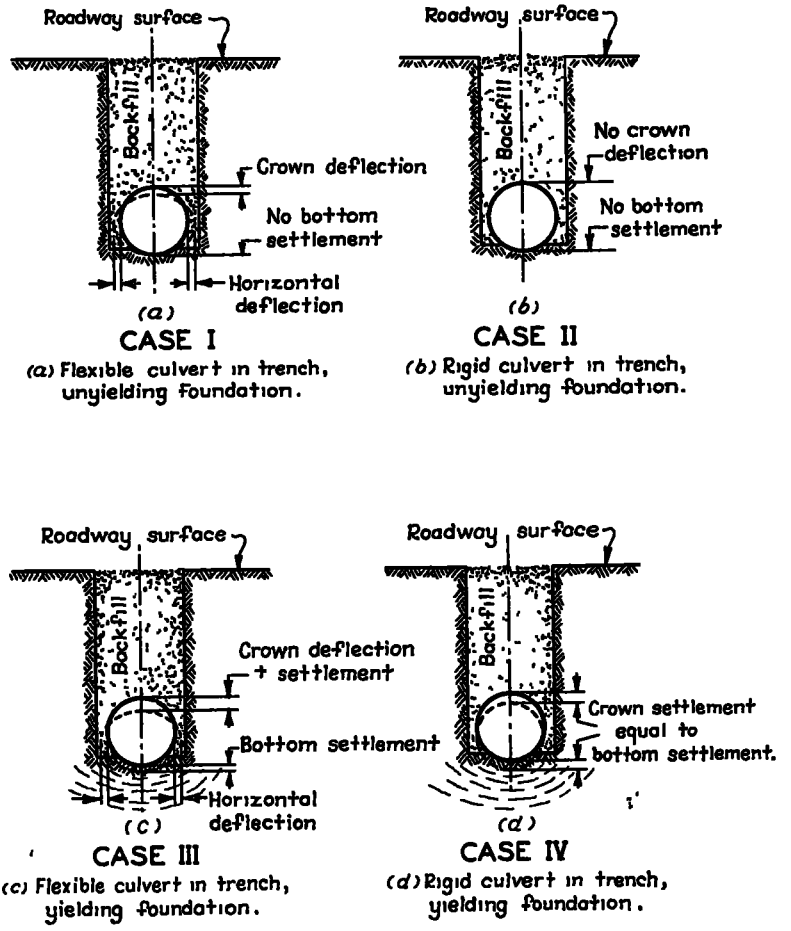


Figure 23. Four Cases of Culverts Installed in Trenches

Case III (Fig 23c), Flexible Culvert in Trench on Yielding Foundation The same conditions prevail as in Case I, with a third added The settlement of the culvert on a yielding subgrade further tends to relieve the pressure

Case IV (Fig 23d), Rigid Culvert in Trench on Yielding Foundation Settlement of the yielding subgrade or cushion tends to relieve the pressure on the culvert.

over it As in Case I, some relief of load occurs when the flexible structure deflects

Case VI (Fig 24f), Rigid Culvert on Unyielding Embankment Subgrade The conditions of Case V apply except that the rigid culvert does not deflect sufficiently to relieve further relief of load may result by yielding of the supporting foundation The load is greater than Case III, other things being equal

Case VIII (Fig 24h), Rigid Culvert on Yielding Embankment Subgrade Case VI

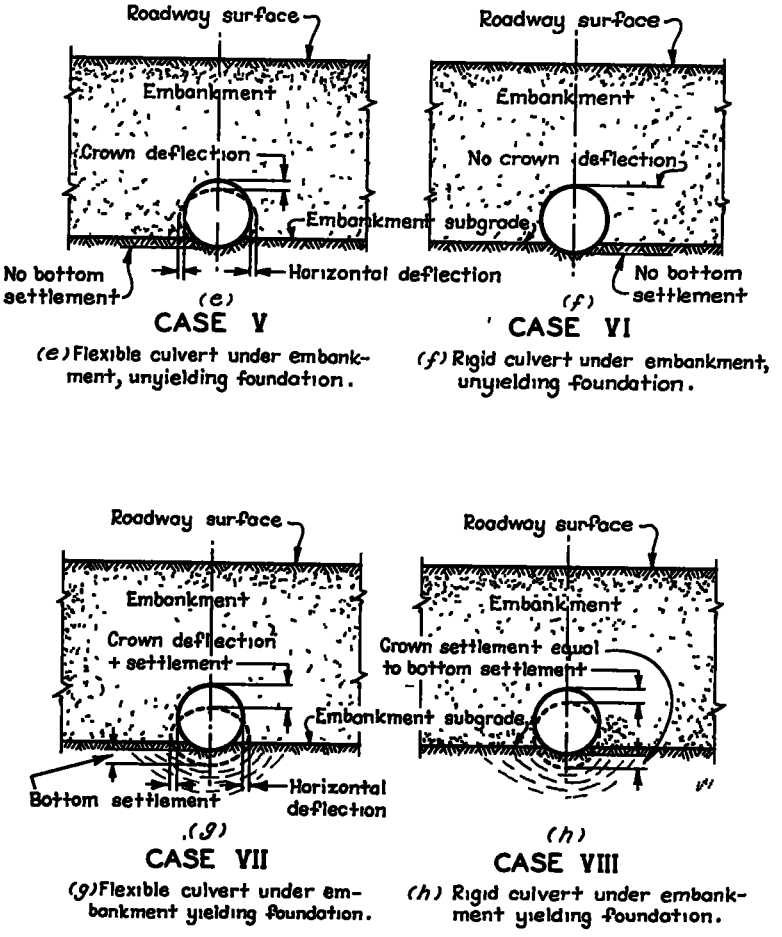


Figure 24. Four Cases of Culverts Installed on Original Embankment Subgrade

appreciably the vertical load. It is the most severe earth-loading condition encountered in the eight standard cases cited Consideration should be given to some method of relieving the load such as by placing a yielding cushion under the culvert or increasing its structural strength or both

Case VII (Fig 24g), Flexible Culvert on Yielding Embankment Subgrade The same conditions apply as in Case V except that applies except that relief of the vertical load results from yielding of the foundation support

Cradles may be desirable in extreme cases. Tests indicate (13) that supporting cradles under concrete pipe will develop supporting strength from 1½ to 2 times that which the pipe developments when not cradled, but this increase in supporting value may be entirely neutralized if the cradles act to reduce the

amount of bottom settlement that would normally occur The load is greater than Case IV.

Case IX (Fig 25i) Wrong Method and Case X (Fig 25j) Right Method of Side Hill Installation California experience with culverts placed in the topsoil stratum of sidehill location has proven to be highly disastrous (Fig 25i). Culverts so installed are subject to a shearing action that takes place in the topsoil stratum between the settling embankment and the firm material back of the overburden Culverts have been completely collapsed in many sidehill locations Figure 25j indicates the recommended installation

bottom and permitting settlement which relieves the load as described in Cases I, II, V, and VI

California 1940 Standard Specifications provide for a cushion as follows (Sec. 56e): "Where solid rock is encountered, it shall be removed below grade and the trench backfilled with suitable material in such a manner as to provide a compacted earth cushion with a thickness under the pipe of not less than ½ in. per foot of height of fill over the top of the pipe, with a minimum allowable thickness of 8 in."

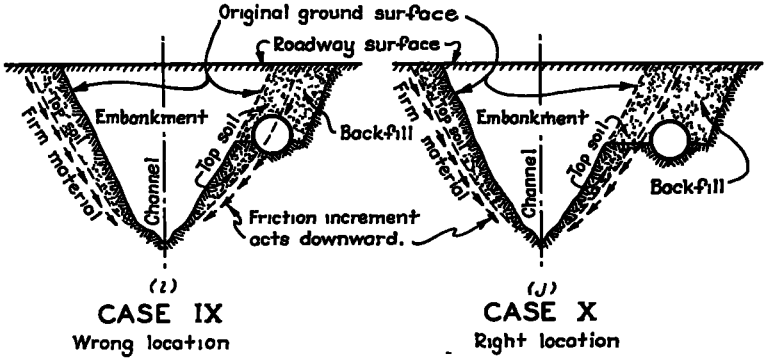


Figure 25. Right and Wrong Way of Installing Culverts in Sidehill Location

Field Control

Flexible and rigid pipe culverts are designed to resist average earth pressures expected from certain maximum heights of fills, but varied conditions are continually encountered including rock foundations, marshy ground, unsatisfactory backfill material, caving trenches and many conditions too numerous to list It is the function of the field engineer to interpret the effects of these various conditions and decide whether the proposed culvert will be subjected to greater or less pressures than the average condition for which the culvert is designed and make such changes in the installation as may be necessary to insure structural stability.

