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Contents

An Evaluation of Techniques for Highway User Cost Computation

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This paper compares the EA-1 (computer simulation) and AASHO methods for computing vehicle operating costs in respect to interchanges and route location. It analyzes the generalized design situation of (a) interchange vs intersection, and (b) over- vs under-grade separations for controlled-access highways.

Selected highway design problems such as (a) high vs low bridge crossing, and (b) expressway vs noncontrolled-access highway also are discussed.

●THIS PAPER discusses the use of a digital computer in analyzing the vehicle operating costs associated with certain types of highway alignments and compares these results to those of other more usual user cost analysis techniques. The computer programs involved are those developed by the Civil Engineering Systems Laboratory of the Civil Engineering Department at M. I. T. and described in the Highway Research Board paper (1961), "A New Technique for the Prediction of Vehicle Operating Cost in Connection with Highway Design."

The research reported has two objectives: (a) to test the suitability of the AASHO Report on Road User Benefit Analyses for Highway Improvements (hereinafter referred to as the "Red Book") for the determination of vehicle operating costs, and (b) to explore ways in which the computer programs (hereinafter referred to as the "EA-1 Programs") could most appropriately be used for determining such costs. This research is far from complete, but the analysis of four types of alignment problems can be discussed at this time. Both the alignment situations and the nature of their analysis were quite different in each case. Thus, this paper is essentially a report on four separate, though closely related, investigations involving the use of the EA-1 Programs. (Actually, two different program sets were used in the research described. The original set of IBM 650 programs discussed in the earlier HRB paper on this work was used for the interchange ramp problem. A newer, faster program coded in FORTRAN for the IBM 709/7090 was used for the remaining analyses.)

The first of these was a preliminary investigation of the assumption in the Red Book that vehicle performance is not significantly affected by variations in highway profile so long as the average or "composite" grade remains constant. This involved running vehicles (in the computer) over several different profiles with the same average grade and recording their performance. The variations in the results were analyzed to give some indication of the constraints that should be placed on the use of the Red Book in estimating fuel consumption under different conditions of alignment and operation.

A second investigation involved a straight-forward analysis of three alternative locations for a 10-mi section of interstate highway. This analysis was performed first with the Red Book and then with the EA-1 Programs and the results compared. Some possible restrictions on the applicability of the Red Book were also inferred from this investigation.

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A third investigation was concerned with the analysis of user costs on two ramps of an existing interchange as compared to the user costs on the ramps of a more elaborate replacement interchange. In this case, the computations were made by three different methods: the EA-1 Programs, the Red Book, and the unit cost tabulations given in Woods' "Highway Engineering Handbook." The results of these analyses suggest certain conclusions on the suitability of each method for the analysis of this type of alignment problem.

The fourth investigation involved a special sort of problem: that of deciding whether to take a nonconnecting secondary road over an expressway or the expressway over the secondary road where the topography does not dictate the selection of one configuration over the other. The differences in the user costs associated with such alignment alternatives are so slight as to be undetectable with the Red Book, but with high traffic volumes these differences are nonetheless significant. The EA-1 Programs can determine these differences. The results in this case were set up in a matrix showing the relative user cost advantage of one alignment configuration over the other for different combinations of expressway and secondary road traffic.

It was possible to draw a few general conclusions from these four somewhat separate investigations. These bore out earlier expections that the Red Book was well suited to many, if not most, alignment situations, but that in situations requiring an analysis technique of high sensitivity the EA-1 Programs may be superior. In addition, it was possible to show that the basic relationships between fuel consumption and gradient now used in the Red Book need further study.

THE EFFECT OF PROFILE ON FUEL CONSUMPTION

The objective of this first investigation was to test one of the major assumptions of the Red Book method; namely, that vehicle performance is not significantly affected by variations in highway profile so long as the average or composite grade remains constant. Also of interest was the determining of fuel consumption and travel time for trucks as compared to cars, so as to determine whether truck performance could be reasonably approximated by multiplying the values obtained for automobiles by a truck factor.

Description of Grade Test

Two test vehicles were run in simulated operation over several profile configurations, each 10,000 ft long and each with an over-all average grade of 1 percent. The test was then repeated using configurations with over-all average grades of 3 percent. Two factors dictated the selection of these particular average grades: (a) average gradients below 1 percent have little effect on automobile fuel consumption, and (b) it is difficult to find many profile configurations for average grades of more than 3 percent that will not involve unrealistically large gradients (say, 7 to 10 percent) by interstate standards.

Both the 1 and 3 percent tests involved two basic types of configurations: (a) profiles made up entirely of 1 and 3 percent grades, and (b) profiles with grades that were neither 1 nor 3 percent, but that averaged to 1 or 3 percent over the total alignment. Figure 1 shows these grade configurations for the 1 percent test. The

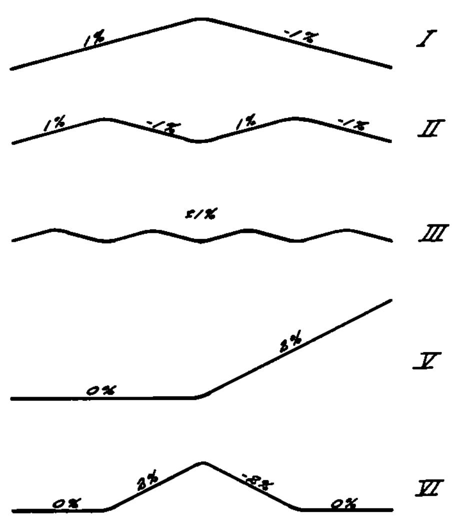

Figure 1. Alternative profiles, 1 percent average grade.

3 percent test was similar, with the following exceptions: (a) run 3 was omitted, (b) run 5 was tried with both 6 and 4 percent grades, and (c) run 6 used 4 percent grades. In all cases, the lengths of the grades were adjusted so that the average gradient over the 2-mi section was 3 percent.

In every case the test vehicles were run at three attempted speeds: 20, 40, and 60 mph.

Test Vehicles

The vehicles used for the test were a 1960 Plymouth station wagon and an International Harvester truck and semitrailer. The important characteristics of these vehicles are shown below:

1. 1960 Plymouth station wagon (8 cylinders, Torqueflite transmission):

Weight (loaded)	5,060 lb
Ratings	max. 230 hp at 4,400 rpm
	max. 340 ft-lb at 2,400 rpm
Transmission ratios	low gear 1.72
	high gear 1.00
Rear-axle ratio	3.31

2. 1960 International tractor (model R205FA) with flat-bed semitrailer:

GCW	55,000 lb
Weight (loaded)	41,480 lb
Ratings	net 166.5 hp at 2,600 rpm
	max. 182 hp at 3,000 rpm
	net 382 ft-lb at 1,200 rpm
Transmission ratios (Int. T51)	1st 8.03
	2nd 4.61
	3rd 2.46
	4th 1.41
	5th 1.00
Two-speed rear axle	low 7.59
	high 5.57

Vehicle Operating Conditions

The vehicles were assumed to be relatively new and in good running order. The truck was loaded, and it was assumed that it only used four forward gears.

Appropriate vertical curves were used on all alignments. Fuel consumption was computed at 2-sec increments and output was punched at 1-sec increments. It was assumed that the vehicles entered each test section at full (attempted) speed.

Results

Table 1 shows a partial summary of the test results. These particular results are for the 40-mph tests, which were most representative of usual operating conditions. The results of the 20- and 60-mph runs showed that fuel consumption is most affected at lower speeds but will generally drop to accepted values at more usual speeds.

Table 1 shows the effect of profile on fuel consumption. Tests 1, 2, and 3 show a fuel consumption of 0.069 gal, and Tests 5 and 6 show an almost 20 percent increase to 0.083 gal, even though the average grade was 1 percent in every case. The same effect can be seen on the 3 percent tests, though it is less pronounced. The variation between Tests 5A and 6A is explained by the fact that Test 5A involved 6 percent grades and Test 6A involved only 4 percent grades. Also, the difference in fuel consumption between the 1 and 3 percent tests amounted to more than 15 percent. The Red Book shows a difference of only about 5 percent between these same two sets of average grades.

TABLE 1

EFFECT OF PROFILE ON FUEL CONSUMPTION[a]

Vehicle	Fuel Consumption (gal)[b]				
	Test 1	Test 2	Test 3	Test 5	Test 6
Car:					
1 Percent	0.069	0.069	0.069	0.083	0.082
3 Percent	0.081	0.081	---	0.092	0.083
Truck:					
1 Percent	0.36	0.36	0.35	0.36	0.38
3 Percent	0.51	0.48	---	0.56	0.51

[a]Runs all at 40 mph.
[b]Per average one-way trip.

The results for the truck test were slightly different. The 1 percent runs produced almost no increase in fuel consumption in going from Tests 1, 2, and 3 to Tests 5 and 6. This is probably explained by the way in which a truck operates. By use of proper gear ratios, it can select the most efficient point in the fuel map at which to operate. The effect on the truck was thus a loss in speed rather than a loss in fuel performance. However, there was a 42 percent increase in truck fuel between the 1 and 3 percent tests. The effect of individual grades was also more pronounced in the 3 percent tests.

Conclusions

The test results suggest the following conclusions:

1. Where the individual grades in a class (as defined in the Red Book) are mixed, the composite grade assumption is probably satisfactory. Where a profile includes widely varying individual grades, the composite grade assumption is probably not too good.
2. The difference in fuel consumption between higher and lower grades becomes pronounced at the lower speeds. The AASHO values are probably satisfactory, nonetheless, at most usual operating speeds.
3. In general, the Red Book may not give entirely satisfactory results for speeds below 50 mph and average grades above 3 percent. The EA-1 Programs provide a more sophisticated method to handle these situations.
4. Fuel consumption for trucks is radically increased on higher grades. The effect on truck time may be even more significant. The Red Book has no way of determining truck performance directly. If truck performance is critical in the evaluation of a project, use of the EA-1 Programs should be considered.
5. The results of these tests appear to disagree with the Red Book fuel consumption curves for automobiles. Values for all grade classes should tend to approach the same asymptote at higher speeds. The AASHO curves do not reflect this fact.

AN INTERSTATE ROUTE LOCATION PROBLEM

A second investigation involved the analysis of a 10-mi section of interstate route for which three alternative locations were being considered. The primary objective of this investigation was to compare annual costs as computed by the Red Book with those obtained by the computer to see whether the greater sensitivity of the computer method could conceivably affect the route location decision itself.

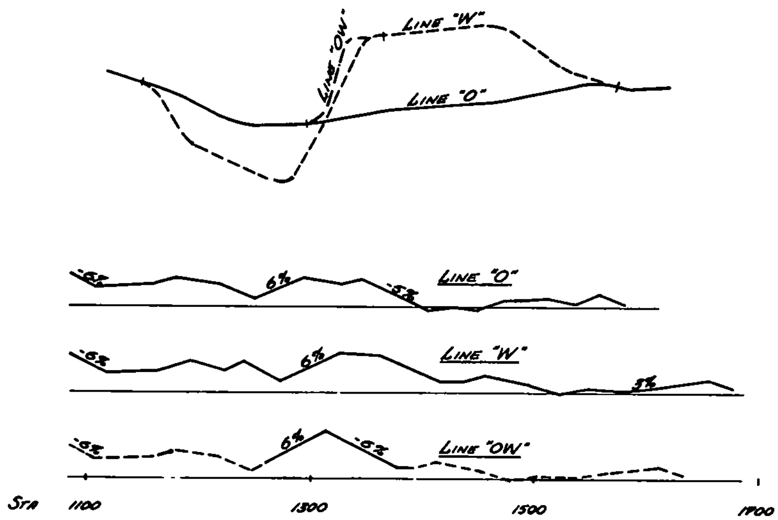

Figure 2. Interstate Route location problem.

The Problem and Its Analysis

Figure 2 shows plan and profile views of the three alignments. Line O, although a very direct route with good grades, encounters soil problems. These would increase the construction cost. Line W would have much lower construction costs, but less favorable grades. Line OW is a compromise line, with even lower construction costs, but with slightly higher gradients. Line O has a total rise and fall of 1,700 ft or a composite grade of 3.62 percent; Line W has a total rise and fall of 2,050 ft or a composite grade of 3.42 percent; and Line OW has the largest rise and fall, 2,150 ft, with an average gradient of 3.91 percent.

The problem resolves itself into a choice between Line O and Line OW. Line O, though it has higher construction costs, has lower road user costs. Line OW has a lower capital cost, but higher road user costs.

The same vehicles used in the previous profile problem were used in the computer analysis of this problem. Both automobile and truck speeds were assumed to be 55 mph. Gasoline cost was taken at $0.32 per gal. Automobile time was valued at $1.55 per hr, while truck time was valued at $4.00 per hr. For both the AASHO and the computer methods, average daily traffic was assumed to be 4,000 vehicles per day with 10 percent trucks.

In the AASHO method, truck cost was taken at four times car cost. The alignments were broken into two sections in computing the composite grade. (It should be noted that the sensitivity of the AASHO method depends on the breaking of an alignment into the various grade sections. If, on the basis of a casual glance, the average grade on the three alignments had been judged the same, the only difference in road user costs would have been due to length.)

Results and Conclusions

Differences between the road user costs as computed by the Red Book and the computer were put on a per mile basis. The results are given in Table 2.

The significant differences in total automobile fuel consumption as obtained by the two methods are readily explained. In the Red Book the basic fuel cost figures were increased by 25 percent to account for the inefficiency of present and future vehicles. If the basic fuel cost figures for the computer were increased by the same factor, the final results would be of about the same absolute magnitude.

TABLE 2

INTERSTATE ROUTE LOCATION PROBLEM

Line	Line Lgt.	Costing Method	Average Per Mile User Costs ($): Car Fuel	Car Time	Truck Fuel	Truck Time	Total Annual User Costs[a] ($)
O	8.9	EA-1	0.017	0.028	0.089	0.085	131,700
		Red Book	0.026	0.028	(0.106)	(0.112)	143,000
W	11.2	EA-1	0.018	0.028	0.099	0.107	132,600
		Red Book	0.026	0.028	(0.106)	(0.112)	144,000
OW	10.5	EA-1	0.018	0.028	0.093	0.113	134,000
		Red Book	0.027	0.028	(0.108)	(0.112)	144,500

[a]ADT = 4,000; 10 percent trucks.

In any case, both automobile fuel and time costs were relatively unimportant in this problem. This was not so true of truck fuel and truck time. Although there was almost no difference in truck fuel consumption as determined by the AASHO method, the computer showed a difference of 3.5 percent for Line OW and 10 percent for Line W when compared to Line O. Differences in truck time were even more significant.

The last column in Table 2 shows the effect of these differences on user costs. The absolute difference between these alternatives reaches a maximum of $1,400 per year, but this difference is relatively insignificant. If the volume of trucks were higher, of course, this difference would have been greater.

One can conclude from this test that the road user costs obtained by the Red Book are acceptable for ordinary analysis purposes. In cases where the make-up of a composite grade is widely variant, where truck volumes are large, or where over-all gradients are large, the use of the computer programs might be preferred.

AN INTERCHANGE RAMP PROBLEM

A third investigation was concerned with the analysis of the user costs on two ramps of an existing interchange as compared to those that would be incurred on the ramps of a more elaborate replacement interchange. The objective of the investigation was to assess the amount of sensitivity needed for this type of problem. Here three different methods of obtaining road user costs were compared: the EA-1 Programs, the Red Book, and the techniques given in Woods' "Highway Engineering Handbook." Both the manner in which each method treats one-way traffic (such as that encountered on ramps) and the methods over-all ease of application were of interest.

Description of the Analysis

Figure 3 shows the alignments selected for analysis. A and B are only two of the eight ramps of each of the new and old interchanges. For simplicity, the others are not shown.

Traffic volumes through the present interchange are very high. Peak hour volumes, including a tourist peak hour, already result in serious congestion. It is expected that the volumes will increase steadily until 1970 when a new route to the east will relieve traffic conditions at this facility. Large turning movements on both ramps A and B have a serious effect on the major routes. Congestion on the ramp often results in a total breakdown of through traffic.

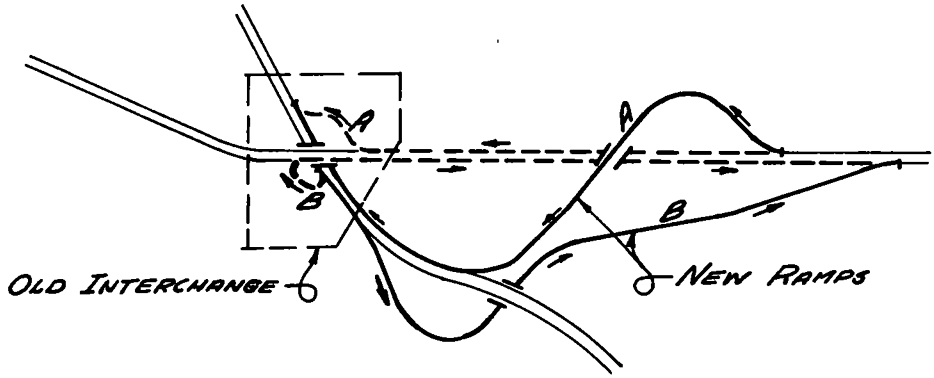

Figure 3. Interstate ramp problem.

Table 3 shows total annual user costs in 1970 as computed by the three methods. The analyses used the same 1960 Plymouth station wagon and the International Harvester tractor-semitrailer combination as in the previous examples. Speed profiles were obtained by considering several factors. The legal speed limit was assumed as the maximum speed for tangent sections. On curves, either the legal speed limit or the design speed (whichever was lower) was assumed to be the maximum speed. At the merge areas of the ramps, where congestion occurs first, these maximum speeds were reduced by an appropriate amount after considering the difference between the ramp traffic volumes and the capacity of the merge areas. (For instance, the cost for present ramp B, as analyzed by the computer, are relatively high. This figure reflects the serious congestion expected in 1970.)

TABLE 3

TOTAL ANNUAL USER COSTS - 1970

	Cost (dollars)		
	EA-1	Red Book	Woods
A:			
Present	403,000	426,000	335,000
Proposed	565,000	577,000	437,000
B:			
Present	679,000	496,000	571,000
Proposed	674,000	763,000	620,000

Results and Conclusions

The results in Table 3 are reasonably close, so it is difficult to draw sharp conclusions. As a result of observations made during the analysis, however, a few points can be made:

1. In very complicated situations, requiring the analysis of several alternatives and a consideration of small differences in grade, alignment, and time delays, the computer programs are almost as easy to use as the other two methods. (This statement must be qualified by the explanation that the use of the computer programs implies a familiarity with the programs and their use and also the ready availability of an appropriate machine.)
2. Higher sensitivity is obtained with the computer programs. This is particularly true for ramps involving one-way traffic. The other methods have no way of handling this problem.
3. The most difficult aspect of this problem is obtaining representative traffic volumes and speeds, including vehicle delays due to congestion. This must be carefully done before any of the methods considered will produce correct answers.

4. The results of the analysis for this particular interchange are inconclusive, because the effect of the ramp traffic on through-route traffic was not considered. In addition, only two of the eight ramps being considered were actually analyzed.

THE OVER-UNDER PROBLEMS

The final investigation dealt with a problem of a rather specialized nature. This involved the decision between taking a nonconnecting, secondary road over an expressway and taking the expressway over the secondary road. There are, of course, many factors to consider in such a problem. Entering grades, right-of-way considerations, and excavation quantities are only three of the important variables involved. In cases where the terrain is flat and other conditions are equal, however, the capital costs for the alternative alignments may be nearly the same, and the vehicle operating costs may therefore be decisive.

The computer programs were used to carry out a user cost analysis of such a problem. The differences in road user costs for over and under conditions were then tabulated for several different main and side road traffic volumes.

Description of the Analysis

Figure 4 shows the assumed profiles for the side road over and for the main road over. A 120-ft opening was assumed for the main road with the side road over; and a 50-ft opening was assumed for the side road with the main road over. The speed profile for the main road through was set at a constant 53 mph. The main road over speed profile was also set at 53 mph. On the side road a speed profile of 42 mph was assumed. In the case where the side road went over, the speed at the top of the bridge was dropped to 37 mph, then increased with a constant acceleration back to 42 mph for the remainder of the pass over the bridge.

A summary of vehicle performance is shown in Table 4. The fuel costs for the main road were greater than those for the side road in all cases. This is due to the increased speed of vehicles on the main road and the resulting higher fuel usage. The problem therefore becomes a study in time savings, not fuel savings.

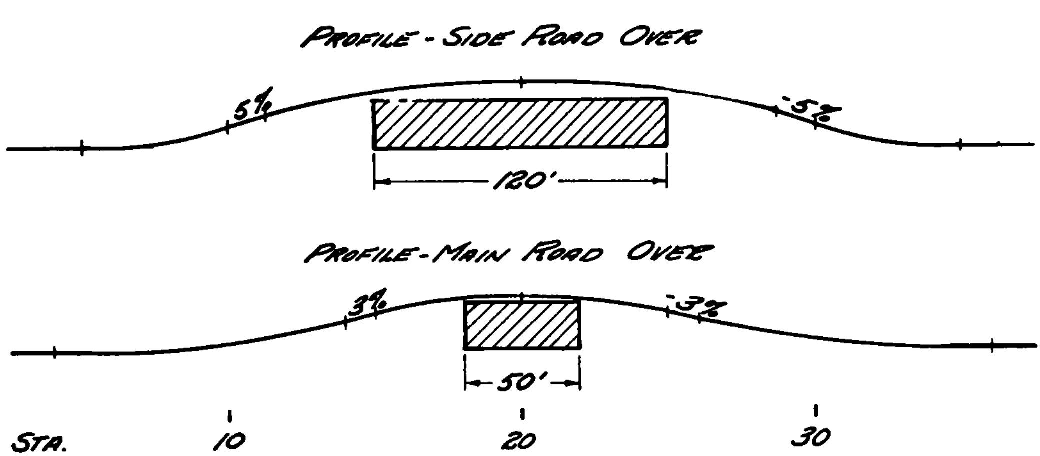

Figure 4. Over-under problem.

TABLE 4

SUMMARY OF VEHICLE PERFORMANCE

Alignment	Car			Truck		
	Speed (mph)	Fuel (gal/trip)	Time (sec/trip)	Speed (mph)	Fuel (gal/trip)	Time (sec/trip)
Side road over:						
Side road	42-37	0.028	66.0	37-32	0.145	75.6
Main road	53	0.032	51.5	53	0.158	51.5
Main road over:						
Side road	42	0.027	64.9	37	0.127	73.7
Main road	53	0.033	51.5	53	0.166	52.2

MAIN ROAD VOLUMES (ADT)

SIDE ROAD VOLUMES (ADT)	5,000	10,000	20,000	50,000	100,000
1,000	-950	-1,950	-5,880	-15,750	-32,200
5,000	1,825	180	-3,100	-12,960	-29,400
10,000	5,300	3,650	360	-9,500	-25,900
20,000	12,200	10,590	730	-2,560	-19,000
30,000	19,200	17,500	14,250	4,380	-12,050

NOTE Entries are total annual user cost differences in dollars

Figure 5. Over-under problem, cost difference matrix.

The results in Figure 5 show for various volumes of ADT, the savings made in road user cost by putting the main road over. Assuming a normal configuration to be the main road over, then the matrix shows that for high volumes on the main road this normal configuration actually has a negative savings (or a cost). For those figures with negative values the side road and not the main road should be put over. For a side road volume of 5,000 vehicles per day and a main road volume of 50,000 vehicles per day, for instance, the value in the matrix is \$12,960. This indicates that (for the conditions assumed) the annual savings from putting the side road over could be approximately \$12,960. On a present worth basis (at 10 percent) this amounts to approximately \$130,000. If the cost of putting the main road over is not \$130,000 less than the cost of putting the side road over, the side road should be put over instead of the main road.

Use of Results

Although the results of the test were obtained by using simplified alignment and configurations, the conditions are typical of those in many places across the U.S. Urban expressways as well as interstate routes in rural locations frequently do not connect with the road over which they pass. With large differences in traffic volumes, the user cost differences can be significant.

Tables for this sort of problem can be prepared quickly and easily using the computer programs. This can be done for different percentages of trucks or for different configurations of over-under alignments.

GENERAL CONCLUSIONS

It is possible to draw a few general conclusions from the four investigations described. The most important of these is that for usual alignment situations the Red Book offers a user cost analysis technique that is not only workable, but probably adequately accurate as well. On the other hand, in alignment situations that are not usual and where user costs are a critical factor in the choice between design alternatives, the EA-1 Programs, though more expensive to use, may offer a superior analysis technique. Unusual situations in this sense would be those where an alignment was geometrically complex, where it involved widely varying or, more especially, steep gradients (say, over 5 percent), where vehicle speeds varied widely, and where heavy trucks comprised a large share of the total traffic.

These conclusions derive from notions of the relative accuracy of the two analysis techniques. The Red Book technique is based, of course, on actual field data, albeit on fewer data than one might wish. The EA-1 method, though based on a conceptually derived model, has been tested out against empirical data with acceptable accuracy. As a result, the absolute accuracy of both techniques is subject to some doubt. It is only because the EA-1 technique is manifestly more sensitive to variations in alignment and vehicle operating conditions that one can reasonably infer it has greater relative accuracy. Given this conclusion, the authors feel such increased accuracy as the computer method affords will justify the expense of its use under the special circumstances cited.

Further, with regard to the question of relative accuracy, the grade tests reported suggest two specific deficiencies in the Red Book technique. The first of these involves the concept of average or composite grade, which apparently breaks down—insofar as fuel consumption is concerned—for profiles with grades widely variant around the average. At the least, this dictates care in the use of Red Book costs. A second problem, however, derives from the apparent errors in the fuel consumption vs grade curves used as a basis for Red Book fuel costs. The correction of these curves is a matter requiring additional study; meanwhile, the EA-1 Programs may be used as a check in alignment situations where this problem seems critical.

The interchange ramp analyses described form the basis for the suggestion that the EA-1 Programs may be superior to other techniques in treating geometrically complex alignment situations. Where the Red Book technique would require many separate detailed analyses, the computer would provide higher sensitivity at little more expense. It should be made clear, however, that this is predicated on the availability of a computer and a working familiarity with the EA-1 Programs. Though relatively simple to apply, these programs can involve the unfamiliar user in the same sort of frustrating minor difficulties that characterize the use of computers in general.

A final, very general conclusion is that, quite apart from production highway design problems, the EA-1 Programs can be a valuable research tool. Undertaking the needed revision of the Red Book fuel consumption curves just mentioned would be an excellent example of such an application of the programs. The "over-under" grade separation investigation is an example of a study with even more direct payoffs. Though the results presented in this paper may not, in themselves, be applicable to the problems of any particular state, the programs could provide this sort of information for any desired set of geometric conditions and do so at very little expense.

Other research studies that come immediately to mind are a general investigation of the relationships between profile and the performance of trucks (supplementing the empirical work heretofore done on this problem) and an investigation of the general effect of interchange geometry on vehicle performance. Specialized studies of traffic congestion on operating costs are also a possibility, though the problems involved in simulating these conditions will be extremely difficult to overcome.

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An Economy Study Aimed at Justifying A Secondary Road Improvement

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The highway system of the United States contains many miles of secondary and feeder roads. Many of these roads, built in the first quarter of this century, had narrow gravel surfaces and poor vertical and horizontal alignment. Subsequently, the surfaces may have been upgraded by adding a bituminous surface treatment or blanket. In most instances surface maintenance costs and accident frequency are high. Many of these roads are now being reconstructed to high standards. Roadways and shoulders are wide and paved; vertical and horizontal alignment are designed for speeds of 60 or 70 mph.

In most instances, however, highway officials plan and execute these improvements without determining whether or not the expenditures represent the best use of the funds at their disposal. This paper presents the findings of such an examination.

The road segment under study was opened to traffic after reconstruction in September, 1959. Findings indicate that this road improvement cannot be justified solely on the basis of savings in market costs, including accidents and time costs of commercial vehicles. The improvement was easily justified if time costs of non-commercial vehicles at currently recommended values were added. The most valuable result of the study, however, has been to pinpoint gaps or weaknesses in present sources of data that must be corrected.

● THE HIGHWAY SYSTEM of the United States contains many miles of secondary and feeder roads which connect small towns with each other or with larger cities, or provide service between town and country, or city and semi-rural residential areas. In many instances, initial construction was carried out in the first quarter of this century. Commonly, surfaces were narrow and of gravel, vertical and horizontal alignment were poor. In later years the surfaces may have been upgraded by adding a bituminous surface treatment or blanket. However, few features of these roads meet modern standards and, with today's traffic, maintenance costs and accident frequency are high.

Since World War II many of these roads have been reconstructed to high standards with roadways and shoulders wide and paved and vertical and horizontal alignment designed for speeds of 60 or 70 mph. This modernization program is continuing as fast as money becomes available. In most instances the agencies that plan and execute these improvements employ no formal economy studies as a means of determining whether or not these expenditures represent the best use of the funds at their disposal.

This paper offers a post-mortem analysis of a high priority secondary highway project. Its aim is to discover by means of an economy study whether or not other similar improvements can be justified on a money basis. Furthermore, it indicates several areas where basic data for economy studies are questionable or entirely lacking.

PROJECT DESCRIPTION

Sand Hill Road is a 2.935-mi link between Santa Cruz Ave. and Whiskey Hill Road in San Mateo County, Calif. Reconstruction, with financing by Federal-Aid Secondary and matching State funds began October 20, 1958, and was completed September 4, 1959. It consisted of right-of-way acquisition, grade and alignment improvement, widening the roadway, and asphalt paving. The area through which the road passes is now relatively undeveloped grazing land of gently rolling hills. However, it has a high potential for residential development. Since the project has been completed, suggestions have been advanced that it is best suited for research-oriented, non-nuisance industrial use. For this analysis, however, it was assumed that the area would remain rural in character with little change in land use along the length of the project.

Sand Hill Road has primarily served as a corridor connecting a large but scattered semi-rural residential area and a few small farms to the cities of Palo Alto and Menlo Park. It also provides access to recreation facilities both distant and close by. The current traffic volume is about 3,500 vehicles per day. On weekdays it is used by persons commuting to work, or making home-to-market and similar home-oriented movements. Weekend peaks of traffic result from trips to recreation areas and driving for pleasure.

SOURCES OF DATA

One purpose of this study was to discover whether or not sound economy studies could be made from data currently available to the engineers of a well-managed county road agency, or if not, to determine what added data were needed. In this instance, construction plans and related construction cost information, maintenance costs, and accident records were made available by the office of the County Engineer. Data on construction costs were complete and satisfactory. Maintenance costs were available on the old road, but were lumped in as part of a larger road unit and could be segregated only by proportion. Maintenance costs for the reconstruction section were of too short duration to be meaningful and there was not in the county a comparable road of longer life to give a longer maintenance cost record. Before and after maintenance costs were therefore built up from the before records on the basis of assumptions that appeared reasonable.

Accident information came from the county copies of the confidential reports filed by officers of the California Highway Patrol. From these reports it was possible to determine the approximate time and location of each accident and its classification; that is, whether the accident involved fatalities, personal injuries, or property damage. Other data useful to an economy study were not available from the report form. As example, some estimate of an accident's relative severity or of the damage caused might lead to more refined methods for estimating accident costs. Again, more detailed information as to whether or not the highway itself was one of the contributing factors leading to the accident would give a better measure of the effect of proposed improvements on accident costs. For this study, the number and classification of accidents was determined from the accident reports. Unit costs assigned to each type were those developed in the Utah Accident Cost Study, since conditions there seemed more representative than those developed in other states (1).

Traffic count data were meager, consisting of an average of one pertinent count per year for the last five years. Projection of these counts and classification of traffic was purely an estimate. It is to be anticipated that means for bettering these predictions will be available soon. San Mateo County has only recently adopted a master plan for land use and is just now implementing a highway needs study based on it. For the purposes of this study, the assumption was made that traffic volume will double over the 20-yr study period. Traffic composition will remain constant at an estimated 92 percent non-commercial passenger cars, 5 percent light commercial vehicles, including pickup trucks and salesman's cars, and 3 percent single-unit trucks. Vehicle running cost data and suggested values for time saving have been taken from data prepared by Winfrey (2).

All of the cost items except the value of time for passenger cars are market costs (those for which the market provides a measure of value). Highway economists generally agree that it is appropriate to include them in economy studies even though their numerical values are uncertain. However, there is argument as to whether time costs for non-commercial vehicles (which is an extra market item) should be included at all. For this reason, these time costs have been reported separately.

COMPUTATION OF ECONOMIC CONSEQUENCES

The primary economic factors considered in this analysis are: (1) capital investment in the improvement, (2) costs to traffic during construction, (3) change in vehicle running costs, between, before and after conditions, (4) value of time savings to commercial and non-commercial vehicles, (5) accident costs, and (6) maintenance costs.

Capital Investment in Improvement

The costs of the project are as follows:

Contract construction items	$241,627
Acquisition of rights-of-way	99,909
Survey and design	27,147
Construction engineering	39,761
Total	$408,444

The analysis period for all elements of the improvement has been set at 20 yr. Salvage value is the cost of right-of-way purchased, less an estimated charge of $75,000 for pavement removal and leveling; therefore, the salvage value at the end of year 20 equals $99,900 - $75,000 = $24,900. Administrative costs of the participating highway agencies and charges for preliminary planning are not included in the analysis inasmuch as they would have been spent whether or not the project was built.

It has been common practice in highway economy to use service lives considerably longer than 20 yr for all roadway elements but pavement. The authors are of the opinion that to do so for projects such as this is unrealistic. It is true that many secondary roads have been in service considerably longer than 20 yr. However, there are many instances where before 20 yr have elapsed, major adjustments in vertical or horizontal alignment have been made or where the existing roadway has been almost completely changed in adapting it to a four- or six-lane design. In other cases changes in land use patterns have caused entire roadways to be abandoned or converted to some secondary use. Overriding even these considerations is technology changing so rapidly it seems unwise to assume that highway use as presently known will continue in its present form for an extended period.

Cost to Traffic During Construction

For a portion of the construction period, some traffic was diverted around the project over Alpine Road, a route some 5 mi longer. Actual operating details of this diversion plan are not now available. Neither were actual observations recorded as to delays suffered by traffic passing through the project during construction. However, it seemed appropriate to recognize traffic delay costs in an exploratory analysis because they should be incorporated in any real-life studies.

Assumptions were as follows: Ten percent of the traffic bypasses the project by taking the longer route. The travel speed of the remaining 90 percent that traversed the project is reduced from 35 mph to 20 mph for 166 days or slightly more than one-half of the construction period. For these vehicles, it is assumed that running costs remain about the same as if they were traveling the old road. This assumption is conservative, but data were not available to support any other conclusion. However, because of the lower speeds, time costs are increased.

Based on these assumptions, the increase in cost to road users during the construction period, over and above the cost of driving through on the old road, can be computed as follows:

1. Added running costs for vehicles traveling through project at 20 rather than 35 mph = 0.
2. Added running costs on 10 percent of vehicles using detour.

Added total vehicle miles = 350 veh per da x 10 mo x 30 $\frac{da}{mo}$ x 5 mi	525,000
Added vehicle miles for passenger cars and light commercial vehicles = 525,000 x 0.97	509,250
Added vehicle miles for single unit trucks = 525,000 x 0.03	15,750

Running conditions on the extra 5 mi are as follows:

Nominal speed, 45 mph; running speed, 35 mph.
Grade, a composite 3 percent. Added horizontal curvature, equivalent to 20 percent of the 5 mi road being on a 10-deg curve.

Running Costs per Mile	Passenger Car and Light Commercial	Single-Unit Truck
Running cost on paved level tangent at 35 mph average if nominal speed is 45 mph cents/mile	4.224	7.547
Increased cost due to 3 percent composite grade cents/mile	0.124	0.723
Increased cost due to 20 percent of the road being on a 10 deg horizontal curve cents/mile	0.034	0.044
Total cents/mile	4.382	8.314

Running costs on detour

Non-commercial and light commercial --	509,250 mi (4.382 cents/mile)/100	$22,300
Single-Unit Truck	15,750 mi (8.314 cents/mile)/100	1,300
	Total added running costs on detour	$23,600

3. Time costs for vehicles traveling through the project at 20 mph rather than 35 mph during 166 days of construction period.

Total added time, min = 2.935 mi $\frac{60}{20} - \frac{60}{35}$ x 166 da x 3,150 veh/da 1,980,000

Time costs, light commercial = 0.05 x 1,980,000 x 0.03	$2,970
Time costs, single-unit trucks = 0.03 x 1,980,000 x 0.035	2,080
Total market time costs	$5,050

Time costs (extra market), non-commercial vehicles 0.92 x 1,980,000 x 0.0225 $41,000

4. Time costs for vehicles using detour (at 35 mph)

Total added time, min = 525,000 mi x $\frac{60}{35}$ = 900,000 min.

Time costs, light commercial = 0.05 x 900,000 x 0.03	$1,350
Time costs, single-unit trucks = 0.03 x 900,000 x 0.035	950
Total market time costs	$2,300

Time costs (extra-market), non-commercial veh = 0.92 x 900,000 x 0.0225 $18,600

Running Costs on New Road vs Old Road:

Running costs are based on an assumed average speed of 35 mph and a nominal speed of 45 mph on the old road, and an average speed of 45 mph and a nominal speed of 50 mph on the new road. Grade and curvature data were roughly calculated from the plan-profile sheets. These calculations are given in Tables 1, 2, and 3. Since the improvement followed the existing alignment very closely, except for flattening the curves, change in length was small and was ignored in the calculations.