Unyielding Foundations Unyielding foundations vary from solid rock to dry hardpan. A cushion placed under a culvert on unyielding foundation serves the dual purpose of insuring uniform distribution of pressure on the

Compaction should be uniform, ranging from a fairly high degree under low fills (to limit undesirable settlement in the roadway surface) to a low degree of compaction under high fills (to permit settlement).

Bedding and Backfill Adequate bedding for pipe culverts can not be obtained uniformly by specifying that the trench bottom be shaped to fit the bottom of the pipe.

This specification is difficult to enforce as years of experience on California highways have shown The bottom rounding specification was removed from the California specifications in 1940 and provision made for compaction of backfill from the bottom of the trench in thin layers, with the option of ponding or jetting granular material in lieu of thin highly compacted layers

The same difficulty in obtaining proper backfill has been experienced as in the case of rounding the culvert trench to fit the pipes. Backfills compacted to 90 per cent relative compaction are exceedingly difficult to obtain where hand tamping is used unless layers of

backfill are tamped in 2-in. or 3-in layers at optimum moisture content and with a sufficiently small tamping head Hand-tamping processes are tedious and rigorous inspection is required to insure specified results The same applies to mechanical tampers Unless the tamping head is sufficiently small and the workman applies his weight to the tamper to keep it from bouncing too much, which he is not prone to do, poor compaction results

Yielding Foundations Yielding foundations vary from marshy ground containing a high percentage of moisture to low-density spongy topsoil. In order to insure uniform distribution of pressure on the bottom and sides of a culvert on yielding foundation a layer of gravel or other material of high bearing value should be placed under and on the sides of the structure

California 1940 Standard Specifications provide for yielding foundation conditions as follows (Sec. 57e)·

"Where a firm foundation is not encountered due to soft, spongy, or other unsuitable material, all of such unsuitable material under the pipe and for a width of not less than one diameter on each side of the pipe shall be removed and the space filled with gravel or other suitable material".

Thickness of gravel support is dependent upon the width of structure and nature of supporting subgrade.

Bedding and Backfill Specifications

Replacing the old rounding-of-the-bottom specifications in California is a provision for placing backfill under and around pipe culverts in compacted layers, containing optimum moisture with an in-lieu specification permitting use of ponded or jetted granular materials.

Section 121 of the 1940 Standard Specifications provides as follows:

Backfill Specification. "Backfill shall be placed in horizontal layers not exceeding 4 in in depth before compaction Each layer shall be moistened and thoroughly tamped, puddled, rolled, or otherwise compacted until the relative compaction is not less than 90 per cent as determined by the compaction test specified in Section 6, Article (d) of these specifications."

In-Lieu Specification "Should the contractor elect to furnish sandy or granular material for backfill, the layer construction may be eliminated and compaction obtained by ponding or jetting Ponding or jetting will not be permitted where the backfill material is not of a sandy nature nor where the foundation material is such that it will soften when saturated. . ."

Culverts in Trenches

Test results (18) indicate that, as width of trench increases, other conditions remaining constant, the load upon the culvert increases, until projection condition is reached. Although there is no definite specification limiting trench width, excessive width is discouraged Specifications disallow payment for structure excavation and backfill outside vertical surfaces 1 ft each side of the external dimensions of pipes or 1 ft. outside the neat lines of concrete structure footings.

Culverts on Subgrade under Embankment

As stated in Cases V, VI, VII, and VIII, for projection condition, culverts installed on embankment subgrades generally sustain more earth load than when installed in trenches, as outlined in Cases I, II, III, and IV.

To reduce the load transmitted to culverts under new embankments, specifications provide for constructing the fill to above the top of the culvert and then excavating a trench

California 1940 Standard Specifications accomplish this as follows (Sec. 57e)

"In the case of pipes 24 in or less in diameter, the roadway embankment shall be constructed to an elevation 6 in above the grade proposed for the top of the pipe, after which the trench shall be excavated and the pipe installed.

"In the case of pipes more than 24 in in diameter, up to and including pipes 90 in in diameter, the roadway embankment shall be constructed to an elevation 30 in above the grade proposed for the bottom of the pipe, after which the trench shall be excavated and the pipe installed

"In the case of pipes more than 90 in in diameter, the roadway embankment shall be constructed to the elevation of the third point of the diameter of the pipe (measured from the grade line proposed for the bottom of the pipe) after which the trench shall be excavated and the pipe installed "

A frequent practice adopted by contractors to comply with this specification consists in building a mound and then excavating a trench in the mound (Fig. 26). This should be discouraged since it tends to defeat the purpose of the specification. It is recommended that construction of compacted embankment, as provided for in the foregoing specification, be required for at least five diameters each side of the proposed installation before trench excavation is made.

Structures projecting above the surface of the embankment subgrade should be backfilled evenly on both sides.

Figure 26. Rigid Culvert Placed on Embankment Subgrade and Covered with Mound of Earth. Poor Practice.

Recommendations

It is concluded that the present height-of-fill limitations for the various types of culverts are not sufficiently flexible to be economically adapted to the various field conditions encountered, and that there is need of a study to establish limiting heights-of-fill for various loading conditions for flexible and rigid structures that take into consideration the effects of highly consolidated modern highway embankments and high-compaction backfill.

DEBRIS

Congestion of drift, debris, and detritus at culvert entrances with resultant impairment of culvert efficiency presents one of the outstanding problems in California.

Field Observations

It was not until 26 to 28 yrs. after the inauguration of the State Highway System in California that widespread storms of extreme intensity occurred so as to focus attention on the importance of controlling debris.

A study was made of experimental debris control measures at scattered locations throughout the State. Significant was the fact that practically all types, wherever encountered, were generally effective regardless of type and suitability. These observations led to classification of the various types of debris barriers and determination of adaptability to various conditions found in the field.

The following recommendations are intended as a guide for selecting debris control measures in accordance with preconceived basic principles.

The sucess of various experimental types installed within the last three years is encouraging, and it is hoped that new types and adaptations will be developed from further experience.

The debris deflector is a V-shaped barrier adapted to diversion or deflection of heavy floating debris and large boulders or rocks carried as a bed-load in moderate to high-velocity flows often encountered in mountainous terrain (Fig. 27). It is particularly useful at the entrances to large culverts and should be of heavy construction with the vertex of the "V" placed upstream. The vertex may be vertical or inclined. Horizontal spacing of vertical members of the deflector should not exceed the horizontal diameter of the culvert. Length of the "V" generally should be not less than three to four times the horizontal diameter of the culvert. Like all other types of debris-separating devices, the debris deflector requires periodic or seasonal removal of accumulated material.