TABLE 1

COMPOSITE GRADE CALCULATIONS

New[a]			Old[b]		
Station	Elevation		Station	Estimated Elevation	
54 + 53	302		54 + 53	302	
		+ 124			+ 138
90 + 00	426		90 + 00	440	
		- 133			- 154
120 + 00	293		120 + 00	286	
		+ 31			+ 44
130 + 00	324		130 + 00	330	
		- 20			- 34
138 + 00	304		138 + 00	296	
		+ 28			+ 36
149 + 00	332		149 + 00	332	
		- 180			- 96
			175 + 00	236	
					+ 14
			178 + 50	250	
					- 60
			191 + 00	190	
					+ 6
			194 + 00	196	
					- 32
			206 + 00		
				164	
					+ 3
			208 + 00	167	
					- 15
210 + 06	152		210 + 06	152	
		516			632

[a]Composite grade $= \frac{516 \times 100}{15{,}553} = 3.32\%$.

[b]Composite grade $= \frac{632 \times 100}{15{,}553} = 4.07\%$.

Running costs per mile for old and new road are as follows:

Old Road	Passenger Car and Light Commercial	Single-Unit Truck
Running cost on paved level tangent at avg 35 mph, nominal 45 mph, ¢/mi	4.173	7 456
Increased cost of curves, ¢/mi (See Table 3)	0.0791	0.2017
Increased cost of 4.07 percent composite grade ¢/mi (See Table 1)	0.242	1.447
Total ¢/mi	4.4941	9.1047

New Road	Passenger Car and Light Commercial	Single-Unit Truck
Running cost on paved level tangent at avg. 45 mph, if nominal 50 mph	4.366	8.0205
Increased cost of curves (See Table 2)	0.0272	0.0391
Increased cost of 3.32 percent composite grade (See Table 1)	0.222	1.1400
Total	4.6152	9.1996

The change in running costs for passenger cars and light commercial vehicles is an increase of (4.6152 - 4.4941) = 0.1211 cents per vehicle mile resulting primarily from higher driving speeds on the new road. For single-unit trucks the increase is (9.1996 - 9.1047) = 0.0949 cents per vehicle mile. No attempt was made in these computations to recognize increased running cost resulting from the congestion brought on by increased traffic volumes in the later years of the study.

Present annual traffic (year 0) equals 3,500 vehicles per day x (365 days per year) x 2.935 miles = 3,750,000 vehicle miles. Of these:

Non-commercial and light commercial = 97% = 3,637,500 veh mi.
Single-unit truck = 3% = 112,500 veh mi.

In year zero, the increase in vehicle running costs because of the road improvement is:

Non-commercial and light commercial: $\frac{0.1211}{100}$ (3,637,500) = \$4,400

Single-unit trucks: $\frac{0.0949}{100}$ (112,500) = 100

Total added running cost (year 0) = \$4,500

The annual increase in running costs of 5 percent per year from year 0 through year 20 = 4,500 x 0.05 = \$225.

<u>Savings in Driver Time:</u>

Time saved per vehicle mile by increasing average speed from 35 mph to 45 mph is

$$\left(\frac{60}{35} - \frac{60}{45}\right) = 0.382 \text{ min.}$$

TABLE 2

COMPUTATION OF COST INCREMENT ATTRIBUTABLE TO CURVES, NEW ROAD

Curve Data			Degree of Curve	Computation of Curve Costs (cents per avg. mile)				
Radius (ft)	Degree of Curve[a]	Length (ft)		ΣL[b]	Car Factor	Cost Factor·ΣL	Truck Factor	Cost Factor ΣL
2,177	2.63	1,269	1½	672	0.0377	25.3	0.0532	35.7
1,477	3.88	890	2	2,684	0.051	136.8	0.0705	189.2
1,423	4.03	660	2½	1,269	0.066	83.8	0.0932	118.3
2,577	2.22	1,048	4	1,550	0.114	176.8	0.1705	264.3
						422.7		607.5
3,523	1.62	672						
3,023	1.90	1,636				$\frac{422.7}{5280(2.935)}$ = 0.027 2¢/mi.		$\frac{607.5}{5,280(2.935)}$ = 0.039 1¢/mi

[a]5730/R.   [b]ΣL = Sum of lengths of curves of approximately this sharpness.

TABLE 3

COMPUTATION OF COST INCREMENT ATTRIBUTABLE TO CURVES, OLD ROAD

Curve Data[a]

Sta.	Central Angle, Δ (deg)	External (ft)	Ext. of 1° Curve[b] (ft)	Deg. of Curve, [c]D (deg)	Length,[d] L (ft)
62	16 R	8	56.3	7.03	227
73	13½ R	9	40.0	4.44	304
77	5 R	3	5.5	1.83	273
81	5 R	3	5.5	1.83	273
82	5 L	1	5.5	5.5	91
85	8½ R	3	15.8	5.27	161
91	24 R	18	128.0	7.12	337
94	6 R	1	7.9	7.9	76
96	7 L	1	10.7	10.7	65
98	5 R	1	5.5	5.5	91
100	4 L	1	3.5	3.5	114
107	25 L	15	139.1	9.27	270
111	2 R	½	0.87	1.74	115
113	6 L	3	7.9	2.63	228
118	6½ R	6	9.2	1.53	425
122	3 L	1	1.96	1.96	153
129	20 R	9	88.4	9.82	204
138	2½ R	2	1.36	0.68	368
146	3½ L	5	2.7	0.54	417
152	8 L	5	14.0	2.80	286
161	2 R	2	0.87	0.435	460
176	5½ L	4	6.6	1.65	333
179	2 R	1	0.87	0.87	230
190	28 L	8	175.4	22.0	127

Computation of Curve Costs (cents/avg. mi)

Deg. of Curve, D (deg)	ΣL[e] (ft)	Car Cost Factor[f] L = 100	Car Cost Factor[f] ΣL	Truck Cost Factor[f] L = 100	Truck Cost Factor[f] ΣL
½	1,245	0.0045	5.60	0.0072	8.97
1	230	0.009	2.07	0.0145	3.34
1½	873	0.015	13.10	0.0215	18.75
2	699	0.022	15.40	0.0285	19.93
2½	228	0.0277	6.32	0.0365	8.32
3	286	0.0335	9.58	0.0445	12.72
3½	114	0.041	4.68	0.0542	6.17
4½	304	0.056	17.03	0.075	22.80
5½	343	0.073	25.05	0.097	33.25
7	564	0.104	58.70	0.1335	75.29
8	76	0.123	9.35	0.159	12.08
9½	270	0.159	42.90	0.2062	55.70
10	204	0.172	35.10	0.222	45.25
10½	65	0.186	12.08	0.2395	15.56
22	127	0.4[g]	50.80	0.6[g]	76.20
			307.76		414.44

Car: $\frac{307.76}{5280(2.935)} =$ 0.0198¢/mi

[h] $\frac{0.488 - 0.314}{2.935} =$ 0.0593¢/mi

Total = 0.0791¢/mi

Truck: $\frac{414.33}{5280(2.935)} =$ 0.0267¢/mi

[h] $\frac{1.073 - 0.558}{2.935} =$ 0.175¢/mi

Total = 0.2017¢/mi

[a]Scaled from plans. [b]For 1° curve of given Δ. [c]Ext. of 1° curve of this central angle divided by measured external. [d]$L = \frac{\Delta}{D} 100$. [e]Sum of lengths of curves of approximately this sharpness. [f]At 35 mph. [g]Estimated. [h]Added cost to slow to 20 mph and resume speed.

Dollar values of time savings in year zero are as follows:

Light commercial:	0.030 x 0.382 x 0.05 x 3,750,000 =	$2,150
Single-unit truck:	0.035 x 0.382 x 0.03 x 3,750,000 =	1,500
	Total market time savings in year 0 =	$3,650

The annual increase in market time savings of 5 percent per year from year 0 through year 20 = $3,650 x 0.05 = $182.

Non-commercial (extra-market) time savings in year 0 = 0.0225 x 0.382 x 0.92 x 3,750,000 = $29,600

The annual increase in extra-market time savings of 5 percent per year from year 0 through year 20 = $29,600 x 0.05 = $1,480.

Accident Costs:

The accident records for the old road, adjusted to a traffic volume of 3,500 vehicles per day, indicate a probable rate of 24.9 accidents per year (Table 4). Direct cost per accident, based on studies in the State of Utah(1), has been set at $1,060 (Table 5). Although "direct" costs as defined for the Utah study and "market" costs include somewhat different elements, the Utah data seem to better fit this study than do those from Massachusetts, the other State where such a study has been made. Assuming that the improvement will bring a reduction of 50 percent in accidents in all classes and at an average market cost of $1,060 per accident the savings in year 0 =

$$\frac{24.9}{2}(1,060) = \$13,200/\text{year}.$$

Assuming that accident costs are proportional to traffic volumes, the annual reduction in accident costs brought by the improvement is 0.05 x 13,200 = $660.

TABLE 4

ANALYSIS OF ACCIDENT RECORDS
(Years 1955 through 1960)

Roadway Condition	Before Constr.	During Constr.	After Constr.
Time period, months	46	10	16
Number of accidents	54	15	7
Fatal accidents	2	1	1
Number of fatalities	2	1	3
Accidents per month, average	1.17	1.50	0.44
Total traffic during periods, millions	2.76	0.75	1.35
Accidents per 1,000,000 vehicles	19.5	20.0	5.2
Accidents per 1,000,000 vehicle miles	6.6	6.8	1.8
Accidents occurring in vicinity of Sta. 130[a]	12	10	2

For an ADT of 3,500; average accident expectancy per year before reconstruction $= \frac{19.5 \times 365 \times 3,500}{1,000,000} = 24.9$. Apparent reduction in accidents per vehicle-mile $= \frac{6.6-1.8}{6.6} \times 100 = 73$ percent. For the purposes of this study, a 50 percent reduction in accidents was assumed.

[a]On the old road at this location, the beginnings of a relatively sharp horizontal curve occurred in conjunction with a crest vertical curve of limited sight distance. Although locations are poorly defined on the accident reports (speedometer readings to 0.1 mi from some point that may not be well-defined), the evidence is strong that some 20 percent of the accidents before construction and a majority during the construction period occurred on this road segment.

Maintenance Costs:

The average cost of maintenance on the old road from 1950 to 1958 was about $4,000, divided as follows:

Patching	$1,400
Ditching	800
Weeding and brushing	400
Clean-up	400
Other	1,000

Maintenance savings are estimated as follows:

Patching.—In year 1, reduced 90 % or by $1,260. In subsequent years, patching costs increase uniformly by $66.30 per yr, to equal present costs by the end of year 20.

Ditching, Weeding, and Brushing.—Reduced by 50 percent or $600 per yr because ditches are better designed and more area is paved.

Clean-Up and Other Items.—No change.

Savings in maintenance from the improvement then total $1,860 in the first year but decrease at the rate of $66.30 (3.56 percent) per year from year 1 to year 20.

COMPUTATION OF PRESENT WORTHS

To make the various estimated present and future costs and cost reductions comparable, they have been converted to present worth employing a time-risk factor (interest rate). A rate of 7 percent has been adopted for this study (2, 3, 4). Results are given in Table 6.

DISCUSSION OF RESULTS

Table 6 indicates that, for the conditions assumed, the investment in this secondary road improvement is not fully returned in benefits that can be measured by market standards. (A similar computation, at 0 interest rate, would indicate that the market returns are substantially greater than the costs). An important contributing factor to this result is that improving alignment, sight distance, and roadway surfaces permits higher vehicle speeds which in turn raise running costs substantially. On the other hand, savings in direct accident costs are large, even when a conservative interpretation is placed on the before-and-after accident experience.

When the extra-market item of savings to non-commercial vehicles is considered the analysis indicates that the expenditure is justified. These time savings were computed at $1.35 per hour, which is 85 cents per occupant hour.

TABLE 5

ACCIDENT COST ANALYSIS[a]

Accident Type	Number	Average Direct Cost[b] ($)	Total Cost ($)
Fatal	7	$3,690	$ 25,800
Injury and property damage	75	1,277	95,800
Property damage only	46	299	13,700
Total	128	--	$135,300

$$\text{Average cost per accident} = \frac{\$135,300}{128} = \$1,060$$

[a]From records on file with San Mateo County Engineer; includes a comparable section outside limits of project under study.

[b]Total average direct cost for each class of accidents as reported in HRB Bull 263 for the 1955 Utah accident study (without increases to reflect inflation).

TABLE 6

COMPUTATION OF PRESENT WORTH (AT 7%) OF COST DIFFERENCES RESULTING FROM PROJECT IMPROVEMENT

Item	$	Position in Time Sequence	Conversion Factor	Present Worth at 7%			
				Market		Extra-Market	
				+	-	+	-
Reconstruction costs	- 408,400	End yr 0	1.000		408,400		
Salvage value of R/W	+ 24,900	End yr 20	0.2584[1]	6,400			
Costs occurring during construction							
Running costs through project	0	End yr 0					
Running costs on detour	- 23,600	End yr 0	1 000		23,600		
Market time costs through project	- 5,000	End yr 0	1 000		5,000		
Extra-market time costs through project	- 41,000	End yr 0	1 000				41,000
Market time costs, detour	- 2,300	End yr 0	1.000		2,300		
Extra-market time costs, detour	- 18,600	End yr 0	1 000				18,600
New vs old.							
Running costs	- 4,500	End yr 0					
	- 5%/yr	for 20 yr	15.0[2]		67,500		
Market time savings	+ 3,650	End yr 0	15.0[2]	54,700			
	+ 5%/yr	for 20 yr					
Extra-market time savings	+ 29,600	End yr 0	15 0			444,000	
	+ 5%/yr	for 20 yr					
Accident cost savings	+ 13,200	End yr 0	15.0	198,000		?[3]	
	+ 5%/yr	for 20 yr					
Maintenance savings	+ 1,860	End yr 1	7.82[4]	14,600			
	- 66.30 yr	for 19 yr					
Total				273,700	506,800	444,000	59,600
Net total					233,100	384,000	

[1]Single payment present worth factor.

[2]Computed as follows: For a present sum of 1.0 and an 0.05 increase for each year for 20 years.

a. Determine amount at end of year 1, 1 00 + 0.05 = 1.05.
b. Determine present worth of a series of 20 end of year payments of 1.05 each.
   1.05 (uniform annual series present worth factor) = 1.05 x 10.594 = 11.12.
c. Determine present worth gradient factor for n of 20, i of 7 percent = 77.51 (See Ref. 3, p. 562).
d Multiply factor by annual increase 77.51 x 0.05 = 3.87.
e Sum is desired factor, 11.12 + 3.87 = 14 99.

Note. The same result can be obtained by solving formula 2, p. 66 of "Highway Engineering", by Hewes and Oglesby, John Wiley, 1954 and multiplying the result by 10.594 (the uniform annual series present worth factor).

[3]Extra-market accident costs might include such items as future lost earnings of persons killed and possibly some measure for pain, discomfort, and inconvenience. No suggested dollar values are now available.

[4]Factor determined by methods paralleling items b, c, d, e, of footnote 2.

With the higher unit values proposed by some highway economists, the justification of the improvement would have been even stronger.

As indicated earlier, there are many uncertainties or voids in the data now available for an analysis such as this. Among the areas where research is needed are the following:

1. Better estimates of future traffic volumes and of the components of the traffic stream. Such information may come from O-D studies if they are made with highway economy studies in mind.
2. Better maintenance cost records.
3. Accident reporting aimed at securing engineering and economy study data as well as that needed to fulfill legal and law enforcement needs. Specific weaknesses of present reports include inaccurate location of accident sites and the assignment of accidents solely to such causes as excessive speed or drinking. Although driver behavior or condition is unquestionably a major link in the chain of events leading to accidents, engineers need other information if they are to consider remedial design measures or are to take account of accident reduction as a factor in economy studies.
4. More nearly complete and reliable data on vehicle operating costs, particularly on various types of roadways and under different driving conditions. Vehicle operating costs make large differences in economy-study results. In this analysis, for example, a difference of 0.11 cents per vehicle mile accounts for a cost difference in the analysis of some $60,000. Is this a reliable figure?
5. Better measures of the value of time for both commercial and noncommercial vehicles.
6. Development of means for predicting motor vehicle operating and time costs during construction.
7. Better means for predicting the expected accident experience of roadways constructed to different standards. As a case in point, emergency reconstruction of short sections of road might have strong justification.
8. Better measures of accident costs.
9. Further study of the extra-market item of comfort and convenience to determine whether or not it has a place in economy studies and if so, how it shall be appraised and valued. Because of these uncertainties, it was omitted from this analysis.

CONCLUSIONS

This paper has proposed a method for measuring the economy of rebuilding low standard secondary roads. The example has purposely been oversimplified to emphasize the techniques of analysis. Results are inconclusive. They indicate, for this particular situation, that the market value of returns to users are less than the capital costs, including interest. However, if a value for non-commercial time is included in the benefits, the investment then is warranted.

The study develops a procedure for analysis that can easily be applied to similar situations. However, better arrangements for data collection and research to provide greater knowledge must be implemented before such economy studies become entirely reliable tools for decision making.

ACKNOWLEDGMENTS

This study was financed under research contract CPR 11-7624 between the Bureau of Public Roads and the Department of Civil Engineering, Stanford University. James Morgali assisted in problem formulation and some phases of data gathering. John W. Spiker and others of the San Mateo County Engineer's Office provided construction and maintenance cost data, and, on receiving permission from the California Highway Patrol, made accident reports available for examination. However, responsibility for data interpretation and conclusions rest solely with the authors.

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Total Annual Cost Analysis

J. AL HEAD and R. C. BLENSLY, Respectively, Assistant Traffic Engineer and Planning Survey Engineer, Oregon State Highway Department

For many years, Oregon has used the benefit quotient as a tool for administrative decisions in selecting the best of alternate route locations. This tool, however, has some disadvantages which prohibit its use in all areas where an economic tool is desirable. Over the course of years, considerable thought has been given to this problem, a review of all methods proposed indicates that the concept of minimum annual cost would have the most universal application to satisfy the needs of the Oregon State Highway Department.

The use of a minimum annual cost for economic analysis does not limit itself to the normal analysis of alternate route locations. This paper discusses its application to interchange justification as it relates to location and alternate design.

● THE GREATEST public works program in history was launched by the Federal-Aid Highway Act of 1956. The transportation services and the traffic pattern that develop will have a tremendous impact on local zoning, land values, business and commerce, residential development, and community customs. Highway administrators, realizing the enlarged and far-reaching responsibilities of this new legislation, are depending today more and more on economic studies to guide them in making decisions regarding the proper location and economic desirability of highway improvements.

The Oregon State Highway Department recognized the need for economic studies in route selection 25 years ago. In 1937, the Oregon State Highway Commission published a technical bulletin (1) that contained procedures for evaluating proposed projects.

Fundamentally, the objective of the bulletin was to develop a formula whereby the earning capacity and the benefits of a project could be combined and evaluated in relation to the project cost, thus providing a composite measure (composite quotient) of the desirability of the project. These procedures, though well documented and mathematically sound, were very complex. This complexity, coupled with the rather long period of time required to execute the analysis, discouraged widespread use. As a result, those portions of the procedures dealing only with road user benefits and highway costs were extracted and put into everyday use.