The debris rack, in its various forms, is essentially a straight rack placed across the path of a defined channel for the purpose of screening floating debris from the stream flow and preventing it from reaching the culvert entrance at times of momentary flood peaks. The rack may be vertical or inclined. If inclined, it may serve some of the purposes of a deflector (Fig. 28). Experience with the debris rack indicates that a spacing of vertical bars equal to one-half the horizontal diameter of the culvert is satisfactory. This spacing of vertical bars permits light debris to pass the rack endwise. If the channel is well defined, the debris rack should be placed

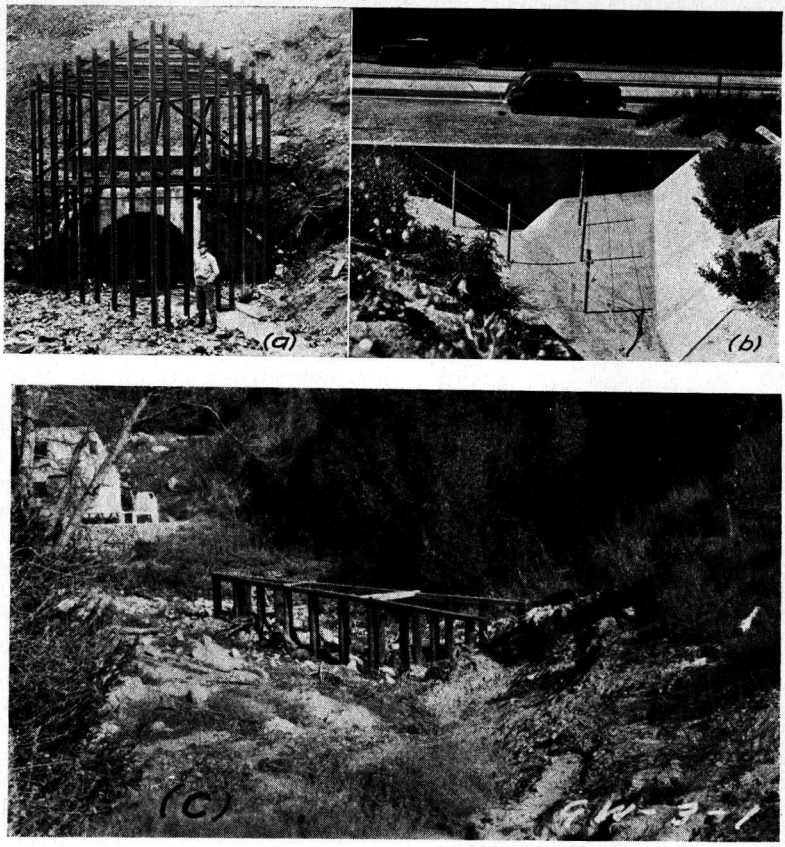


Figure 27. (a) Steel Rail Debris Deflector at Entrance to a Large Concrete Arch Culvert to Deflect Heavy Debris. (b) Wire and Pipe Debris Deflector at Entrance to R. C. P. Culvert to Deflect Light Floating Debris. Note drop-down approach to culvert entrance. (c) Wooden Pile Debris Deflector at Entrance to a Large Culvert for Purpose of Deflecting Bed-Load of Large Boulders and Heavy Floating Debris.

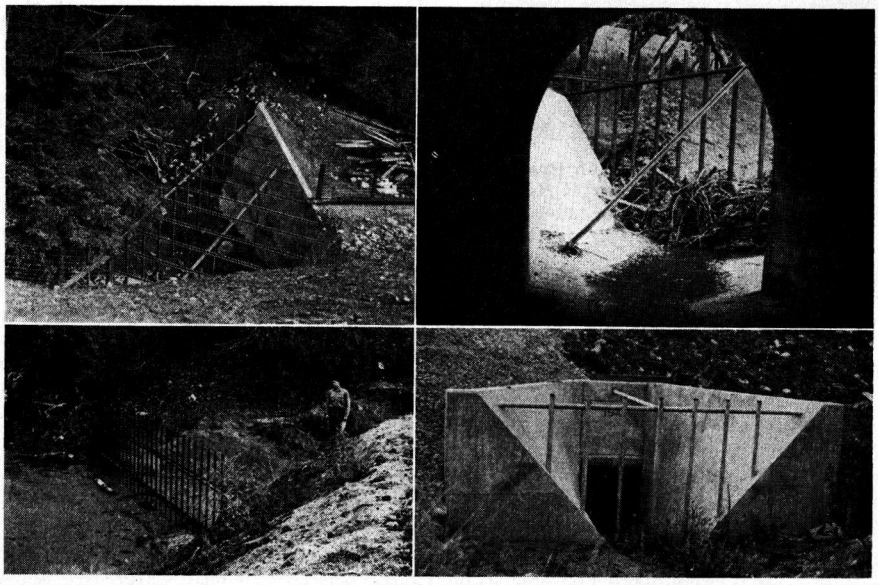


Figure 28. Upper left: Steel Debris Rack at End of Wingwalls to Large Reinforced Concrete Arch Culvert. Upper Right: Steel Rail Debris Rack at End of Wingwalls to Reinforced Concrete Arch Culvert. Lower Left: Steel Rail Debris Rack Placed in Channel Well Above a Culvert Entrance. Lower Right: Pipe Debris Rack at End of Wingwalls to Reinforced Concrete Culvert. Note warped wingwalls and curved-lip crown.

well upstream from the culvert entrance. If limited by right of way or channel shape, it may be installed at the head of the wings, in which case the rack should be as high as the culvert parapet.

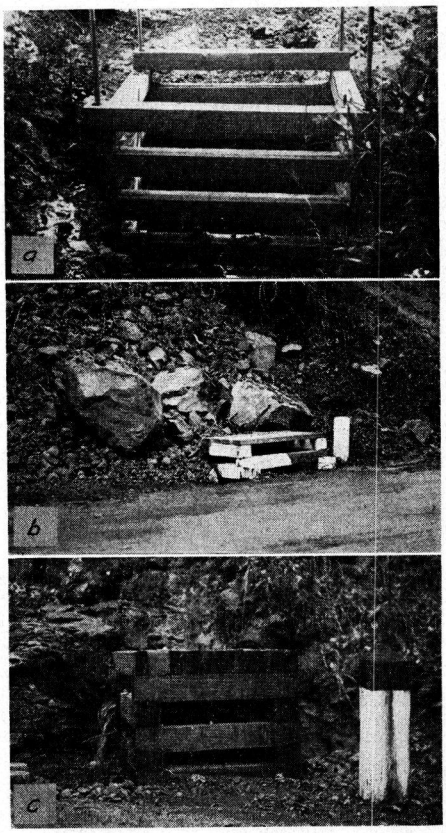


Figure 29. (a) Debris Crib of Reinforced Concrete with Provision for Vertical Extension Placed Over Culvert Entrance in Small Debris Basin and Commonly Known as a "Bear Trap." (b) Timber Debris Crib Installed Over Culvert Entrance in Roadway Gutter. Note how debris has been prevented from blocking culvert entrance. (c) Debris Crib of Hewn Timber with a Solid Cover Placed Over Culvert Entrance in Roadway Gutter.

The debris crib, commonly called a "bear trap", has been successfully developed for locations where abrupt grade breaks cause deposition of detritus and clogging of the culvert entrance. For many small culverts at or near the roadbed level, the abrupt break in grade can not be avoided. The open type of debris crib is built of either timber, metal, or concrete with or without a cover and is placed directly over the culvert entrance. So placed, with a cover, debris can practically envelop the crib without completely shutting off the entry of water (Fig. 29b). Where the open crib type is used as a riser in anticipation of a considerable depth of detritus, as in a debris basin, it should be built well above the estimated height of deposit on the culvert entrance, with provision for further increase in height as required (Fig. 29a).

Debris Riser. In mountainous terrain with high embankments, a common drainage practice requires the location of a culvert in the bottom of a waterway, which places the culvert entrance in the lowest part of a debris basin susceptible to rapid filling with flowing detritus. At such locations it is essential that the muck be kept from entering and clogging the culvert entrance. A successful solution to this problem includes a perforated vertical riser of pipe, timber, or concrete, placed directly over the culvert entrance. Perforations should be large enough to permit entry of water and small enough to exclude muck. The riser or chimney should be carried well above anticipated deposit and increased in height as necessary (Fig. 30).

Debris Basin. In certain mountainous areas of easily eroded materials, particularly granitic materials, with steep slopes and heavy run-off, it has been found economically impractical to provide a culvert large enough to carry safely surges of flowing detritus. The perforated debris riser as an expedient prevents clogging of the culvert entrance at a critical period when the debris basin is filling with a surging heterogeneous mass of muck, rock, and debris from surface accumulations. The basin simply acts as a reservoir to store the flowing detritus until it can be dewatered by the perforated debris riser and later removed. Under certain conditions, the debris riser may be built up progressively, forming an extensive debris table. This will be self-sustaining to a greater extent year by year, finally depositing its load at a safe distance above the culvert entrance.

Debris Spillway. The most serious losses of embankment occurred in mountainous areas throughout the State where debris basins proved inadequate. Complete washouts occurred where water overflowed the

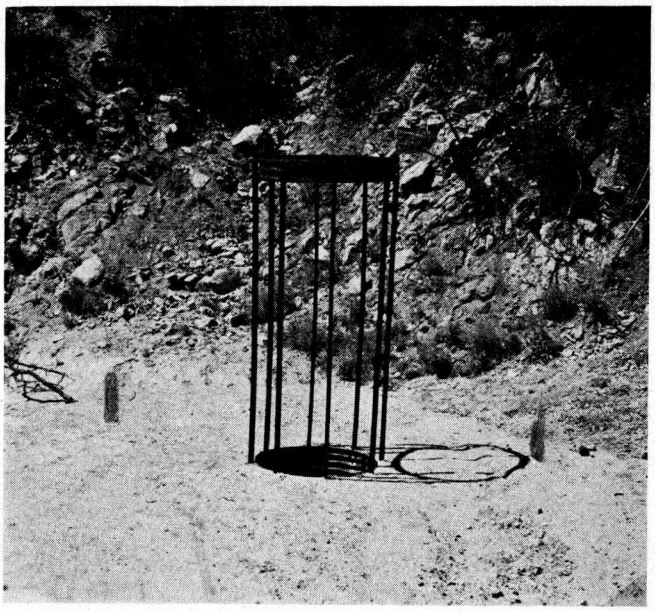


Figure 30. Metal Debris Riser Placed Over Culvert Entrance in Debris Basin. Note provision for extension when debris basin has filled to such an extent as to cause deposition of most of the detritus before it reaches the riser.

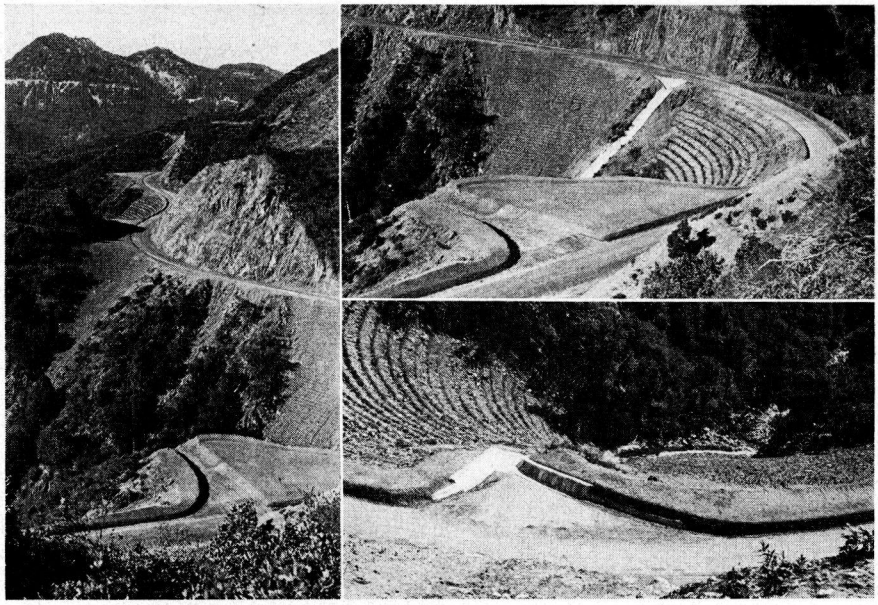


Figure 31. Left: Debris Spillway Entrance on Rock Slope in Foreground. Lower right: Lined Debris Spillway Entrance. Upper right: Typical Debris Basin Spillway in Mountainous Terrain Above Highway Lined Spillway in Background and Unlined Spillway in Foreground.

roadway embankment. The debris spillway, as an adjunct to the debris basin, is considered to be one of the foremost improvements in California drainage practice developed subsequent to the 1938–40 storms—notably on the Angeles Crest Highway east of La Canada in Los Angeles County (Fig. 31).

As pointed out previously a storm of 100-yr. frequency strikes some spot nearly every year, and when it does occur, the damage is usually serious The expense of providing against heavy damage from even the 100-yr. frequency storm is relatively small In the combination debris basin-spillway the debris basin is provided with a culvert and riser for normal drainage, supplemented by auxiliary drainage facilities Flowage into the roadbed from an overtaxed debris basin is confined inside the shoulder dyke so as to flow down the channelized roadbed until a safe and natural place is found to spill

Debris basin-spillways should be lined to resist erosion In many cases on grades, spillways may be several hundred feet from the debris basin where the flow originated Dykes and spillways, to be successful, must be constructed on a large scale

Recommendations

These observations have led to recommendations that·

1 Culvert sites be classified for debris characteristics at the time of the preliminary survey

2 Debris control measures be considered an essential part of culvert design

3 Debris barrier types be selected and adapted in accord with the general principles outlined herein

4 Debris control devices be cleaned and maintained regularly.

SIPHONS

In ancient times, engineers found that gravity-borne water could be led over depressions by pressure conduits connecting their grade-tour canals, supplanting more expensive works, such as a circuitous canal or a trestled flume Unfortunately, engineers were less inventive of words than of works, for such a pressure conduit became known as an "inverted siphon "

Functionally a siphon ceases to be a siphon when inverted, so that the term was self-contradictory At that time, the true siphon was seldom, if ever, used by civil engineers, so that the expression was reduced to "siphon," a corruption which still persists. Hence, we had two "siphons,"—the true siphon of the wine sampler and the false siphon of the hydraulic engineer.

Subsequently engineers found useful applications of the true siphon,—notably in automatic spillway controls To erase the conflict, the American Society of Civil Engineers (14) recommends that false siphons be called "sag pipes " Since culverts may take either form, the Committee conformed to this recommendation, naming and defining several specific types

Sag Culvert Defined. Generally, the adjective "sag" will be used to qualify a conduit structure or portion thereof for which the flow line is depressed below a uniform grade line Depending upon its section, the conduit will be designated a "sag pipe," "sag box," or "sag arch," and structures will be called "sag-pipe culverts," etc When flowing full, the culvert crown will be below the hydraulic grade line, so no vacuum will exist (Fig 32E)

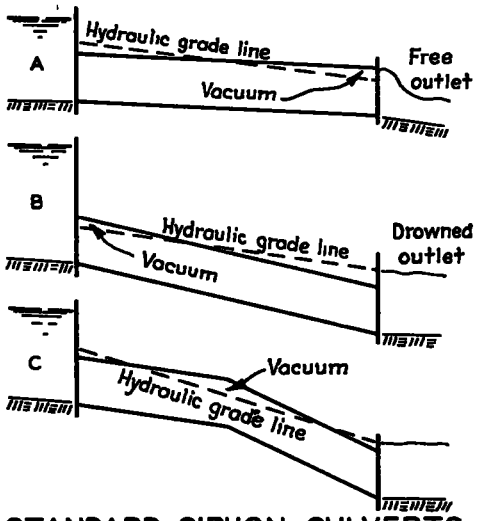
For simplicity, these terms will not be employed unless the sag is significant in the hydraulics of the structure, either because of pressure on conduit crown or adverse gradient of flow line. Thus, canal flow may pass under a highway through a "sag-box culvert," but a pipe laid on nonuniform gradient in a natural channel may be more appropriately called a "channel-grade pipe culvert."

Siphons Classified and Defined. Generally, a siphon is any conduit within which the absolute pressure, at some point or stage, falls below atmospheric At such points, the relative pressure is negative and is usually expressed in terms of equivalent "vacuum head " For culverts, the unit of vacuum head is the negative hydrostatic head in feet.

As usually pictured, the siphon has a uniform hydraulic gradient below an elevated crown line, as for the wine sampler It will be shown that standard culverts can act in this way If the siphon action is important, functionally, it will be called a "standard-siphon culvert" (Fig 32A,C).

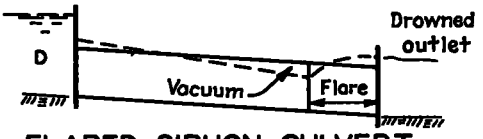
On the other hand, the siphon may have a depressed hydraulic grade line below a uniform crown line This is true, substantially,

of venturi tubes, pump intakes, draft tubes, etc. Culverts may act in this way if the



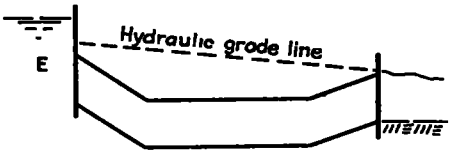
STANDARD-SIPHON CULVERTS
CONDUIT SECTION CONSTANT

A. Flow line uniform; gradient mild; outlet free; maximum vacuum at outlet.
B. Flow line uniform; gradient steep, outlet drowned; maximum vacuum at entrance.
C. Flow line on down-broken grade; maximum vacuum at break in grade.