The benefit quotient can be used for several objectives as outlined by Winfrey (2). The first specific objective listed by Winfrey was "to determine whether the facility is economically justified." Common usage of this tool to measure economic justification has resulted in the misconception that any benefit quotient greater than unity indicates that the project under study is economically justified. Although the benefit quotient provides a measure of the relative desirability for alternate improvements, it does not in itself provide a complete measure of its economic desirability. The erroneous connotation applied to benefit quotients tends to lead the uninitiated to make improper applications of this important tool. It is extremely desirable in providing measures for administrators for their everyday use that the information be in as clear and concise a form as possible, and further the possibility of misinterpretation and misuse be reduced to a minimum.

The analysis of alternate improvements is normally made comparing one or more proposed improvements to an existing facility. This type of comparison provides a direct measure of the desirability of each of the proposed improvements to the existing condition. There are many instances, however, where the question is not whether an improvement will be made but which of several alternates is the most desirable. The use of the benefit quotient generally implies that the benefits for each of the proposed improvements must be determined with the existing condition as the basis for these computations. In comparing alternate improvements, Grant (3) has stated, "Only the differences between alternatives are relevant in their comparison." There are many times when it is desirable to compute only the differences between alternates and not attempt to compare them to an existing condition.

Difficulty encountered in Oregon in the use of the benefit quotient has resulted in a critical analysis of its use. A review was made of the various alternate methods of making these comparative analyses. A comprehensive discussion of the various methods has been presented by Grant and Oglesby (4); therefore, no detailed discussion of the various methods will be included in this paper as they are well documented in other sources.

The more common methods of comparative analysis are the equivalent uniform annual cost, benefit cost ratios, and rate of return. Proper analysis of the data and use of these measures will indicate the same order of desirability for alternate projects. The procedure that must be followed to arrive at the final decision differs for each method. In the instance of the annual cost computation, it is necessary to add all costs that can be identified with each of the considered alternates. This is a straightforward

OREGON STATE HIGHWAY DEPARTMENT
Traffic Engineering Division
Planning Survey Section

Summary Sheet

HIGHWAY PROJECT ANALYSIS

________ To ________

Description of Projects

Plan 1 ________

Plan 2 ________

Plan 3 ________

COMPARATIVE PHYSICAL DATA BETWEEN POINTS

________ And ________

ITEM	ROUTES			
	Existing	Plan 1	Plan 2	Plan 3
Length (miles)				
Length of New Construction (miles)				
Traffic Lanes (number)				
Total Rise and Fall (feet)				
Maximum Grade (percent)				
Length of Maximum Grade (feet)				
Total Central Angle (degrees)				
Maximum Degree Curve (degrees)				
Number of Maximum Degree Curves				

COMPARATIVE ROUTE COST DATA

Right-of-Way Costs	$			
Other Construction Costs	$			
Gross Construction Costs	$			
Total Annual Costs	$			

Figure 1.

and direct approach. The benefit cost ratio and the rate of return method, however, require an incremental computation; that is, it is necessary to compute ratios and rates of return for increments of benefits to increments of cost. This incremental analysis is difficult to explain to persons not well-versed in the principles of finance and may therefore leave the impression that the figures are being manipulated to provide the results desired.

To provide a basis for selecting the most desirable of alternatives without including the implication that a project is economically justified and without the necessity of accompanying the results with a detailed explanation, the Oregon State Highway Department has selected the minimum annual cost concept.

For many years, the use of economic measures has been confined to a comparison of alternate highway locations, whereas there has no real reason for excluding other types of problems from this type of analysis. The use of minimum annual costs has facilitated the application of economic measures to interchange location and design problems in the State of Oregon. The Oregon method of computing annual costs is similar to those recommended by AASHO (5). Figures 1 through 8 are the basic work sheets which are used in Oregon. Because the basic procedures are similar to those recommended by AASHO, a detailed explanation of the computations will be omitted.

The remainder of this paper shows how economic measures have been used for interchange location and design studies in Oregon.

Annual Costs

OREGON STATE HIGHWAY DEPARTMENT
Traffic Engineering Division
Planning Survey Section

HIGHWAY PROJECT ANALYSIS

ITEM	ROUTES			
I. ANNUAL VEH. OPERATION COSTS				
A. TIME ELEMENT COSTS				
a. Passenger Vehicles......	$			
b. Light trucks............	$			
c. Heavy trucks............	$			
d. Busses..................	$			
e. Total...................	$			
B. MILEAGE ELEMENT COSTS				
a. Dist. and Surf. Costs...	$			
b. Rise and Fall Costs.....	$			
c. Alignment Costs.........	$			
d. Traffic Stop Costs......	$			
e. Accident Costs..........	$			
f. Total.........	$			
TOTAL ANNUAL VEHICLE OPERATION COSTS (A_e + B_f).......	$			
II. ANNUAL HIGHWAY COSTS				
a. Capital Costs...........	$			
b. Maintenance Costs.......	$			
c. Operation Costs.........	$			
TOTAL ANNUAL HIGHWAY COSTS..	$			
III. TOTAL ANNUAL COSTS (I + II). . .	$			

Figure 2.

OREGON STATE HIGHWAY DEPARTMENT
Traffic Engineering Division
Planning Survey Section

Traffic Data

HIGHWAY PROJECT ANALYSIS

Route ____________
Section ____________
Sub-Section ____________

____________ Highway , No. ______ County ____________

Description of Project ____________ Length ______

Present ADT ______ Present Average Annual Traffic (AAT) ______ Date of Analysis ______

Amortization Period ______ years ____th Year Growth Factor (GF) ______

Traffic Type	% by Classification (1)	Wt. (tons) (2)	Projected Average Annual Traffic: Vehicles (Col. 1 x AAT x GF) (3)	Projected Average Annual Traffic: Tons (Col. 2 x Col. 3) (4)
Total Passenger Vehicles	____	1.800	____	____
Trucks:				
(a) Light	____	3.147	____	____
(b) Heavy	____	9.190	____	____
(c) Semi-trailer	____	27.087	____	____
(d) Full-trailer	____	26.967	____	____
Subtotal (b+c+d)	____		____	
Busses	____	13.920	____	____
Total Trucks & Busses	____		____	____
Total All Vehicles	100%		____	____

Average Weight of Trucks and Busses ____________ tons.

Figure 3.

OREGON STATE HIGHWAY DEPARTMENT
Traffic Engineering Division
Planning Survey Section

Accident Computations

HIGHWAY PROJECT ANALYSIS

Route ____________
Section ____________

	SUB-SECTIONS			
	____	____	____	____
URBAN				
ADT (present)	____	____	____	____
ADT (projected)	____	____	____	____
Pavement Width	____	____	____	____
Intersections/mile	____	____	____	____
Signals/mile	____	____	____	____
Indicated Speed (MPH)	____	____	____	____
Com'l. Units/mile	____	____	____	____
Com'l. Driveways/mile	____	____	____	____
Prediction Source	N-	N-	N-	N-
Accidents/MVM	____	____	____	____
RURAL				
ADT (present)	____	____	____	____
ADT (projected)	____	____	____	____
Pavement Width	____	____	____	____
Shoulder Width	____	____	____	____
Sight Distance Restriction	____	____	____	____
Intersections/mile	____	____	____	____
Com'l Driveways/mile	____	____	____	____
Residential Driveways/mile	____	____	____	____
Prediction Source	N-	N-	N-	N-
Accidents/mile	____	____	____	____

Figure 4.

OREGON STATE HIGHWAY DEPARTMENT
Traffic Engineering Division
Planning Survey Section

HIGHWAY PROJECT ANALYSIS

Time Element Costs

ROUTE ____________

SECTION ____________

SUB - SECTIONS | Total

A. TIME ELEMENT COSTS

a. Passenger Vehicles

1. Average Speed (MPH)
2 Distance (miles).
3 Time (hrs/trip) (a_2/a_1)
4 Cost ($/veh. hr) $
5 Projected Average Annual Traffic (vehicles) 1/.

6. Annual Cost $(a_3 \times a_4 \times a_5)$. $

b Trucks, Light

1 Average Speed (MPH)
2 Distance (miles).
3 Time (hrs/trip) (b_2/b_1)
4 Cost ($/veh hr) $
5. Projected Average Annual Traffic (vehicles) 1/

6. Annual Cost $(b_3 \times b_4 \times b_5)$. $

c. Trucks, Heavy, Semi & Full

1. Average Speed (MPH)
2. Distance (miles)
3. Time (hrs/trip) (c_2/c_1)
4. Cost ($/veh. hr) $
5. Projected Average Annual Traffic (vehicles) 1/

6. Annual Cost $(c_3 \times c_4 \times c_5)$ $

d. Busses

1. Average Speed (MPH)
2 Distance (miles)
3 Time (hrs/trip) (d_2/d_1)
4 Cost ($/veh hr) $
5 Projected Average Annual Traffic (vehicles) 1/

6. Annual Cost $(d_3 \times d_4 \times d_5)$ $

1/ See Traffic Data Sheet

Figure 5.

OREGON STATE HIGHWAY DEPARTMENT
Traffic Engineering Division
Planning Survey Section

HIGHWAY PROJECT ANALYSIS

Mileage Element Costs (sheet 1)

ROUTE ____________

SECTION ____________

SUB - SECTIONS | Total

B. MILEAGE ELEMENT COSTS

a. DISTANCE & SURFACE COSTS
1. Length (miles) ....
2. Roadway Surface Type. ......
3. Surface Type Factor 1/..............
4. Projected Average Annual Traffic (tons) 2/..................
5. Projected Average Annual Traffic (ton-miles) $(a_1 \times a_4)$......
6. Cost ($/ton-mile) (fig. 1)........$

7. Annual Cost $(a_3 \times a_5 \times a_6)$. ....$

b. RISE & FALL COSTS
1. Rise & Fall (foot)...............
2. Projected Average Annual Traffic (tons) 2/...............
3. Foot-Tons $(b_1 \times b_2)$. ....
4. Length (feet) $(5280 \times a_1)$.......
5 Gradient in % $(100 \times b_1/b_4)$.. .....

Truck & Bus Fuel Costs

6. % Trucks & Busses (as decimal)...0.
7. Cost ($/foot-ton) (fig. 2)..........$
8. Cost $(b_6 \times b_7)$. . . . .$

Passenger Vehicle Fuel Costs

9. % Passenger Vehicles (as decimal) ..0.
10. Cost ($/foot-ton) . . ..$
11. Cost $(b_9 \times b_{10})$.... . . . .$

12. Combined Cost Factor $(b_8 + b_{11})$......$

13. Annual Cost $(b_3 \times b_{12})$.. . . . .$

c. ALIGNMENT COSTS
1 Degrees of Central Angle.. ....
2. Degrees of Central Angle/mile. .
3 Cost ($/ton-mile) (fig. 3).. ....$
4. Projected Average Annual Traffic (ton-miles) (a_5). . . ...
5. Annual Cost $(c_3 \times c_4)$ . . ..$

1/ Type I Unimproved earth.... . . . . 1.25
Type II Loose gravel & crushed rock. . 1.15
Type III Oiled Surface (light treatment)..... .. . . 1.05
Type IV Bituminous & Portland Cement concrete pavements . 1.00

2/ See Traffic Data Sheet

Figure 6.

Mileage Element Costs (sheet 2)

ROUTE ____________

SECTION ____________

SUB - SECTIONS ____ ____ ____ ____ ____ ____ ____ Total

B. MILEAGE ELEMENT COSTS (CON'T)

d TRAFFIC STOP COSTS

Passenger Vehicles

1. Number of Stops per year
2. Approach Speed (MPH)
3. Average Vehicle Weight (tons)
4 Cost ($/ton-stop) 1/ $
5 Cost ($d_1 \times d_3 \times d_4$) $

Trucks and Busses

6. Number of Stops per year
7. Approach Speed (MPH)
8 Average Vehicle Weight (tons)
9 Cost ($/ton-stop) 1/ $
10 Cost ($d_6 \times d_8 \times d_9$) $

11 Annual Cost ($d_5 + d_{10}$) $

e ACCIDENT COSTS

1 Projected ADT
2 Nomogram Used

Existing, Proposed Freeway & Urban Routes

3 Accidents/MVM
4 Length (miles)
5 Accidents (0 000365 x $e_1 \times e_3 \times e_4$)

Proposed Rural Routes

6. Accidents/Mile
7 Length (miles)
8 Accidents ($e_6 \times e_7$)

9 Total Accidents ($e_5 + e_8$)
10 Cost ($/accident) $
11 Annual Cost ($e_9 \times e_{10}$) $

1/

Approach Speed (MPH)	Cost ($) Per Ton Stop: Passenger Vehicles	Cost ($) Per Ton Stop: Trucks and Busses
10	0 00047	0 00043
20	0 00123	0 00112
30	0 00212	0 00194
40	0 00341	0 00317
50	0 00592	0 00553
60	0 00762	0 00714

Figure 7.

OREGON STATE HIGHWAY DEPARTMENT
Traffic Engineering Division
Planning Survey Section

HIGHWAY PROJECT ANALYSIS

Highway Costs

ROUTE ____________

SECTION ____________

SUB - SECTIONS ____ ____ ____ ____ ____ ____ ____ Total

A. CAPITAL COSTS

Gross Construction Costs 1/ .... $

TOTAL ANNUAL CAPITAL COSTS 2/ .... $

B. MAINTENANCE COSTS 3/

1. Surface Type..........................
2. Length (miles)..........................
3. Cost ($/mile)..........................$
4. Roadway Maintenance Cost ($B_2 \times B_3$) ....$

5. Type of Structure......................
6. Length (feet)..........................
7. Cost ($/foot)..........................$
8. Maintenance Cost ($B_6 \times B_7$)..............$

9. Type of Structure......................
10. Length (feet)..........................
11. Cost ($/foot)..........................$
12. Maintenance Cost ($B_{10} \times B_{11}$)..........$

13. Total Net Maintenance Cost ($B_4 + B_8 + B_{12}$).$
14. Contingencies 4/........................$

TOTAL ANNUAL MAINTENANCE COSTS ($B_{13} + B_{14}$).$

C. OPERATION COSTS

1. Length (miles)..........................
2. Cost ($/mile)..........................$

TOTAL ANNUAL OPERATION COSTS ($C_1 \times C_2$).....$

1/ Gross costs comprise net costs plus all contingencies.
2/ Amortization based on 30-year life at ___% interest.
3/ Maintenance costs of four-lane sections are twice the costs of two-lane sections.
4/ 3 3/4% of net maintenance cost for supervision, engineering and other contingencies.

Figure 8.

INTERCHANGE LOCATION

The accelerated construction program of freeways resulting from the Federal-Aid Highway Act of 1956 and subsequent acts has increased the frequency of decisions regarding the proper location of an interchange. Among the items that must be considered in interchange location are design features such as acceleration and deceleration lanes, weaving sections and minimum spacing necessary to allow adequate signing, and the pattern of traffic flow and desires. These items do not measure economic feasibility for providing access. The final decision with respect to interchange location should consider economic as well as design and traffic service features.

London Road Interchange

The proposed London Road Interchange is located on I 5 south of the City of Cottage Grove. Cottage Grove, a city of approximately 4,000 people, is located near the south end of the Willamette Valley approximately 22 mi south of Eugene (see Fig. 9). Its economy is based primarily on wood products, with the manufacturing of plywood and lumbering the basic industries.

In 1956 and 1957, the State Highway Department constructed a bypass of Cottage Grove having two lanes and full access control. Access to the Cottage Grove area was provided by an interchange northeast of the city. The next closest interchanges were at Saginaw Road 2 mi north and at Divide 5 mi south. All intermediate county roads were separated from the interstate route (see Fig. 10). Traffic usage and the Interstate highway program call for improving the bypass facility to four lanes. During the public hearing of the proposed improvement, it was called to the attention of the State highway department that London Road, which is a county road crossing I 5 south of Cottage Grove, serves an area with a sizeable timber resource.

Figure 10 shows that there is a direct route from London Road into Cottage Grove, and that such route would be the logical path for vehicles traveling between the London Road area and Cottage Grove and points north. The city street on this

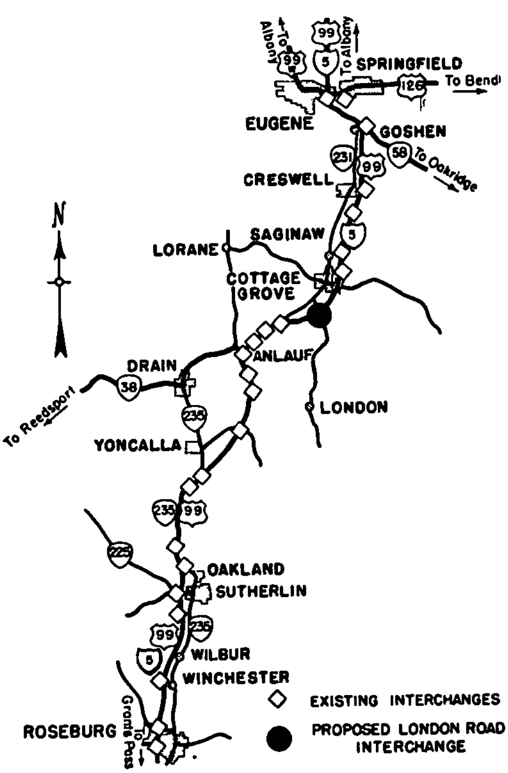

Figure 9. Area map, London Road Interchange study.

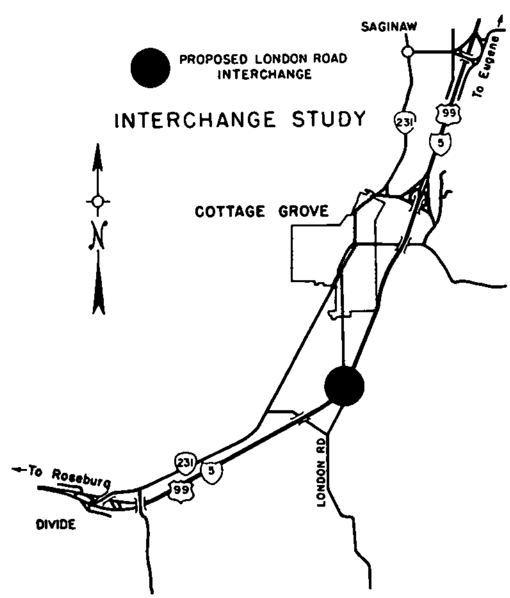

Figure 10. Vicinity map, London Road Interchange study.

route, however, was quite narrow and was not structurally adequate to carry heavy log trucks. The City therefore, had it posted to exclude this heavy truck traffic. The posting of this city street by the City of Cottage Grove required all log trucks to travel an indirect route in moving the logs from the woods to the mills to the north. In addition to the indirectness of the route, the log trucks had to travel through the city on extensions of the State highway system and over city streets. These routings in the City of Cottage Grove carried relatively heavy traffic volumes and have signalized intersections. The provision of access over London Road to I 5 would facilitate the movement of log trucks and other traffic traveling from the London Road area south of Cottage Grove to points to the north. This would also reduce congestion on local streets.