FLARED-SIPHON CULVERT

Conduit section flared; outlet drowned; maximum vacuum at point of divergence.



SAG CULVERT

Flow line depressed; no vacuum; not a siphon.

Figure 32. Classification of Siphons and Sag Culverts

downstream end is divergent, which type we have named the "flared-siphon culvert" (Fig 32D) This, and its most economical form, the "ideal-flared-siphon culvert," will be discussed in detail.

In combination of the two principles, a siphon may have a depressed hydraulic grade line below an elevated crown line, as in the siphon spillway This combination does not appear to offer any advantage in culvert design.

Sag-Culvert Practice

Sag culverts of pipe or box section are used extensively to pass irrigation canals under highways Because of the interest of water users in maintaining an efficient section, little difficulty is ever experienced Design principles are available in many texts, but one point is frequently overlooked

Under a narrow roadbed, the initial design usually provides a constricted section so that hydraulic gradient is much steeper than for the canal as a whole. If the roadbed is widened the culvert must be extended, and the friction in the extension will increase canal stage at the entrance or reduce its capacity

The change will be greater if the widening is in the form of a divided highway with two sag culverts in series, doubling the entrance and outfall losses of head In such improvements the designer should provide generous extensions to assure against loss of capacity, using smooth-bore conduit with section 25 to 50 per cent larger than the existing section

Sag culverts are also used to minimize culvert width under low roadbeds, particularly to pass local drainage via tule sloughs These are reasonably successful, as span is minimized, stagnancy is no worse than in the sloughs, and little silt is borne by such streams Even where the bed is not paved, tule seldom impairs the waterway.

Standard culverts have become sag culverts because of general aggradation of the streambed Higher velocities through the culvert tend to maintain a fair section, but many of these are choked each year by material deposited on a falling stage The cost of cleaning these culverts is an unreasonable maintenance item.

Sag culverts should be avoided on ephemeral or intermittent streams if the consequent stagnation will be objectionable Short periods of stagnation are tolerable but long periods will be objectionable in many ways.

Standard-Siphon Culverts

Contrary to general belief, a culvert of constant section on a uniform gradient may siphon The phenomenon was demonstrated by the Iowa test (9), particularly (Plate XIX, Tests 213, 291) for smooth-bore pipes with submerged entrances and free outfalls. In the latter test, vacuum head on crown of pipe varied uniformly from 0 15 ft at entrance to 0 75 ft at outfall.

No theory has been presented to translate these experiments into design, but the problem is being investigated further (15) The possibility of siphoning should be kept in mind whenever estimates of discharge are made from stage observed above a culvert. For example, if the flow of Test 291 had occurred in the field and discharge had been computed on the erroneous premise that the culvert could not siphon, the result would have been 19 per cent under the actual discharge Since it is doubtful that extra capacity due to siphoning could be depended upon for drift-laden flow, it is not recommended that it be used in design.

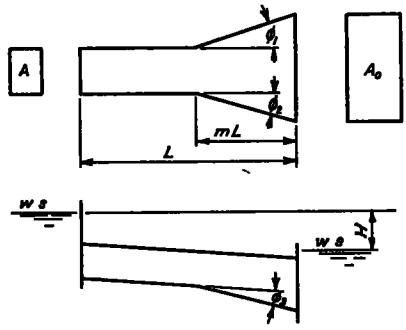
Flared-Siphon Culvert

The flared siphon (Fig. 32D) utilizes the principle of an expanding tube (venturi) to salvage a large part of the water's kinetic energy Ordinarily, the sudden enlargement at the outlet wastes 95 per cent or more of the velocity head This waste is desirable for steep channels, but for cross drainage in broad valleys, the wasted energy is reflected in damaging stages above the culvert entrance

If the flare is built to diverge on eased curves, little energy will be lost in the transition The longer the flare and the larger the outlet, the greater the salvage of energy. Thus, if outlet area is doubled, the outfall velocity is halved and kinetic energy wasted at outfall will be quartered In theory, it is possible to salvage 90, or even 99, per cent of the kinetic energy, but at a large structure cost

In practice, the flare can be built with flat instead of curved walls and with outlet from 1 5 to 2 0 times the standard section in area, so as to salvage 60 to 70 per cent of the kinetic energy For any particular site, the most economical flare dimensions will depend upon local factors,—requiring hydraulic computations For reference, this most economical design is termed the "ideal flared-siphon"

General Formula for Flared Siphons. The University of Iowa tests (9) showed the hydraulic advantages of the flared siphon,—quantitatively for particular designs and generally for similar designs For example, the



$$Q = \frac{A\sqrt{2gH}}{\sqrt{0.05 + pt(13+15t)(1-a^2) + a^2 + \frac{0.0045L}{R^{1.25}}(1 - 0.144mr)}}$$

Where
$a = A/A_o$, the ratio of areas in sq ft
P = Perimeter of standard culvert, ft
P_f = Perimeter of standard culvert sides which are to flare, ft
$p = P_f/P$
R, R_o = Hydraulic radii of areas A and A_o
$r = R_o/R$
t = Weighted mean tangent of flare angles ϕ_1, ϕ_2, ϕ_3

Figure 33. Discharge Formula for Flared-siphon Culvert

report stated that capacity of a box culvert 36 ft long could be increased about 60 per cent by flaring the downstream 10 or 12 ft so as to double the outlet area. For the general case, the writers submit a formula (Fig 33) agreeable to all test data and extended by theoretical considerations This formula is suitable for design, for the complicated expressions become quite simple when some proportions are determined arbitrarily

Fig 34 illustrates the methods of computation Suppose the 10-yr. flood had determined the size of standard section as a 2 by 2 box and it was proposed to increase the capacity by flaring to a 4 by 2 section in the last 6 ft. of a total length of 30 ft Then

$A=4$, $P=8$; $R=0.5$, $L=30$, as in usual formulae

At the outlet, the hydraulic radius is 0.667, so $r=1\ 33$ Two sides of the box will be flared, so $Pf=4$; hence $p=0.5$. Also a is the ratio of the areas, $2\times2/4\times2=0.5$, and m is the proportion of length to be flared $=\frac{6}{30}=0.2$.

The flare angle may be complicated for skew culverts or boxes flared on three or four sides The top slab should never be flared, for that raises the stage (and discharge) for priming the siphon Rarely will there be advantage in flaring the bottom slab; if a lower outfall flow line is practicable, capacity can be obtained more economically by lowering the flow line to a uniform gradient In the example, each side wall flares 1 ft in 6, so $t=0\ 167$

Substituting these simple values in the formula, the expression reduces to $Q=34.8\sqrt{H}$, where H is the difference in stage between forebay and tailwater Without the flare, the relation would be $Q=27\ 4\sqrt{H}$ Assuming a typical tailwater stage, two stage discharge curves were drawn for comparison of the flared siphon with the uniform section.