The total annual cost analysis was based on the consideration of the cost of operating all traffic divertable to the proposed interchange. As is the case in most total annual cost analyses conducted in Oregon, traffic is projected to the midyear of the assumed life of the proposed facility. For most cases, this means a 15-yr traffic projection for a 30-yr service life. Table 1 shows that not all of the cost items have been included in this analysis. The procedure used in Oregon does not assign very much weight for the cost of operation for additional grades, poor alignment, traffic stops, and accident costs. The procedures are set up to provide for these costs, however, for the example cited herein, it was felt that the differences were so slight that the resultant effect of these costs could be omitted from the analysis without any detrimental effects. The computations indicated that during the estimated service life of this proposed interchange, the divertable traffic would expend $131,600 per year for vehicle operating costs if they were required to use existing facilities. On the other hand, the provision of the proposed interchange would reduce the total vehicle operating costs to $102,600 per year, or a reduction in vehicle operation costs of $29,000 per year.

TABLE 1

HIGHWAY PROJECT ANALYSIS, PACIFIC HIGHWAY, I 5, LONDON ROAD INTERCHANGE

Annual Costs	Routes	
	Existing	Proposed
Vehicle Operation: ($)		
Time element:		
Passenger vehicles	22,566	14,229
Light trucks	293	185
Heavy trucks	7,142	4,876
Busses	--	--
Total	30,001	19,290
Mileage Element: ($)		
Dist. and surf.	101,603	83,326
Rise and fall	--	--
Alignment	--	--
Traffic stop	--	--
Accident	--	--
Total	131,604	102,616
Highway: ($)		
Capital	--	3,730
Maintenance	--	591
Operation	--	266
Total	---	4,587
Total ($)	131,604	107,203

The proposed improvement would add, however, $4,600 per year in capital, maintenance, and operation costs, resulting in a total annual cost of $107,200 per year. This compares to the $131,600 per year for the existing system, or a net savings to the road user of $24,400 per year. The use of total annual costs does not require the subtraction of differences in costs for the various alternates for a comparison of alternatives. Instead, all that is required is a direct comparison of the total annual costs. In this instance total annual costs with the interchange were less than the cost that would be incurred by using the existing facility, and therefore, they indicated that the provision of an interchange as requested was economically desirable.

Donald Road Interchange

The Donald Road crossing of I 5 is in a rural area approximately 24 mi south of Portland and 21 mi north of Salem (see Fig. 11). It directly serves an area predominately agricultural with some small concentrations of population in rural communities. The nearest interchange north of Donald is little more than 5 mi away and south $5\frac{1}{2}$ mi away. Figure 11 shows that Donald Road has a somewhat peculiar location in that it is the first freeway crossing that can be conveniently used to provide a connection to the freeway for traffic on US 99E from the southeastern part of Portland, Oregon City, and eastern Clackamas County. Figure 12 shows an existing pattern of county roads which serves the area immediately adjacent to the Donald Road crossing of the freeway.

To provide an indication of the potential traffic divertable to an interchange of the Donald Road Overcrossing, a roadside origin-destination study was conducted. The result of this origin-destination study indicated that there was a substantial volume of traffic originating from Aurora and points north along US 99E that had a desire to use the Interstate Freeway but that did not have an opportunity for access to the freeway until it reached the Woodburn Interchange $5\frac{1}{2}$ mi south of the proposed location.

This analysis presented a unique problem in the computation of the average annual road user costs for the life of the facility because additional portions of the freeway system in the Portland Area would have an effect on the routing of traffic at about the midyear of the estimated period of service life for the interchange. For this reason, computations were actually based on two separate traffic assignments.

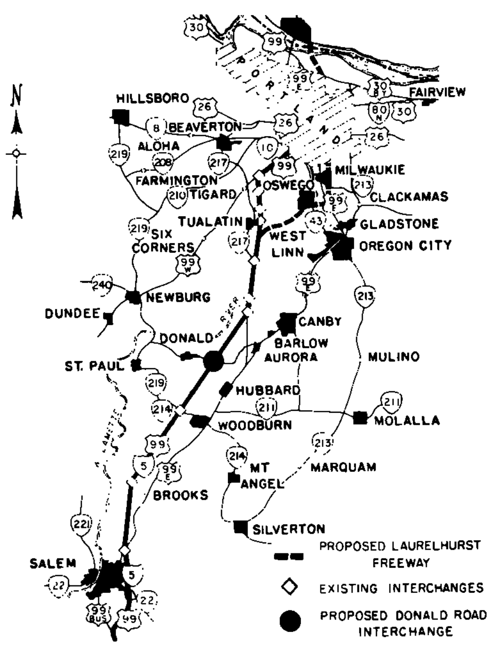

Figure 11. Area map, Donald Road Interchange study.

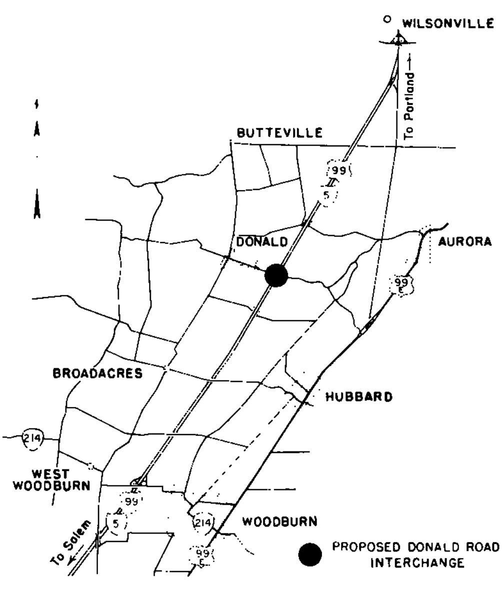

Figure 12. Vicinity map, Donald Road Interchange study.

Table 2 shows vehicle operating costs for passenger vehicles only. This was done to facilitate the computations, inasmuch as the traffic studies indicated very few trucks or buses would be divertable over the freeway by the proposed interchange. For the reason cited under the London Road analysis, costs were omitted for changes in rise and fall, alignment, traffic stops, and accidents. Table 2 shows the resulting annual cost for traffic utilizing the existing system of roads and streets and traffic using the proposed interchange. The total annual vehicle operation costs were $969, 200 via the existing highways, and $943, 500 via the proposed interchange. The annual vehicle operation costs resulting from mileage costs increased with the proposed interchange. The time costs, however, decreased, resulting in a composite reduction in annual vehicle operation costs with the proposed interchange. Annual highway costs for capital improvements, maintenance and operation, were $6, 800 resulting in a total annual cost of $950, 400 for the proposed interchange. In this instance, the total annual costs were less for the proposed interchange than via the existing route.

Although this proposed interchange would reduce the total annual costs, other factors such as the existing pattern of roads, duplication of service, and traffic flow desires must be evaluated to determine the desirability of the proposed interchange. The small reduction in annual costs at this location increases the importance of other factors in arriving at a final decision. At the time of writing, a detailed study of all factors was being made by the Oregon State Highway Department and the Bureau of Public Roads to determine the desirability for an interchange at this location.

TABLE 2

HIGHWAY PROJECT ANALYSIS, PACIFIC HIGHWAY, I 5, DONALD ROAD INTERCHANGE

Annual Costs	Routes	
	Existing	Proposed
Vehicle Operation: ($)		
Time element:		
Passenger vehicles	349, 747	305, 582
Light trucks	--	--
Heavy trucks	--	--
Busses	--	--
Total	--	--
Mileage Element: ($)		
Dist. and surf.	619, 502	637, 937
Rise and fall	--	--
Alignment	--	--
Traffic stop	--	--
Accident	--	--
Total	969, 249	943, 519
Highway: ($)		
Capital	--	5, 959
Maintenance	--	573
Operation	--	322
Total	--	6, 854
Total ($)	969, 249	950, 373

INTERCHANGE DESIGN

The use of the minimum annual cost concept has been used to help determine the most desirable interchange design. The example used for this analysis involves alternate designs for an interchange located on US 97 within the City of Klamath Falls and at the edge of its business district. Klamath Falls is a city with an urban area of 30,000 population located near the California border in central Oregon. It is the only center of population for a considerable distance both north and south along US 97, and therefore, it is a focal point of stops for a large portion of the approaching traffic. The

Figure 13. Area map, Klamath Falls Interchange study.

interchange volume of anticipated traffic and the importance of Klamath Falls to the motoring public dictate that an interchange be provided giving easy access to the Klamath Falls business district. However, physical features require a special interchange design, and it was desirable to make an economic study of each of the proposed designs.

Figure 14. Plan 1, Klamath Falls Interchange study.

Figure 13 is an aerial photograph of Klamath Falls showing the main traveled routes and the location of the proposed interchange. Although it may not be too readily apparent from the figures, the section of US 97 north of Main Street (US 97 Business) rises at a rapid rate. On the west side of Link River and on the west side of US 97, shadows can be seen in a cut section which give some indication of the topography of the area. In order to obtain the present at-grade intersection with Main Street US 97

Figure 15. Plan 2, Klamath Falls Interchange study.

was constructed on a 6 percent grade for 800 ft to the north, followed by another 800 ft of 2 percent grade and a section of 4 percent grade. The restrictions due to land topography, the Link River, and Lake Ewauna present a problem in the design of this interchange. Three alternate proposals are shown in Figures 14, 15, and 16.

Figure 16. Plan 3, Klamath Falls Interchange study.

The predominate movement of traffic on the interchange ramps will be to and from the south. Traffic to and from the north has alternate routes to the central business district and, therefore, will make less use of the interchange ramps. The three alternates have been designed to provide maximum service for the traffic to and from the south, which has resulted in some design inconveniences to traffic from the north. Plan 1 shows the original design for an interchange at this location. Southbound traffic from the freeway (US 97) was not provided with a direct access ramp to Main Street because of adverse grades and the limited distance available for overcoming the grade differential. The local city street (Conger Street) on the west side of the freeway has a grade diametrically opposite from the grade on the freeway. Figure 14 shows that a relatively short distance north of Main Street is a major structure crossing an existing city street. There is insufficient distance between this structure and Main Street to provide a direct off-ramp for southbound traffic.

Consideration has been given to shifting the freeway line. There is a large motel on the east side of the freeway which precludes movement in that direction without adding substantially to the right-of-way cost. The residential property on Conger Street is located on select building sites because of the river frontage and consists of some of the more expensive dwelling units in the area. The additional right-of-way costs are estimated to be about equal to the additional construction costs incurred under Plan 2.

Plan 2 provides for a more direct approach to the city for the southbound traffic from the north; however, it requires additional structure to carry it over Conger Street, the Link River Bridge, and under the freeway. In addition, it is necessary to raise the grade of the freeway line so that full clearance can be obtained on the off-ramp.

Plan 3 reduces to a considerable extent the out-of-direction travel required of southbound vehicles leaving the freeway at the proposed interchange.

TABLE 3

HIGHWAY PROJECT ANALYSIS, DALLES-CALIFORNIA HIGHWAY, US 97, MAIN STREET INTERCHANGE, KLAMATH FALLS

Annual Cost	Route Plan		
	1	2	3
Vehicle Operation: ($)			
Time element:			
Passenger vehicles	90,301	87,008	81,301
Light trucks	1,253	1,206	1,128
Heavy trucks	16,780	16,308	14,872
Busses	1,369	1,319	1,244
Total	109,703	105,841	98,554
Mileage Element: ($)			
Dist. and surf.	267,574	250,749	252,907
Rise and fall	--	--	--
Alignment	--	--	--
Traffic stop	--	--	--
Total	377,277	356,590	351,461
Highway: ($)			
Capital	83,330	115,703	83,599
Maintenance	3,576	4,349	2,571
Operation	1,260	1,414	966
Total	88,166	121,466	87,136
Total	465,443	478,056	438,597

For this analysis, operational costs resulting from changes in rise and fall, alignment, traffic stops, and accidents were assumed to be negligible and omitted from the computations. Plan 1 (Table 3) resulted in a total annual cost of $465,400, Plan 2 in $478,000; and Plan 3 in $438,600 which indicates that Plan 3 is the most desirable of the alternates proposed.

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Procedures for Determining the Most Economical Design for Bridges and Roadways Crossing Flood Plains

JAMES MORGALI, Assistant Professor of Civil Engineering, University of the Pacific, and C. H. OGLESBY, Professor of Civil Engineering, Stanford University

Highways crossing the flood plains of major streams are combinations of bridges and approach embankments. The decision as to which portion of the total roadway length shall be on bridge and which on fill involves engineering economy as well as bridge design. Bridges cost more per unit length than approach fills so that, within reasonable limits, the combination of short bridge and long approach has lower first costs. On the other hand, this combination restricts the channel and during floods raises the water level upstream, which may cause damage from flooding. A second and interrelated decision concerns the roadway elevation of the approach fill. Lower fills are overtopped by smaller floods, with damage to the fill and interruption of traffic during and after a flood. On the other hand, overtopping lowers the upstream water level, thereby reducing upstream flood damage.

This paper presents a procedure for determining the most economical combination of bridge and embankment lengths and approach roadway elevation. The analysis takes account of the following costs: capital recovery on the initial investment, maintenance, embankment flood damage, traffic delay and detours, and backwater damage. Two separate examples are presented considering streams in flood plains 900 and 5,000 ft wide.

● FUNDS for highway improvement are limited and highway needs are great. Because a sizeable percentage of highway expenditures is for major drainage structures, economy in their design is highly desirable. In the case of bridges crossing streams having broad flood plains, the first decision probably is to determine how long the bridge is to be and at what height to place the approach embankments if they are to serve as overflow spillways during major floods. This paper proposes a method for determining the most economical combination of bridge and approach embankments for this situation.

To demonstrate the proposed method, two typical examples are worked. The procedure is as follows:

1. Based on an analysis of crossing conditions at the site in question, several bridges ranging from short to long are laid out and priced. The elevation of these structures is set sufficiently high to clear any anticipated flood.
2. Approach fills built to several heights are fitted to each of these bridges and their costs determined. It is anticipated that under extreme flows all but the highest of these approach fills will be overtopped.
3. By an analysis involving the predicted flood flows on the stream, the characteristics of the site, the length of the bridge, and the height of the approach embankment, stream water surface elevations are determined for the several conditions.
4. For each length of bridge, the most economical approach embankment height is determined. Factors taken into account include the capital costs of embankment and pavement, the statistically predicted annual costs of anticipated flood damage to the

particular site would, if possible, be based on past stream flow records. For more information on frequency curves see Linsley et al (4, pp. 555-559).

Number of days and times a given flow has been exceeded. —This information is given in two graphs; one, the number of days that a given flow is exceeded, and the other, the number of times various flows have been exceeded (see Figs. 7 and 8). On large streams, this information may be available from past records.

Stage-discharge curve. —The stage discharge relationship is shown in Figure 9. The values for plotting this curve may be computed for any site by using conveyance and river slope described by Bradley (1). Normal stage represents the elevation of the water surface at the bridge site when the channel is unrestricted by any crossing at all.

Stage-damage curve. —The stage-damage curve is a plot of expected damage to improvements lying in or adjacent to the flood plain for a given stage (see Fig. 10). This must be constructed for each individual bridge site, recognizing future changes in flood plain use. In constructing the damage curve for the example problems, it has been assumed that damage is linear with stage to simplify the computations. Some of the U. S. Geological Survey water-supply papers give information on various flood magnitudes and damages. Also, the U. S. Army Corps of Engineers has made numerous studies on this subject. As yet, however, authoritative procedures for estimating flood damages are still lacking.

Stage-damage relationships are, of course, dependent on bridge site location. Damages in unsettled areas would be extremely low; they would increase with the intensity of land use. Again, stage-damage relationships would vary depending on encroachment of developments into the flood plain and the presence of dikes or levees that might be overtopped.

Traffic detour costs. —The traffic detour cost is the added cost to vehicle owners who detour by way of another stream crossing or who defer an intended trip. A detailed presentation on detour driving costs is outside the scope of this paper. As is the case with flood damage costs, basic data and procedures for making such computations have not yet been fully agreed on. Their magnitude will, of course, be dependent on such factors as the number of cars, the added distances traveled in using the detour, detour road configuration, expected speeds, and appropriate charges for added commercial and noncommercial time.

Methods and cost data for reasonably approximating the cost of detouring by another crossing are found in Woods (5). On the other hand, economic measures of the cost of postponed travel are lacking.

Costs of damage to embankment from flood overflow detour time during damage repair. —In the example problems, embankment damage is assumed to be proportional to the stage above the embankment roadway. The time for damage repair is assumed to be proportional to the embankment damage. These approximations were made because very limited information was available on how these damages might be evaluated. (It is assumed that the bridge proper is designed to withstand a flood of any magnitude without damage.)

Maintenance costs for bridge and embankment. —This information should come from cost records of the highway agency. It is to be expected that bridge maintenance costs will vary with the type of bridge, climate, and region; embankment maintenance costs (exclusive of flood repairs) will be a function of rainfall and other happenings that bring erosion and parallel deterioration. In this study, these maintenance items have been charged as an annual cost per lineal foot of bridge or embankment.

SELECTING LEAST COSTLY COMBINATION

In this study, cost comparisons are made between bridges of several selected lengths. In turn the bridge of each length has several alternative approach embankments of different heights. The first step in the analysis is to determine, for each bridge length, the least costly embankment height. Then the total costs of bridges of different lengths are compared, each with its most favorable embankment arrangement. The tables accompanying the report show in detail how the various costs are computed. In an actual cost study some of the columns and tables can be combined to simplify the computations.

embankment, traffic detours or delays during and after flooding, and backward damage to upstream property.

5. The total annual cost of bridge, approaches, and anticipated flood damage and traffic detours or delays for each bridge length is determined by combining the capital and maintenance costs of the bridge with those associated with the embankment. The bridge length of lowest total cost is the most desirable from an economic point of view.

In certain instances irreducibles may assume such importance that economy alone should not govern the final decision. For example, it could be undesirable to have a strategic bridge on a major route completely out of service for even a short time. On the other hand, the possibility of a short loss of use should not be controlling in the design of a stream crossing for a secondary road carrying little traffic. Even where such irreducibles might appear important, however, an economy study provides a dollar measure against which such irreducibles can be weighed, thus narrowing the area of uncertainty and providing a valuable tool for decision making.

Some of the costs employed in this paper are not based on actual situations. Rather, seemingly reasonable values have been taken from a variety of sources or, in some cases, assumed without detailed explanation. This was purposely done in order not to obscure the main reason for the paper, which is to develop a procedure for the analysis. It is anticipated that the analyst following this procedure in a real-life situation will develop his own cost information from a study of the site coupled with data supplied by the various divisions of his highway agency.

Another criticism of the proposed method concerns the considerable amount of data collecting and computation required to carry out the procedure as outlined. For many years design engineers have been attempting to weigh the factors included in this analysis. Often this weighing could only be done in a qualitative way because data and procedures were lacking. What is now proposed is that these factors be quantified and converted to money terms to provide a more reliable appraisal of each situation. Investments in major structures are large; it would seem logical to apply an added increment of time and effort to prove that the design makes solid economic sense.

DESCRIPTION ON EXAMPLE PROBLEMS

Problem 1

A two-lane bridge with approach embankment is proposed for crossing a river and wide flood plain. Five alternative bridge lengths are to be compared; these are 800, 1,100, 1,500, 2,000, and 2,500 ft. With each bridge, approach embankments have been set at several levels. Bridges less than 800 ft in length were not considered because they would encroach on the natural channel of the stream (see Fig. 1).

Background information and graphs necessary for the economy study are found in Figures 1 through 13. Tables 1 through 5 outline the method of computation, Table 6 shows the resulting costs. A detailed description of the procedure is included in the text of this report.

Problem 2

This example shows the results of an economy study for a shorter bridge. Lengths considered are 100, 150, 200, and 300 ft. It was chosen because the lengths fall within the range of field verification for the backwater method employed in the analysis.

The proposed bridge and embankment are to carry a two-lane road across a river and flood plain whose cross-section at the bridge site is shown in Figure 14. Table 7 summarizes the results of the analysis; Figures 15 through 20 supply a portion of the necessary data. The remainder comes from source documents.

DATA SOURCES FOR EXAMPLES

For an actual situation, much of the hydraulic and cost information for an economy study is developed as a part of the conventional design process; the remainder can be obtained with a reasonable amount of additional effort. For this paper, however, the

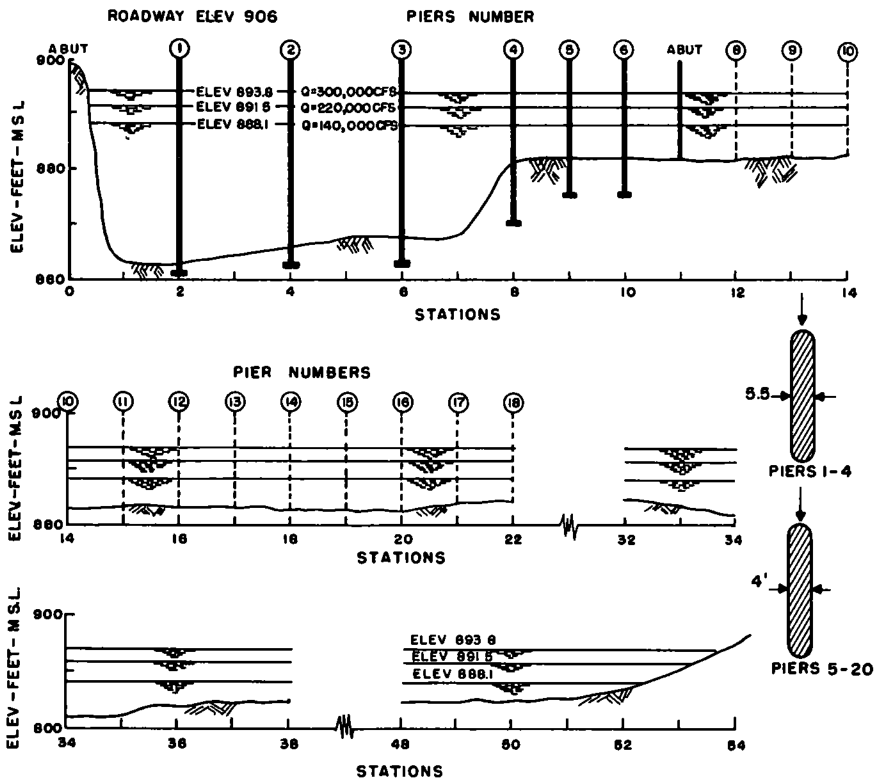

Figure 1. Section of river at bridge facing upstream, example 1 (long bridge) (courtesy of J.N. Bradley).

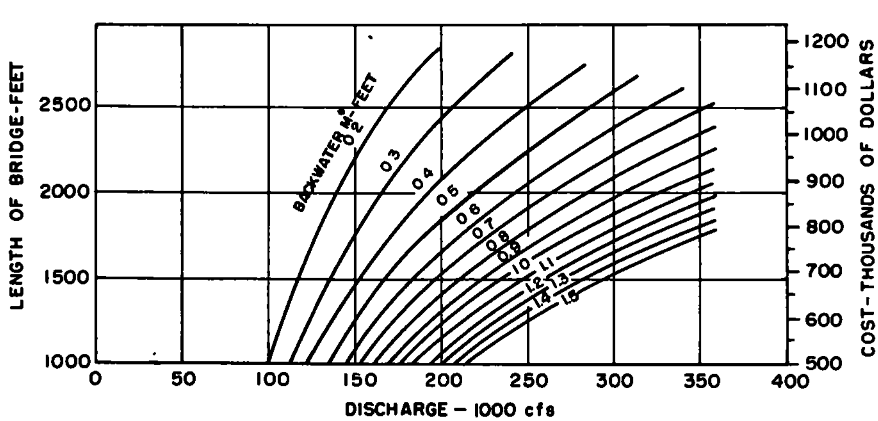

Figure 2. Bridge backwater (courtesy of J.N. Bradley).

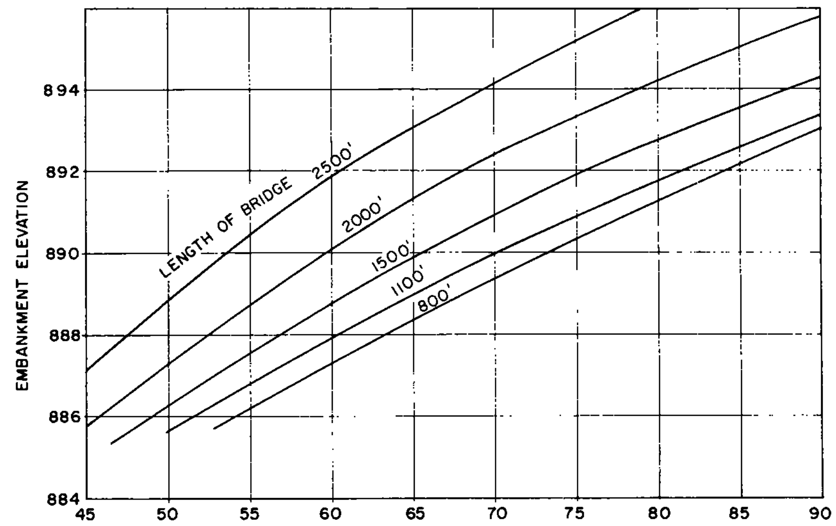

Figure 3. Total cost of embankment and paving (courtesy of J.N. Bradley).

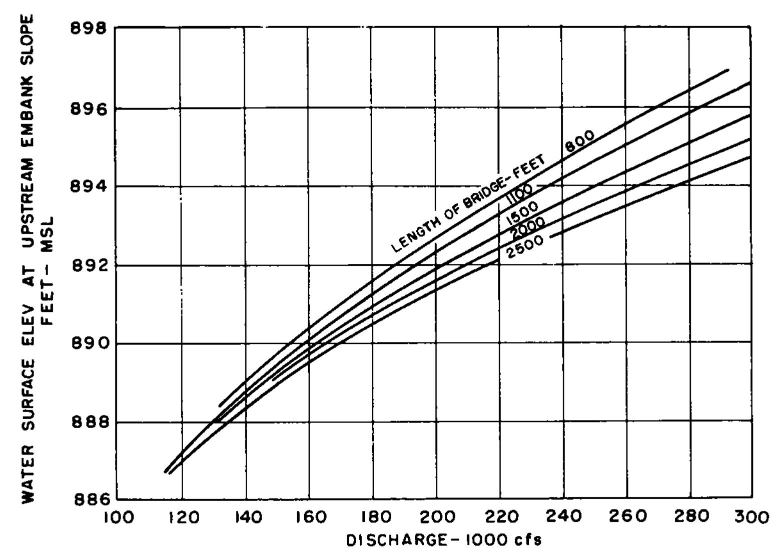

Figure 4. Water surface elevation at upstream embankment slope (courtesy of J. N. Bradley).

In an economy study such as this, cost comparisons should be between alternative bridge-roadway combinations of equal lengths. In cases where, because of differences in approach embankment height, the bridge plus embankment lengths differ among alternatives, pavement lengths have been increased for the shorter alternatives to give each the same over-all length. Again, an economy study is concerned with differences between alternatives. It is differences in costs that are relevant. This means that costs common to all alternatives may be ignored as far as choosing the most attractive alternative is concerned. Furthermore, it is often proper to employ a "with" and "without" approach. For example, this is done with backwater damage costs for each

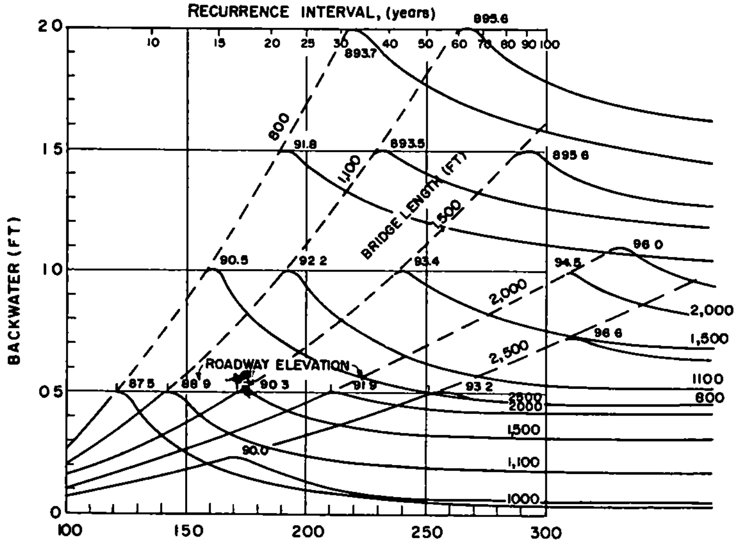

Figure 5. Flow with limited backwater for several bridge lengths (courtesy of J. N. Bradley).

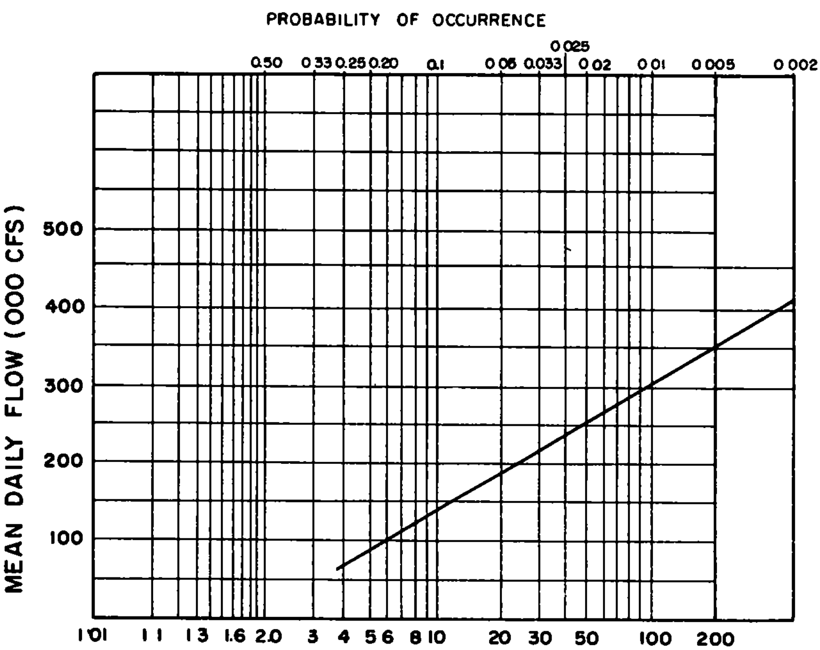

Figure 6. Frequency curve (assumed).

combination of bridge and embankment. With large floods, some damage will probably occur with no bridge at all; this is the base condition. Only the increment of damage resulting because of each bridge-embankment combination is pertinent and is computed.

Costs Related to Embankment Height

There are several annual costs included in most economy studies of approach embankments: (a) capital recovery for embankment, (b) embankment maintenance,

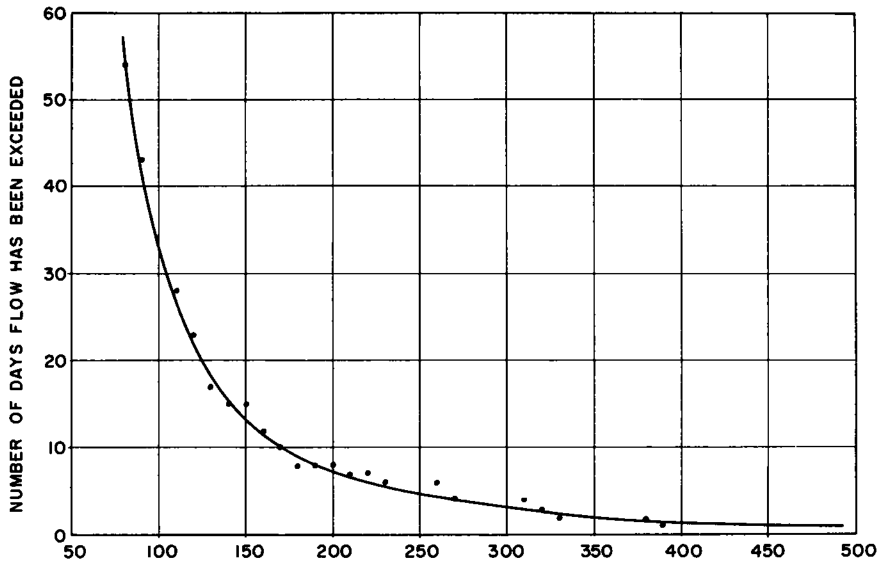

Figure 7. Number of days flow exceeded in past 50 years (courtesy of J. N. Bradley).

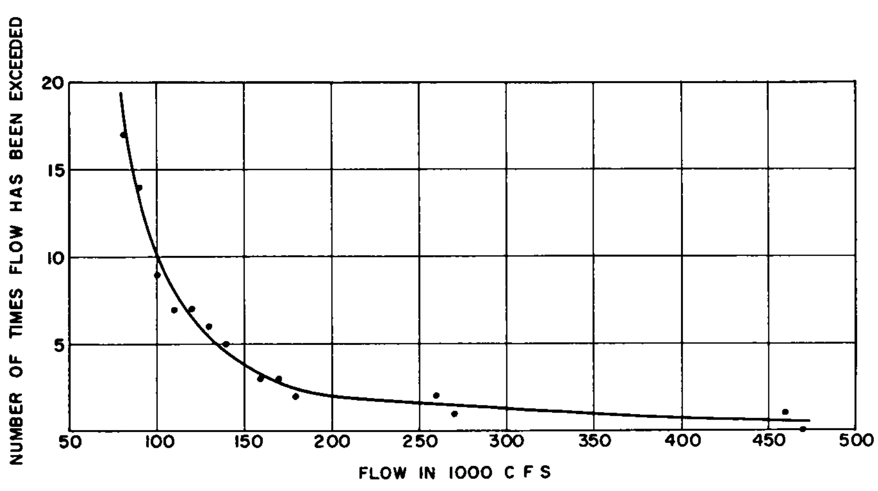

Figure 8. Number of times a flow exceeded in past 50 years (courtesy of J.N. Bradley).

(c) expected flood damage to embankment, (d) expected detouring, and (e) expected increment of backwater damage. Any other variables that might affect vehicle-operating or other costs in a particular case should also be included.

Method for Predicting Expected Average Annual Damage

A numerical procedure suggested by B. Franzini (6) is used to evaluate the annual expected flood damage. A typical annual probability-damage curve is shown as Figure 11. The probability axis is divided into elements P_1P_2----P_n. For each probability P_i there is a damage d_i. The area of a typical element 1-2 is given by

Elemental area 1-2 = $\left(\frac{D_1 + D_2}{2}\right)(P_2 - P_1)$. The sum of all elemental areas under the probability-damage curve is the expected annual cost. A method for summing these elemental areas is shown in Table 1.

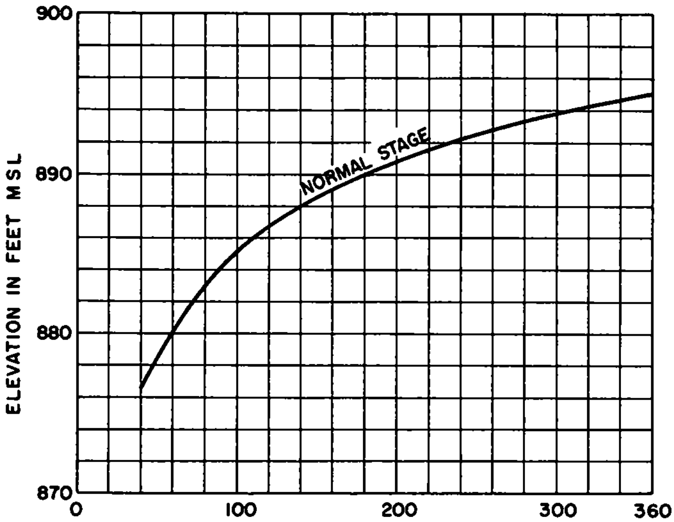

Figure 9. Stage discharge curve for river at bridge site (courtesy of J.N. Bradley).

Method of Computing Embankment Costs

Cost comparisons are made on an annual cost basis; each of the embankment costs is listed with a brief discussion and explanation of how it is computed.

Annual cost of capital recovery for embankment. —Annual cost = (first cost) x (crf-i-n), where First cost = Total cost of embankment and paving; (crf-i-n) = capital recovery factor for interest rate i and analysis period n. The example problems are solved at an interest rate of 7 percent, a period of 30 years for bridge and embankment and with zero salvage value (see Woods (5) or Grant and Ireson (7) for detailed procedures for economy studies and for compound interest tables.)

Annual embankment maintenance. —Embankment maintenance costs have been assumed to be proportional to embankment length. They were set at $0.30 per lineal foot, based on maintenance cost figures supplied by G. S. Paxson of the Oregon Highway Department. This figure is approximate and may be low because it is not necessarily for embankments subject to flooding.

Annual expected embankment damage. —These costs are for repairing damage caused by flood flows overtopping the embankment. Anticipated annual costs decrease as embankment heights increase because overtopping of higher embankments is less frequent. For Example 1 damage costs were assumed to be 5 percent of the total embankment cost for each foot of flow energy head above the embankment roadway elevation.

Very little has been published concerning damage to embankments from overtopping. Kindsvater (8) reports how embankment damage by flood waters occurs and Yarnell and Nagler (9) give some examples of damages from flood flows.