At the right of Figure 34 are profiles of the hydraulic grade line and energy line, on which are shown the relative losses in the culvert. The total head, H, has four components. The flare design has little or no effect on entrance and friction components The turbulence loss in the flare can be reduced by lengthening or curving that portion and the loss at outlet h_o depends on area of outlet section. These losses should be visualized by the designer, to avoid disproportionate loss of energy or increase in construction cost

Ideal Flare Design. In any case, the ideal flared-siphon culvert must be designed by cut and try, at least until a wider variety of governing factors have been analyzed As a guide, the area ratio a will vary from 1.5 for moderate velocities to 2 0 for high velocities. The flare-length ratio m will vary from 0.1 for long culverts under moderate fills to 0.3 for short culverts under no fill. The flare angle tangent t should not exceed 0 2 for moderate velocities or 0 1 for high velocities, or the diverging jet will not wet the outer walls (causing a gurgling turbulence as prime is intermittently lost)

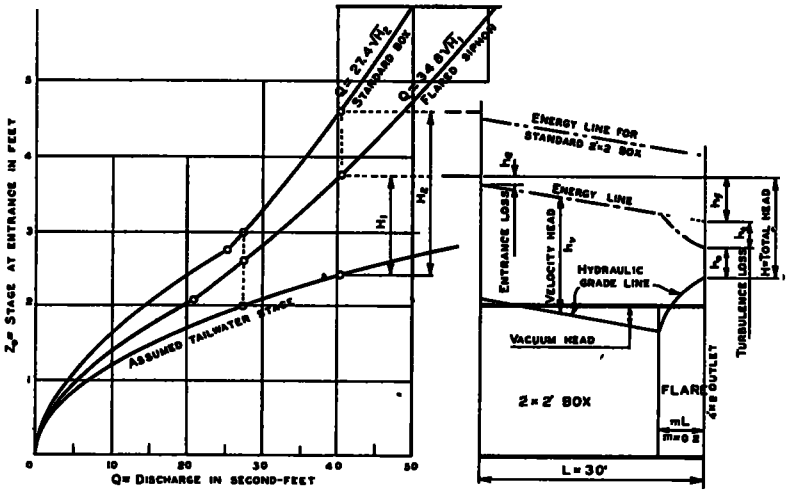


Figure 34. Hydraulic Comparison of Flared-siphon with Standard Culvert

Application of Flared Siphons The flared-siphon culvert is an ideal solution for many drainage problems The first installation in California is shown diagrammatically in Figure 35 At this site, an existing culvert had proved inadequate after a rural area had been developed into a residential suburb, crowding a stream of the meadow-overflow type Economy demanded full capacity without exceeding the damage-incidence stage. The flared siphon proved much cheaper than standard culverts of the same capacity at limiting stage.

The design is experimental in that the flare was applied to a triple box. We hope to observe whether the outer boxes are as efficient as the central box. In this case the slab over the flare had to carry the same loads as the standard section, which would not be true for a long culvert under a high fill.

The flared siphon should be considered in all widening plans in the broad valley areas, because existing culverts can be so extended as to increase their capacities Submergence of outlet is a necessary condition, of course, if submergence is not natural, it can be obtained by building a sill or weir on the apron There is great promise in this design, as otherwise the existing culvert may have to be

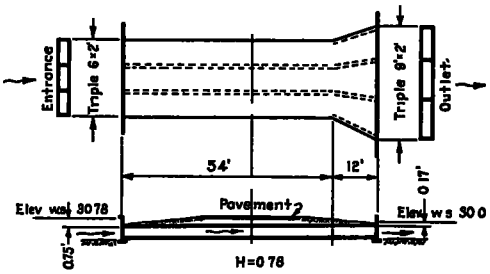


Figure 35. Layout of Experimental Vallejo Creek Triple Flared-siphon Box Culvert

reconstructed to avoid overtopping The flare extension can be added without interruption to traffic.

RATED WATERWAYS

The effect of modern standards of highway alignment and grade on culvert design has been noted, in part, before In addition to the longer and stronger conduits required by high embankments, the same trend has increased the size of waterway for which a culvert is more economical than a bridge

For this premise and the discussion to follow, the term "large culvert" includes structures classed as bridges for administrative purposes but which are actually culverts on a large scale Without attempting too fine a division line, any waterway under a highway may be considered a culvert if its entire perimeter is functionally streamlined from end to end.

Objectively, the culvert so confines a stream laterally (to shorten spans) that depth is materially increased Crown elevation is usually determined as the minimum consistent with the waterway requirment, so that confinement may be vertical as well as lateral. Hence the crown must be streamlined to pass floods without entrapment of drift or generation of major turbulence

The longer the waterway the more the saving in structure cost if the stream is narrowly confined This follows in part from the cost of end transitions (headwalls, endwalls, debris racks and energy dissipators) which are disproportionately expensive for short culverts Consequently, if a reconstructed highway is multilane or on a high grade over a minor stream, a large culvert may be more economical than a small bridge.

Such replacements may be made at widely differing sites In our experience, old bridges have been replaced by pipes as small as 18 in in diameter and by arch culverts as large as 285 sq ft in section For the small sections, determinant of size is mostly a matter of hydrology, but for those over 15 sq. ft in section, hydraulic principles become more and more important.

Extrapolation of Formulae

Whether the basic culvert formula leads first to a design discharge or directly to a waterway area, the shape of the waterway under the highway is not taken into consideration In a particular case, if the required area is 16 sq ft, the designer may choose a 54-in pipe or a 4 by 4 box The formula would be satisfied if he selected a 8 by 2 box or a 2 by 8 box The latter would be a hydraulic absurdity and also expensive. Up to this size, the cheapest section should be satisfactory hydraulically

In another case, suppose the required area is 100 sq ft Hydrology will be satisfied by a 135-in pipe or a 10 by 10 box or a D-19 arch culvert (10 08-ft span by 12 33-ft depth). But if the outfall is free, any of these flowing full will discharge at high, probably damaging, velocity This outfall velocity would be much less if the designer chose a double 10 by 5 box or a battery of five 60-in pipes.

The area equivalence of these sections governs only at the culvert entrance Once flood water has entered the culvert at a certain rate, velocity through the conduit will depend upon roughness of walls, shape of section, gradient of flow line and freedom of

outlet This is apparent at once by examining the Manning formula.

$$V = \frac{1.486}{n} R^{2/3} S^{1/2}$$

For sections of similar shape, the hydraulic radius, R, increases directly with the size ratio, and velocity as the two-thirds power. Hence doubling the diameter of a pipe adds 59 per cent to the velocity at full capacity.

Since velocity also varies as the square root of the slope and inversely as the roughness, the effect of doubling the diameter will be worse for steep, smooth conduits than for rough ones on mild gradients In fact, the disadvantage of high velocity in large culverts can be offset in many cases by roughening the walls or stepping the grade

Most important, however, is the freedom of outlet If the outlet is submerged, the large culvert will flow full to the outlet, then outfall in a decelerating transition through the wider channel But if the outlet is free, the culvert will outfall in an accelerating transition, as illustrated in Figure 14 C,E,G and estimated for typical cases in Figure 15.

From these general considerations, it may be concluded that cross-section area of large culverts should not be computed from formulae devised for small culverts, unless there is assurance that the outlet will be submerged.

Shape of Section

If the outlet is free, it is most important that the shape of the culvert section be selected with careful regard for the natural cross-section of the stream Natural channels in rocky canyons may have width and depth nearly equal, but stable channels through softer materials are 5 to 50 times as wide as deep

Economy demands some reduction of channel width at highway crossings For bridges the reduction of main channel is seldom as much as 40 per cent, but greater reduction is practicable for culverts, because of protection built into the structures. The greater the reduction, the more costly the protective appurtenances, so that there is, for each site, an economic limit to span reduction This limit may be as great as 50 per cent of main channel

In selecting the shape of wetted section, it is obvious that a circular section is the most economical for pre-fabricated conduits and a rectangular section (nearly square) the simplest to form on the job This latter is economical up to a certain combination of span and earth load, beyond which it is cheaper to form a thin arch than to cast a thick top slab

Now the natural section of most streams is a concave bed under a level water surface. Deforming the upper surface to the crown of a pipe or an arch is not natural and should not be attempted without weighing the consequences At the culvert entrance, the consequences are not severe—backwater, reduced velocity of approach, moderate eddy action, debris entrapment. But at a free outfall the consequences are usually damaging—drawdown flow, increased velocity of retreat, erosion of banks by direct or eddy currents, scour by high bed velocities in supercritical flow.

Structural Voids

The answer, of course, is that the archway of a free culvert is primarily a structural void instead of a waterway. Part of it may be usable for waterway or driftway, but it is careless practice to determine waterway area requirement from, say, the Talbot formula, then select an arch culvert with barely that requirement.

Structural voids are not limited to arch culverts The upper portion of a large pipe culvert is similar in shape and primary function. Even for rectangular sections, the crown slab may be elevated to serve as roadway deck, leaving a structural void

As thus defined, the structural void is readily apparent, but determination of the boundary between usable waterway and the void is more difficult. The writers propose two steps—first, an arbitrary boundary for preliminary studies and estimates and, second, a method of analysis for final design.

Rated Waterway Defined

The first step is to define a "rated waterway" for each large culvert with free outlet as the equivalent usable waterway (exclusive of structural void) for the culvert under the most favorable conditions If the culvert is

laid on sub-critical gradient and if the downstream channel is reasonably secure against erosion and scour, then the tentative selection of a section by its rated waterway may be confirmed by the hydraulic analysis

The rated waterway of any culvert section is determined arbitrarily by the formula.

$$D_{rw} = \frac{b + c\sqrt{n}}{2}$$

Where D_{rw} is the depth of rated waterway above the flow line, b is the culvert span, n the number of spans (usually one) and c depends on type, as follows

$c = 4$ ft. for RC arches and pipe,
$c = 5$ ft. for RC boxes and CM arches,
$c = 6$ ft. for CM pipe.

fills of any height should be tested Use is not uncommon for low fills where clearance is limited

Method of Hydraulic Analysis

It must be emphasized again that the "rated waterway" is a first approximation to design, which should be confirmed or corrected by hydraulic analysis. The objective is to determine stage and velocity at outlet, anticipating, as the next step in design, either modification of section until velocity is tolerable or protecting the culvert and roadway structure against intolerable velocity

The general problem is that of accelerated flow in an open channel—controlled at inlet and free at outlet The analysis should start with the assumption that flow will be critical

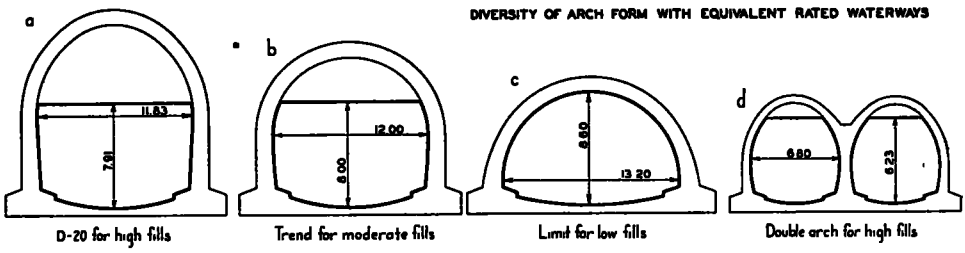


Figure 36. Trend of Arch Design for Least Cost of Rated Waterway

Effect on Arch Design

The reduction is properly severe for concrete arches High and narrow sections with smooth walls, for which a low value of c is applicable, are rated at 62 to 84 per cent of gross area, the percentage rating being less for the larger sections

Such sections are usually designed to give gross area at least cost Redesign to obtain rated waterway at least cost would result in flatter arches—especially under low or moderate fills This could be accomplished by decreasing the rise-to-span ratio of the arch (requiring a heavier arch ring) or lowering the spring line

The conflict between hydraulic and structural arch shapes under moderate-to-high fills should be compromised Figure 36 compares the standard arch (a) with other general shapes which may prove more economical for free-outlet culverts.

Adaptability of the multiple arch (d) for

just inside the culvert entrance Using the design discharge, compute the critical stage, for which velocity head is half the mean depth For this purpose, mean depth is the wetted area divided by the width of the free water surface.

Working upstream from the critical section, the entrance stage (including head of approach velocity) will be the energy head at critical section plus an allowance for entrance loss

Working downstream, the easiest procedure (Fig 37) is to calculate the distance L_1 to a section somewhat smaller than the critical section, assuming that the mean slope of the energy line between two sections is the mean of the slopes at those sections The method will be illustrated for a circular section, using Table 4, a chart (such as Scobey's) (16) for Kutter's formula and the approximation

$$L = \frac{h_{e_1} - h_{e_2}}{S_1 - S_m}$$

For an example, assume a 72-in. RC pipe culvert 200 ft long on a slope of 0.02 and with free outlet,—proposed to carry 252 sec.-ft. (=10 ft. per sec. through 25.2 sq. ft. of its rated waterway) Dividing Q (=252) by $D^{2.5}$ (=88 2), the quotient, 2 86, may be used

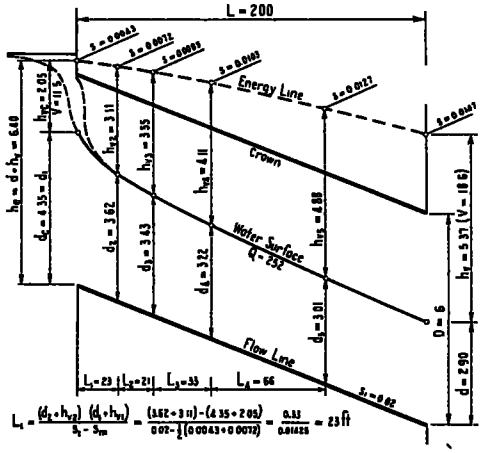


Figure 37. Application of Table 4 and Formula for Non-uniform Flow to 72 in. RC Pipe Culvert with Free Outlet

TABLE 4
CRITICAL ELEMENTS OF CIRCULAR SECTIONS

$\frac{Q}{D^{2.5}}$	$\frac{Vc}{\sqrt{D}}$	$\frac{he}{D}$	$\frac{d}{D}$	$\frac{R}{D}$	$\frac{A}{D^2}$
1 0	3 20	0 579	0 420	0 222	0 312
1 2	3 38	0 640	0 462	0 237	0 355
1 4	3 55	0 698	0 501	0 250	0 395
1 6	3 72	0 752	0 537	0 261	0 430
1 8	3 88	0 805	0 571	0 270	0 463
2 0	4 04	0 857	0 604	0 279	0 495
2 2	4 19	0 907	0 634	0 285	0 525
2 4	4 34	0 956	0 663	0 291	0.553
2 581	4 47	1 000	0 6887	0 295	0 577
2 6	4 49	1 005	0 691	0 295	0 579
2 8	4 65	1 053	0 718	0 299	0 602
3 0	4 80	1 100	0 743	0 301	0 625
3 5	5 20	1 220	0 800	0 304	0 673
4 0	5 64	1 342	0 848	0 303	0 710
4 5	6 12	1 468	0 887	0 300	0 735
5 0	6 64	1 600	0 918	0 294	0 753
5 5	7 18	1 740	0 939	0 289	0 767
6 0	7 76	1 892	0 956	0 284	0 773

to interpolate the critical elements from Table 4, viz :

Critical velocity, $V_c = 4.70\sqrt{6} = 11\ 5$ ft. per sec.
Least energy, $h_{ec} = 1.067 \times 6 = 6\ 40$ ft
Critical depth, $d_c = 0.762 \times 6 = 4\ 35$ ft
Hydraulic radius, $R = 0\ 300 \times 6 = 1.80$ ft

Using Kutter's $n = 0\ 013$ and these values for V and R, find the slope ($S = 0.0043$) of the energy line at this point. Since the culvert slope ($S_1 = 0\ 02$) is steeper, flow will accelerate and downstream sections will be smaller than the critical

Take some smaller section for which elements are given in Table 4,—e. g., that for which $d = 0.604\ D = 3.62$ ft. In Table 4 the first 3 columns are computed for critical flow, but the last 3 columns are geometric relations independent of flow. Hence for the supercritical flow at this second section, $R = 0\ 279$, $D = 1.67$ ft ; $A = 0\ 495$, $D^2 = 17.82$ sq.ft. and $V = 252 \div 17.82 = 14.14$ ft per sec. Again using a Kutter chart, find $S_2 = 0\ 0072$.

Application of the approximation to find $L_1 = 23$ ft is now obvious, with the detail shown on Figure 37. The figure is a profile summarizing a series of steps like the first The steps continue until the sum of L_1, L_2, etc. exceeds the length of culvert, then the depth of flow at outlet can be interpolated and outfall velocity ($V = 18.6$ ft per sec.) computed.