The computation for embankment damage is an application of the method described earlier for evaluating annual expected damage. The embankment damage computation can be set up as shown in Table 2. (A sample calculation for this item combined with

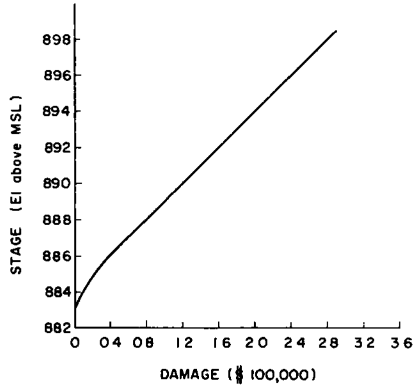

Figure 10. Stage-demand curve (assumed).

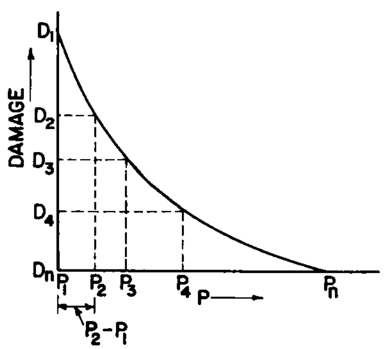

Figure 11. Probability-damage curve.

background information of Example 1 has been taken, for the most part, from materials supplied to the authors by Bradley (1). Example 2 employed the same methods; however, the specific problem was assumed by the authors of this paper.

The following items of information are needed before the economic analysis can be made.

1. Cross-section of the river and flood plain at the bridge site (see Figs. 1 and 14).

2. Bridge costs for the various bridge lengths. For preliminary studies such as these, cost might be roughly approximated as the sum of a fixed cost, plus a constant times the bridge length; e.g., for a bridge length L, bridge cost = a+bL, in which a = the sum of all fixed costs (abutments, etc.) and b = the cost per unit length for piers and superstructure. Bridge costs for Example 1 are plotted on the right-hand ordinate of Figure 2. For Example 2 they were assumed as \$6,300 + \$420 x bridge length (see Fig. 18).

3. Embankment costs for various bridge lengths and embankment elevations. Estimated costs have been plotted against embankment elevation with bridge length as a parameter (see Figs. 3 and 17).

4. Water surface elevation at the upstream embankment slope. Figure 4 shows this as a plot of water surface elevation discharge using length of bridge as a parameter. The method for calculating values for this plot is found in Bradley (1).

5. Bridge backwater. This is recorded in a plot showing bridge backwater without embankment overflow for a given river discharge and bridge length (see Fig. 2). The method for calculating values is found in Bradley (1).

6. Flow with limited backwater for bridges of several lengths. This is shown in a plot of backwater vs river discharge, with bridge length as a parameter (see Fig. 5). The data for the curves of backwater vs discharge without embankment overflow are the same as are found in Figure 2. To develop the portions of the curves to the right of their peaks, it is first necessary to choose a specific value for backwater height, which is the rise in the water surface resulting from the presence of bridge and embankment. The river discharge corresponding to that backwater height represents the flow at which the approach embankment is first overtopped. At higher discharges, the roadway acts as a broad crested weir with a head equal to the difference in elevation between the water surface (flow energy line) and the roadway elevation. The backwater height decreases after overtopping.

Most of the data on Figure 5 was supplied by Bradley. However, the authors approximated the curves sweeping downward to the right for backwaters at overtopping of 1.0, 1.5, and 2.0 ft. Further information on the flow of water over roadway embankments can be found in Sigurdsson (2) and Bradley (3). The backwater computation method is based on model tests conducted at Colorado State University for the Bureau of Public Roads.

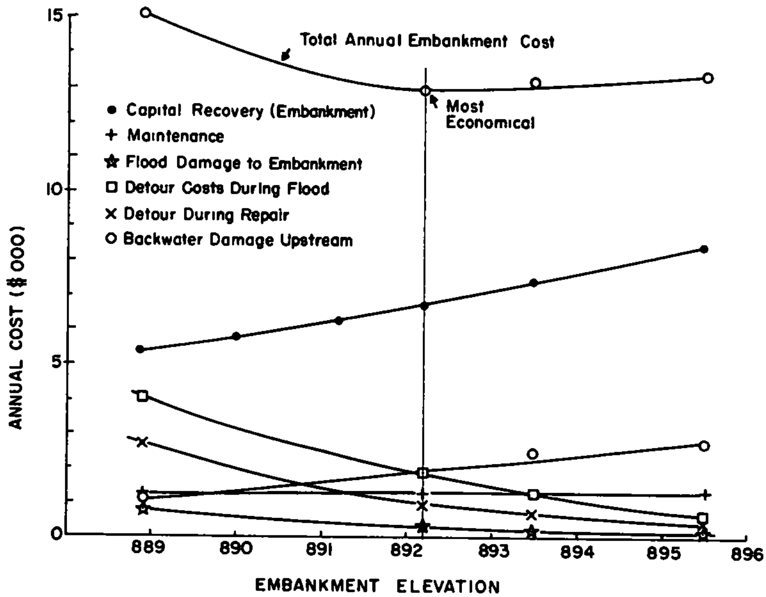

Figure 12. Embankment cost, 1,100-ft bridge.

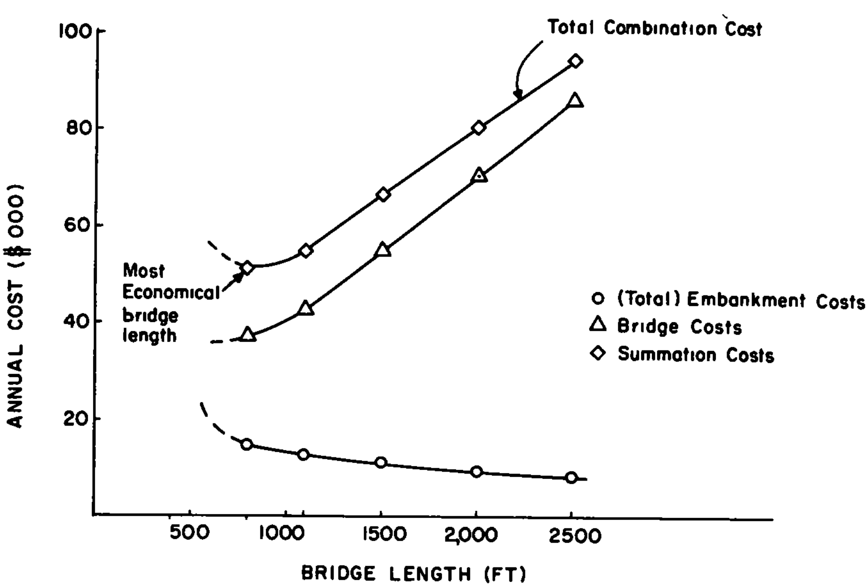

Figure 13. Combined annual cost of bridge and embankment.

Limitations

A few of the limitations of backwater computations, taken from Bradley (1), should be noted:

1. The method of computing backwater is intended for use with relatively straight reaches of streams with approximately uniform cross-section and slope.
2. The U. S. Geological Survey field measurements which were used to verify the application of the laboratory data to field conditions were limited to single bridges up to 220 ft in length on streams with a maximum width of ⅛ mi at flood stage. Verification for flood plains of much greater widths is lacking at the present time.

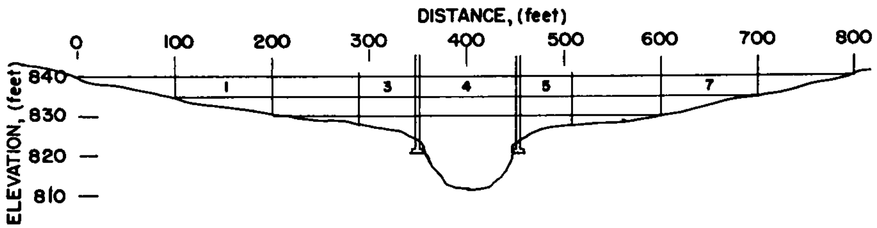

Figure 14. Section of river at bridge (example 2, short bridge).

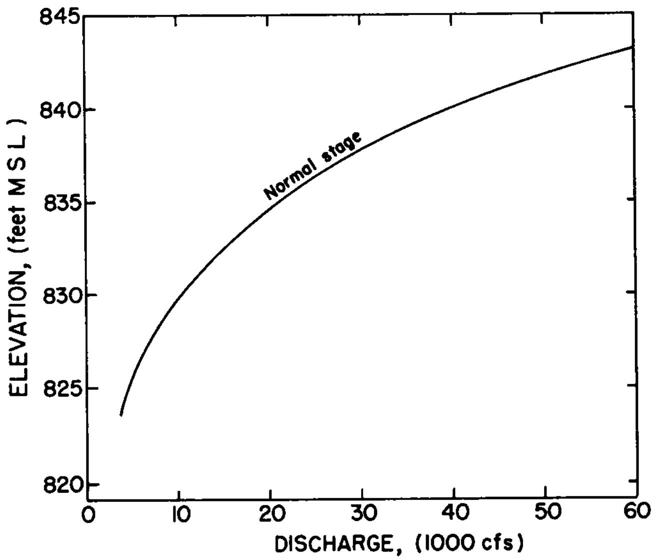

Figure 15. Stage-discharge curve (example 2, short bridge).

3. The computations for backwater assume no scour occurs at the bridge or embankment.

Frequency curve. —The frequency curve gives the probability of an equal or larger mean daily flow occurring in a given year. Figure 6, the assumed frequency curve for Example 1, is developed on Gumbel probability paper. However, any comparable method for finding probabilities is acceptable. In actuality, flow probabilities for a particular site would, if possible, be based on past stream flow records. For more information on frequency curves see Linsley et al. (4, pp. 555-559).

Number of days and times a given flow has been exeeeded. —This information is given in two graphs; one, the number of days that a given flow is exceeded, and the other, the number of times various flows have been exceeded (see Figs. 7 and 8). On large streams, this information may be available from past records.

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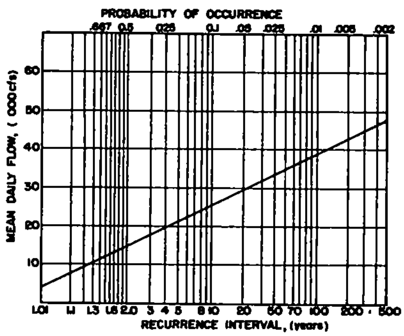

Figure 16. Frequency curve (example 2, short bridge).

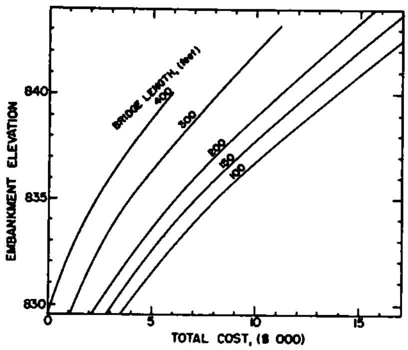

Figure 17. Total embankment costs includes embankment and paving (example 2, short bridge).

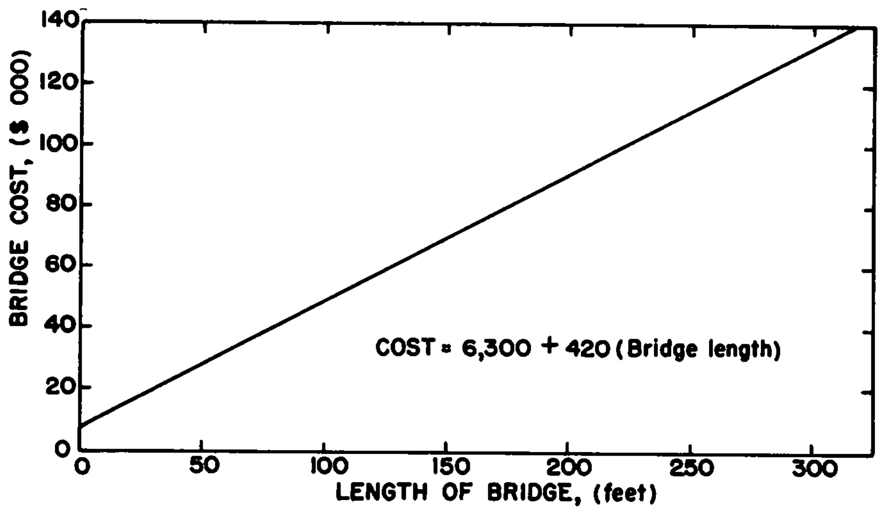

Figure 18. Bridge costs (example 2, short bridge).

plain use. In constructing the damage curve for the example problems, it has been assumed that damage is linear with stage to simplify the computations. Some of the U. S. Geological Survey water-supply papers give information on various flood magnitudes and damages. Also, the U. S. Army Corps of Engineers has made numerous studies on this subject. As yet, however, authoritative procedures for estimating flood damages are still lacking.

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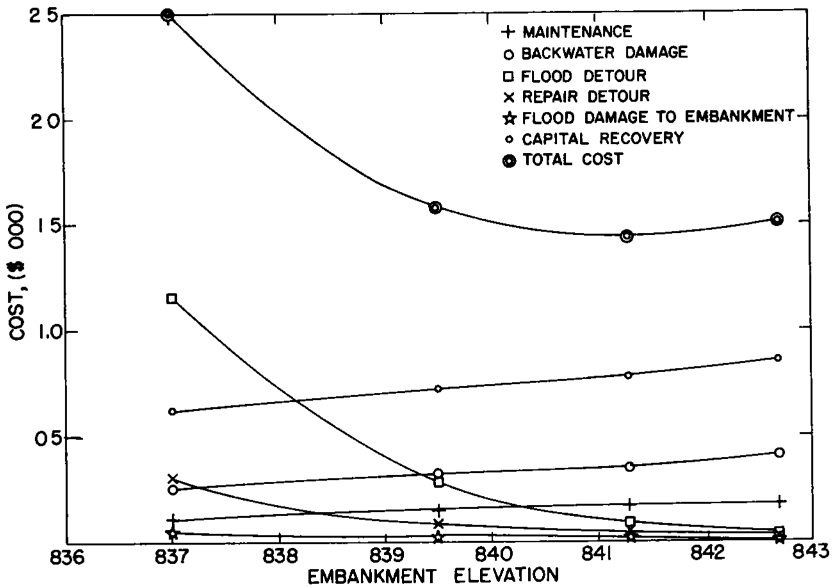

Figure 19. Embankment cost, 300-ft bridge (example 2).

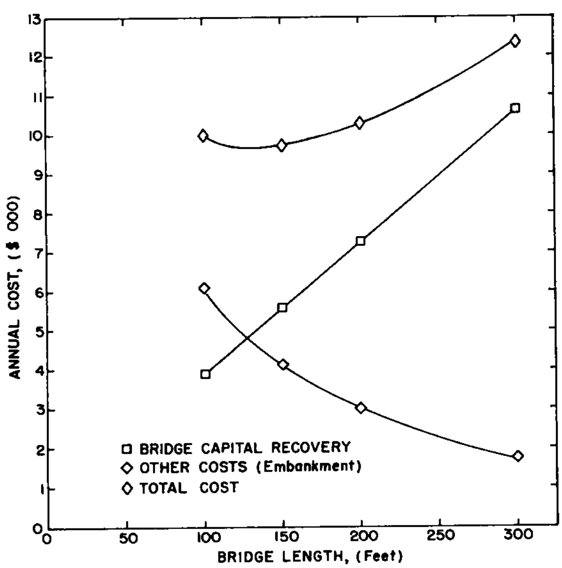

Figure 20. Combination-bridge and embankment costs (example 2, short bridge).

ent on such factors as the number of cars, the added distances traveled in using the detour, detour road configuration, expected speeds, and appropriate charges for added commercial and noncommercial time.

Methods and cost data for reasonably approximating the cost of detouring by ano another crossing are found in Woods (5). On the other hand, economic measures of the cost of postponed travel are lacking.

Costs of damage to embankment from flood overflow detour time during damage repair. — In the example problems, em- damage is assumed to be proportional to the stage above the embankment roadway. The time for damage repair is assumed to be proportional to the embankment damages. These approximations were made because very limited information was available on how these damages might be evaluated. (It is assumed that the bridge proper is designed to withstand a flood of any magnitude without damage.)

Maintenance costs for bridge and embankments. – This information should come from cost records of the highway agency. It is to be expected that bridge maintenance costs will vary with the type of bridge, climate, and region; embankment maintenance costs (exclusive of flood repairs) will be a function of rainfall and other happenings that bring erosion and parallel deterioration. In this study, these maintenance items have been charged as an annual cost per lineal foot of bridge or embankment.

SELECTING LEAST COSTLY COMBINATION

In this study, cost comparisons are made between bridges of several selected lengths. In turn the bridge of each length has several alternative approach embankments of different heights. The first step in the analysis is to determine, for each bridge length, the least costly embankment height. Then the total costs of bridges of different lengths are compared, each with its most favorable embankment arrangement. The tables accompanying the report show in detail how the various costs are computed. In an actual cost study some of the columns and tables can be combined to simplify the computations.

In an economy study such as this, cost comparisons should be between alternative bridge-roadway combinations of equal length. In cases where, because of differences in approach embankment height, the bridge plus embankment lengths differ among alternatives, pavement lengths have been increased for the shorter alternatives to give each the same over-all length. Again, an economy study is concerned with differences between alternatives. It is differences in costs that are relevant. This means that costs common to all alternatives may be ignored as far as choosing the most attractive alternative is concerned. Furthermore, it is often proper to employ a "with" and "without" approach. For example, this is done with backwater damage costs for each combination of bridge and embankment. With large floods, some damage will probably occur with no bridge at all; this is the base condition. Only the increment of damage resulting because of each bridge-embankment combination is pertinent and is computed.

Costs Related to Embankment Height

There are several annual costs included in most economy studies of approach embankments: (a) capital recovery for embankment, (b) embankment maintenance, (c) expected flood damage to embankment, (d) expected detouring, and (e) expected increment of backwater damage. Any other variables that might affect vehicle-operating or other costs in a particular case should also be included.

Method for Predicting Expected Average Annual Damage

A numerical procedure suggested by Franzini (6) is used to evaluate the annual expected flood damage. A typical annual probability-damage curve is shown as Figure 11. The probability axis is divided into elements P_1, P_2----P_n. For each probability P_1 there is a damage d_1. The area of a typical element 1-2 is given by

$$\text{Elemental area 1-2} = \left(\frac{D_1 + D_2}{2}\right)(P_2 - P_1).$$

The sum of all elemental areas under the probability-damage curve is the expected annual cost. A method for summing these elemental areas is shown in Table 1.

Method of Computing Embankment Costs

Cost comparisons are made on an annual cost basis; each of the embankment costs is listed with a brief discussion and explanation of how it is computed.

Annual cost of capital recovery for embankment. —Annual cost = (first cost) x (crf-i-n), where first cost = total cost of embankment and paving; (crf-i-n) = capital recovery factor for interest rate i and analysis period n. The example problems are solved at an interest rate of 7 percent, a period of 30 years for bridge and embankment and with zero salvage value (see Woods (5) or Grant and Ireson (7) for detailed procedures for economy studies and for compound interest tables.)