Justification for an analysis for such a design is now apparent Assumption that the pipe flowed full at outlet would have given an outfall velocity of only 10 ft. per sec. Velocity of 21 ft. per sec. would have been figured from an assumption that energy gradient equaled the culvert slope of 0.02

Like computations for box culverts are much simpler and no table is needed For the first step, the critical depth in terms of discharge, Q and span, b, is

$$d_c = \sqrt[3]{\frac{Q^2}{b^2 g}} = 0.314 \left(\frac{Q}{b}\right)^{2/3}$$

For arch culverts, the computations would be just like those for pipes if a table (like Table 4) of elements was available It is recommended that such a table be prepared for each standard arch and placed on the detail drawing.

MAINTENANCE RECORDS

Historical data in regard to the more important culverts where debris, erosion, or other conditions have presented specific problems, are of inestimable value both in regard to conditions at their particular locations and to providing a basis for future design The acquisition of information concerning actual

performance of culverts logically rests with maintenance field personnel, and should be readily accessible to the designing engineer It is of particular value to record flood stage elevations while identifying evidence is yet clearly discernible

Check List Proposed

These data, for reasons of uniformity and ready reference should follow some order as suggested by the following check list

A. Flood Stage
   a Height above invert at entrance.
   b Height above invert at outlet
B. Condition of Culvert
   a. Metal—1 Extent of abrasion, 2 Pitting, 3 Rust, 4 Rivet and bolt condition.
   b Concrete—1 Extent of spalls and abrasion, 2 Reinforcing exposed, 3 Cracks—location, extent, 4. Underminded side walls
   c Joints—1. Open—locations, 2 Water entering or disappearing through joints
   d Drift and detritus (in barrel)—1 Kind and amount.
C · Entrance (Channel, Headwall, Debris Rack)
   a Scour (describe location and extent) 1 In channel, 2 Headwall, 3 Wingwalls, 4 Embankment slopes
   b Obstructing vegetation—1 Nature and extent
   c Drift and detritus deposit (describe nature and extent)—1 In channel, 2 At entrance, 3 On debris rack (if any), 4 In debris reservoir (if any)
D Outlet (Channel, Endwall, Velocity, Check, etc)
   a Scour (similar to entrance)
   b Obstructing vegetation (similar to entrance)
E Roadway Prism
   a Settlement over culvert—1 Amount of sag in grade, 2 Length of roadway affected
   b Probable cause of settlement—1 Saturation following blocking of inlet, 2 Saturation through culvert joints, 3 Other causes.
F. Work Done (give dates)
   a Removal of debris, detritus
   b Additional erosion protection
   c Repaving invert, etc
   d Construct debris rack
   e Other
G Recommendations
   a Work needed beyond the scope of ordinary maintenance—1. Construct debris rack, 2. Lower footings, cutoff walls, etc , 3 Raise headwall to increase head at entrance, 4 Construct flared-siphon outlet, 5. Place riprap, etc , 6. Other.

PRACTICE IN OTHER STATES

Practice and experience in California disclosed several conflicts Between research and rule there was a conflict in the propriety of designing culverts to flow full under head. Between rule and experience there was the certainty that many culverts operated under considerable head without damage Between hydraulic study and experience there was the certainty that many culverts operated under considerable head when free entrances had been designed

To compare these observations with experience elsewhere, a questionnaire was sent to the Highway Department of each State. It was hoped that the replies (Fig 38) would determine whether these conflicts were due to singularity of California hydrology or, perhaps, to a lag of design principles behind hydraulic and hydrologic research.

Summarizing, the questionnaire reveals the following facts and opinions relative to operation of culverts under head·

1 Culverts designed for 30-yr. floods will occasionally operate under head
2 Such operation is not harmful to the structure and seldom damages private property, but will cause damage, usually nominal, to embankments.
3. Damage may be severe near the culvert outlet, eroding toe of embankment and occasionally undercutting the endwalls; but at entrance the hazard is merely the impoundment of debris
4. Few States make a practice of designing for such operation, either as a determinant of culvert size or as a guide to design of protective appurtenances; but many others are agreeable to the practice for (1) a combination of high embankment, non-porous soils and moder-

STATE	Frequency of Design Flood							Operation Under Head				Resultant Damage: Situs			Resultant Damage: Extent		Resultant Damage				Design Policy		
	10-Year	15-Year	20-Year	25-Year	40-Year	50-Year	100-Year	Historical	Regular	Frequent	Occasional	Never	Culvert	Embankment	Outside R/W	Severe	Nominal	Debris impounded	Erosion of inlet	Erosion of outlet	Undercutting	Head allowed	Formula used	Freeboard
Alabama			■								■			⊠	•		■	⊠	■	■			—	■
Arizona				■							■		•	■	•		■	•	■	■	•	•	M	■
Arkansas											■		•	•	•		■	—	—	—	—			•
California							■				■		•	■	⊠		■	⊠	⊠	■	■		I	■
Colorado								■			■		•	■	•		■	⊠	■	■	■			•
Connecticut								■			■		•	•	•	■	■	•	•	■	•			
Delaware			■								■		•	■	•		■	⊠	•	■	⊠		C	•
Florida					■						■		•	■	•		■	•	•	■	•			•
Georgia						■					■		•	⊠	•		■	•	•	■	•	⊠	—	⊠
Idaho			■								■		■	⊠	⊠		■	■	■	•	■			
Illinois								■			■			■	■		■							
Iowa			■								■						■							
Kansas			■								■						■	•	•	■	⊠		I	■
Louisiana			■								■		•	•	•		■	•	■	•	•		C	•
Massachusetts						■					■		•				■							
Michigan		■									■		•	■	■		■	•	•	■	■			•
Minnesota			■								■				■		■			■	■			■
Mississippi										■	■		—	—	—	—	—	—	—	—	—			
Nebraska			■								■				⊠		■	⊠		■	⊠		—	
Nevada	■										■		•	■	•		■			■	■		I	■
New Hampshire							■				■		■	■	■	■	■	■		■	■			•
New Jersey	⊠		■			⊠					■				⊠		■	—	—	—	—			■
North Carolina											■		⊠	⊠	■	⊠	■	■	⊠	■	⊠		I	
North Dakota			■								■				⊠		■			■	⊠			
Oregon			■								■		■	■	•		■			■	■			
Pennsylvania								■			■			⊠	⊠		■			■	⊠			•
Rhode Island	■										■		•	■	■		■		•	•	■			
South Carolina			■								■			⊠	⊠		■	•	•	■	⊠			
South Dakota			■								■		•	■	■		■		•	■	■			
Tennessee						■					■			⊠	⊠		■	⊠	⊠	■	⊠		I	
Utah			■								■			⊠	⊠		■	•	⊠	■	⊠			
Vermont							■			■	■				•		■	⊠	•	■	⊠		—	
Washington								■			■			•	•		■	⊠	⊠	■	⊠			
West Virginia								■			■		■	■	•	■	■	■	■	■	■			•
Wyoming						■					■						■			■			I	

LEGEND

■ Yes
⊠ Occasionally
— No answer
• No, negatively
□ No, positively
I Iowa Formula
C Chart
M Miscellaneous

Figure 38

ate-to-steep channel gradient and for (2) submerged culverts in flat terrain

5. Except for empirical practice in local areas, the Iowa formulae constitute the only system used in such design

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DESIGN OF SIGNS FOR THE PENTAGON ROAD NETWORK

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SYNOPSIS

To complete the facilities for traffic in the vicinity of the new Pentagon Building in Arlington, Virginia (housing 40,000 employees in War Department offices) over 400 signs were needed to direct and control traffic The major network of roads providing arterial connections between all important streets and highways in Arlington, Virginia, and the three principal bridges across the Potomac River to Washington, D C., and access connections to the Pentagon Building and other Government units nearby, included 16 principal forks of 1-way roads, 29 inner loop ramps, 30 outer connection ramps and 10 T-type intersections or crossings at grade on street connections Signs were largely plyboard units of a size from 2 by 2 feet to 5 by 10 feet, mounted on the shoulders, and between curbs at forknoses Rounded letters, black legends on white background, were used in conjunction with straight stemmed tilted arrows and U. S shield route numbers. Letter heights of 4 to 12 inches were used for legibility consistent with likely speeds of travel and the importance of the intersection. At main forks of the 1-way roads two separate signs were used, each with legend and the arrow for one direction of travel located on the side to which it applied Included were three pairs of signs, mounted to hang from arms of a central mast, suspended over the roadways to