Annual embankment maintenance. -Embankment maintenance costs have been assumed to be proportional to embankment length. They were set at \$0.30 per lineal foot, based on maintenance cost figures supplied by G. S. Paxson of the Oregon Highway Department. This figure is approximate and may be low because it is not necessarily for embankments subject to flooding.

Annual expected embankment damage. -These costs are for repairing damage caused by flood flows overtopping the embankment. Anticipated annual costs decrease as embankment heights increase because overtopping of higher embankments is less frequent.

TABLE 1

COMPUTATION OF EXPECTED ANNUAL FLOOD DAMAGE

Damage D	$\frac{D_1 + D_2}{2}$	Probability, P	$(P_2 - P_1)$	$\frac{D_1 + D_2}{2}(P_2 - P_1)$
D_1		P_1		
	$\frac{D_1 + D_2}{2}$		$(P_2 - P_1)$	$\frac{(D_1 + D_2)(P_2 - P_1)}{2}$
D_2		P_2		
	$\frac{D_2 + D_3}{2}$		$(P_3 - P_2)$	
D_3		P_3		
	.			.
	$\frac{D_N\text{-}1\text{+}D_N}{2}$		$(P_N - P_N - 1)$	$\frac{D_{N-1} + D_N \, P_N\text{-}P_{N-1}}{2}$
D_N		P_N		

$$\text{Expected annual damage} = \sum_{i=1}^{i=N-1} \frac{D_i + D_{i+1}}{} \left(P_{i+1} - P_i\right)$$

TABLE 2

COMPUTATION OF EXPECTED ANNUAL EMBANKMENT DAMAGE

Bridge Length	Emb Elev	Emb Cost	Bridge Stage No	Flow (cfc)	Stage at Embankment	Energy Head Above Embankment	Percent Damage to Embankment	Increment of Average Percent Damage	Increment of Average Damage Cost	Probability of flow Occurring	Increment Probability	Increment Embankment Damage
(a)	(b)	(c)	(d)	(e)	(f)	(g)	(h)	(i)	(j)	(k)	(l)	(m)

Total annual embankment damage cost = $\sum$ (m)

TABLE 3

COMPUTATION OF EXPECTED ANNUAL DETOUR COSTS DURING EMBANKMENT REPAIR

Bridge Length	Embankment Elev	Flow (cfs)	Increment Average % Damage to Embankment	Increment Probability	Increment Average Time to Repair	Increment Average Detour Cost	Increment Detour Cost
(a)	(b)	(e)	(j)	(l)	(n)	(o)	(p)

Average annual detour cost during repair = $\sum$ (p)

TABLE 4

COMPUTATION OF EXPECTED ANNUAL COST TO DETOURED TRAFFIC

Bridge Length	Embankment Elev.	Flood Routing Stage	Flow (cfc)	Days Above Stage	Times Above Stage	Average Days per Time	Cost per Time	Probability of Occurrence	Expected Cost of Detoured Traffic
(a)	(b)	(c)	(d)	(e)	(f)	(g)	(h)	(i)	(j)

TABLE 5

COMPUTATION OF EXPECTED ANNUAL INCREMENTAL OF BACKWATER DAMAGE CAUSED BY BRIDGE

Bridge Length	Embankment Elev.	Stage No. Bridge	Flow (cfc)	Increment of Backwater to Cause Damage	Incremental Backwater Damage	Average Incre-mental Damage	Probability of Occurrence	Incre-mental Proba-bility	Incre-mental Damage
(a)	(b)	(c)	(d)	(e)	(f)	(g)	(h)	(i)	(j)

SUMMARY AND RESULTS OF EX

	Row						
Bridge length (ft)	(A)	800	800	800	800	1,100	1,100
Max backwater (ft)	(B)	0 5	1 0	2 0	2 5	0 5	1
Embank elev (ft)	(C)	887 5	890 5	893 7	895 2	888 9	892
Embank length (ft)	(D)	4,400	4,500	4,600	4,625	4,150	4,250
Embank and paving cost ($)	(E)	61,000	76,000	92,500	103,000	64,750	82,700
Length of paving for equal length (ft)	(F)	225	125	25		175	75
Paving cost (at 50,000/mi) ($)	(G)	2,131	1,184	237		1,657	710
Total cost embankment paving ($)	(H)	63,131	77,184	92,737	103,000	66,407	83,410
Expected Annual Costs, embankment ($)							
Capital recovery, embank and paving	(I)	5,088	6,220	7,474	8,301	5,352	6,722
Embank maintenance (at $0 30/ft)	(J)	1,320	1,350	1,380	1,388	1,245	1,275
Flood damage to embankment	(K)	1,221	605	239	167	777	276
Detour during repair	(L)	3,876	1,846	728	463	2,698	934
Detour during flood	(M)	4,995	3,042	1,261	764	4,004	1,800
Increment backwater damage	(N)	866	2,132	3,394	3,725	1,055	1,850
Total	(O)	$ 17,366	15,195	14,476	14,808	15,131	12,857
Expected annual costs, embank and bridge							
Bridge length (ft)	(P)	800	1,100	1,500		2,000	2,
Bridge cost ($)	(Q)	460,000	525,000	680,000		875,000	1,070,
Length of combination (ft)	(R)	5,400	5,350	5,375		5,400	5,
Length of pave for equal length (ft)	(S)	0	50	25		0	
Capital recovery of bridge invest ($)	(T)	37,071	42,310	54,801		70,516	86,
Added pave (capital recovery) (Length x 9 4,797/ft x 0 08059) (crf - 7% - 30)	(U)	0	38	19		0	
Bridge maintenance at $0 $0/ft	(V)	160	220	300		400	5
Embankment ($)	(W)	14,476	12,857	11,518		9,730	8,
Total ($)	(X)	51,707	55,425	66,638		80,646	95,
Present worth (pwf - 30 - 7%) = 12 409	(Y)	$641,632	687,769	826,910		1,000,736	1,178,

SUMMARY AND RESULTS O

	Row				
Bridge length (ft)	(A)	100	100	100	10
Max backwater (ft)	(B)	0.5	1.0	1.5	
Embank elev (ft)	(C)	834.7	837 3	839.7	84
Embank. length (ft)	(D)	490	600	690	75
Embankment cost (including paving) ($)	(E)	7,750	10,700	13,500	15,50
Paving needed for equal length projects (ft)	(F)	310	200	110	5
Paving cost (at $50,000/mi) ($)	(G)	2,939	1,896	1,043	47
Total cost embankment and paving ($)	(H)	10,689	12,596	14,543	15,97
Expected annual costs, embankment ($),					
Capital recovery (embankment and paving) (crf-7%-30 = 0 08059)	(I)	861	1,015	1,172	1,28
Embankment maintenance (at $0 30/ft)	(J)	117	180	207	21
Flood damage to embankment	(K)	214	109	64	5
Traffic interruption during embankment repair (based on $5,000 for detour/day)	(L)	1,265	573	285	19
Traffic interruption during flood (based on $5,000 per detour/day)	(M)	3,400	1,530	712	29
Increment backwater damage	(N)	1,733	3,189	3,740	3,94
Total	(O)	7,590	6,596	6,180	5,98
Expected annual costs, embankment and bridge					
Bridge length (ft)	(P)	100	150	200	30
Bridge cost ($)	(Q)	48,300	69,300	90,300	123,30
Combination length (ft)	(R)	900	950	960	91
Length of pavement for equal length (ft) added pavement ($)	(S)	60	10	-	5
Bridge capital recovery		569	95	-	47
(crf-7%-30 = 0 08059) ($)	(T)	3,892	5,585	7,277	10,66
Added pavement capital recovery ($)	(U)	46	7	-	3
Bridge maintenance (at $0 50/ft)	(V)	50	75	100	15
Embankment ($)	(W)	5,989	4,012	2,839	1,43
Total ($)	(X)	9,977	9,679	10,216	12,28
Present Worth (pwf-30-7%) = 12 409 ($)		123,805	120,107	126,770	152,44

6

LE PROBLEM 1 (LONG BRIDGE)

100	1,100	1,500	1,500	1,500	2,000	2,000	2,000	2,500	2,600	2,500	2,500
1 5	2 0	0 5	1.0	1 5	0 5	1.0	1 1	0 23	0 5	0 75	1 0
893 5	895.5	890 3	893.4	895.6	891 9	894.5	896 0	890 0	893 2	895 0	896 6
275	4,325	3,800	3,875	3,950	3,350	3,400	3,450	2,775	2,850	2,900	2,950
750	103,000	66,750	84,000	98,500	67,750	81,750	91,000	53,500	65,200	74,000	82,000
50		150	75		100	50		175	100	50	
473		1,420	710		947	473		1,657	947	473	
223	103,000	68,170	84,710	98,500	68,697	82,223	91,000	55,157	66,147	74,473	82,000
352	8,301	5,494	6,827	7,938	5,536	6,626	7,334	4,445	5,331	6,002	6,608
282	1,297	1,140	1,162	1,185	1,005	1,020	1,035	832	855	870	885
205	105	445	159	82	238	82	59	371	112	59	44
691	356	1,568	566	249	800	234	154	1,592	423	214	133
225	576	3,089	1,040	470	1,604	576	246	3,042	930	333	80
410	2,682	1,386	1,764	2,026	976	1,192	1,204	462	726	800	832
165	13,317	13,122	11,518	11,950	10,159	9,730	10,032	10,744	8,377	8,278	8,582

BLE 7

MPLE PROBLEM 2 (SHORT BRIDGE)

100	150	150	150	200	200	200	300	300	300	300
2.5	1.0	1 5	2.0	0.5	1.0	1.5	0.1	0.2	0.3	0.4
842.3	839.3	841 5	843.7	838 3	841.8	844.0	837.0	839.5	841.3	842.7
800	620	710	800	530	670	760	380	480	560	610
,700	11,800	14,250	16,900	9,300	13,300	15,700	5,500	7,750	9,300	10,700
-	180	90	-	230	90	-	230	130	50	-
-	1,706	853	-	2,180	853	-	2,180	1,232	474	-
,700	13,506	15,103	16,900	11,480	14,153	15,700	7,680	8,982	9,774	10,700
,346	1,088	1,217	1,362	925	1,141	1,265	619	724	788	862
270	186	213	240	159	201	228	114	144	168	183
56	37	24	25	35	20	8	41	15	9	8
175	193	88	71	226	65	23	303	81	36	24
178	510	157	46	690	83	20	1,160	284	88	32
,089	2,124	2,313	2,363	1,166	1,329	1,346	258	326	346	403
,114	4,138	4,012	4,107	3,201	2,839	2,890	2,495	1,574	1,435	1,512

TABLE 8

SAMPLE COMPUTATION FOR PROBLEM 1 (1,100-FT BRIDGE), COST OF FLOOD TO EMBANKMENT AND OF DETOUR COSTS DURING REPAIRS[1]

Bridge Length (ft)	Embankment Elev. (ft)	No Bridge Stage	Q (000 cfs)	Stage at Embankment	Water Head over Embankment	% of Damage to Embankment	Ave % Damage to Embankment	Ave Damage Costs ($)	Ave Time to Repair (days)	Ave Detour Costs	Prob of Occurrence	Increment Probability	Increment Emb Damage ($)	Increment Detour Costs ($)
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)
1,100	888 9		140	888 9	0	0					0.10			
							5 8	3,756	0.58	5,800		0.065	244	377
		891 0	205	891 22	2 32	11 6					0 035			
							14 05	9,097	1 405	14,050		0 005	45	70
		892 0	235	892 20	3 30	16 5					0 030			
							18 95	12,270	1 895	18,950		0 013	160	246
		893 0	270	893 18	4 28	21 4					0 017			
							23 9	15,475	2 39	23,900		0 008	124	191
		894 0	310	894 18	5 28	26 4					0 009			
							28 9	18,713	2 89	28,900		0 005	94	144
		895 0	360	895 18	6 28	31 4					0 004			
							33 9	21,950	3 39	33,900		0 003	66	102
		896 0	450	896 18	7 28	36 4					0 001			
							68 2	44,159	6 82	68,200		0 001	44	68
			largest	-	-	100 0					0 0			
											Total		777	1,198

[1]Embankment cost = $64,750

TABLE 9

SAMPLE COMPUTATION FOR PROBLEM 1 (1,100-FT BRIDGE) DETOUR COSTS DURING FLOOD

Bridge Length (ft)	Embankment Elev (ft)	Routing Stage of Bridge	Flow (000 cfs)	Days Above Stage	Times Above Stage	Average Days per Time	Cost per Time ($000)	Probability of Occurrence	Cost of Detoured Traffic ($)
(a)	(b)	(c)	(d)	(e)	(f)	(g)	(h)	(i)	(j)
1,100	888 9	887 9	129	18 2	5	3 64	3 64	0 11	4,004

TABLE 10

SAMPLE COMPUTATION FOR PROBLEM 1, (1,100-FT BRIDGE), INCREMENT OF BACKWATER DAMAGE CAUSED BY BRIDGE

Bridge Length (ft)	Embankment Elev (ft)	No Bridge Stage	Flow (000 cfs)	Increment of Backwater to Cause Damage	Incremental Backwater Damage	Average Damage ($)	Probability of Occurrence	Incremental Probability	Incremental
(a)	(b)	(c)	(d)	(e)	(f)	(g)	(h)	(i)	(j)
1,100	888 9	883 0	80	0 1	0		0 20		
						2,600		0 06	156
		886.0	110	0 26	5,200		0 14		
						7,600		0 04	304
		888 0	140	0 5	10,000		0 10		
						7,200		0 065	468
		891 0	205	0 22	4,400		0 035		
						4,200		0 005	21
		892 0	235	0 20	4,000		0 030		
						3,800		0 013	50
		893 0	270	0 18	3,600		0 017		
						3,600		0 008	28
		894 0	310	0 18	3,600		0 009		
						3,600		0 005	18
		895 0	360	0 18	3,600		0 004		
						3,600		0 003	10
		896 0	450	0 18	3,600		0 001		

Estimated Annual Backwater Damage = $1,055

TABLE 11

SUMMARY OF EFFECT OF EMBANKMENT HEIGHT ON VARIOUS EMBANKMENT COSTS

Embankment	Embankment Cost	Maintenance Cost	Damage to Embankment	Traffic Routing Costs	Increment Backwater Costs
High	Higher	Higher	Lower	Lower	Higher
Low	Lower	Lower	Higher	Higher	Lower

TABLE 12

COMPARISON OF ESTIMATED COSTS FOR BRIDGES OF VARIOUS LENGTHS

Bridge Length (ft)	Total Expected Annual Cost ($)	Present Worth ($)	Percent Savings of Most Economical Bridge
Example 1:			
800	51,710	641,630	0.00
1,100	55,430	687,770	7.20
1,500	66,640	826,910	28.9
2,000	80,650	1,000,740	56.0
2,500	95,010	1,178,970	83.8
Example 2:			
100	9,980	123,810	3.1
150	9,680	120,110	0.0
200	10,220	126,770	5.5
300	12,290	152,450	26.9

For Example 1 damage costs were assumed to be 5 percent of the total embankment cost cost for each foot of flow energy above the embankment roadway elevation.

Very little has been published concerning damage to embankments from overtopping. Kindsvater (8) reports how embankment damage by flood waters occurs and Yarnell and Nagler (9) give some examples of damages from flood flows.

The computation for embankment damage is an application of the method described earlier for evaluating annual expected damage. The embankment damage computation can be set up as shown in Table 2. (A sample calculation for this item combined with detour costs, appears as Table 8.) Explanation of Table 2 where the headings may not be fully descriptive are as follows:

Col. (c) Embankment costs taken from Figure 3.
Col. (d) No bridge stage. These are water surface elevations without the bridge and embankment. They are computed for selected values of Q (flow) as given in Col. (e) see Fig. 9).

Col. (f) Stage at embankment is found in two steps. First, the rise in the water surface resulting from backwater after the fill is overtopped is read from Figure 5, for this appropriate roadway (approach fill) elevation and bridge length. This value is added to the water-surface elevation, without bridge, as shown in Col. (d). It is an approximation of the flow energy line because Figure 4 is computed without embankment overflow.

Col. (g) Head above the embankment is the stage at the embankment minus the embankment elevation:

$$\text{Col. (g)} = \text{Col. (f)} - \text{Col. (b)}.$$

Col. (h) Percent damage to embankment is an assumed constant stated as percent damage per foot of energy head above embankment times the head above the embankment (Col. g).

Col. (h) = k. Col. (g) in which k = percent damage per foot energy head above embankment.

Col. (i) Increment of average percent damage is the average between the successive rows in Col. (h).

$$\text{Col. (i)}_{1-2} = \frac{\text{Col. (h)}_1 + \text{Col. (h)}_2}{2}.$$

Col. (j) Increment of average embankment damage cost is the average percent damage times the cost of the embankment.

$$\text{Col. (j)} = \text{Col. (i)} \times \text{Col. (c)}.$$

Col. (k) Probability of flow occuring is taken from the frequency curve (Fig. 6) for the flows found in Col. (e).

Col. (l) Incremental probability is the difference between successive rows of flow probabilities.

$$\text{Col. (l)}_{1-2} = \text{Col. (k)}_1 - \text{Col. (k)}_2.$$

Col. (m) Incremental embankment damage is the product of the increment average damage times the incremental probability.

$$\text{Col. (m)} = \text{Col. (j)} \times \text{Col. (l)}.$$

Expected annual flood damage to the embankment is the sum of all incremental embankment damages.

$$\begin{array}{c}\text{Annual}\\ \text{Embankment}\\ \text{Damage}\end{array} = \sum_{i=1}^{n} \text{Col. (m)}_i$$

Annual expected detour costs.—Detour occurs when flood waters are of sufficient stage that traffic cannot cross the bridge and embankment. The delays caused by flood are divided into three types: (a) flood detour, (b) recession detour, and (c) repair detour.

Traffic rerouting is assumed to occur when the flood waters reach an elevation somewhat below the embankment roadway elevation. Any time that a flood is above this stage traffic is to be detoured. The cost of routing vehicles during these flood stages is computed separately under the heading of annual expected detour cost during flood.

If the flood has a stage above the roadway elevation, it is assumed to cause embankment damage. If damage occurs, traffic will be detoured during the time the flood recedes from the flood detour elevation to the elevation where repair can take place (recession detour) and also during the time of repair (repair detour). Recession and repair detours are closely related so both are included in the computation of annual expected detour cost during embankment repair.

Annual expected detour costs during embankment repair. – The detour cost during embankment repair is the added cost for vehicles and drivers caused by the detour plus detour set-up and maintenance costs. The detour time in this paper was assumed to be directly proportional to the damage. For instance, in Example 1, the detour time was assumed as 1 day for each 10 percent embankment damage. The traffic detour cost per day was set as a flat sum; no detailed computations were made for it.

Repair detour costs and embankment damage costs can be computed in the same table. The detour cost computation columns are shown separately in Table 3 in order that the procedure can be followed more easily. Table 8 is a calculation from Example 1; this shows how Tables 2 and 3 look when combined.

The computation procedure for Table 3 is described as follows:

Columns (a), (b), (e), (j), and (l) are taken from Table 2, Annual Expected Embankment Damage.

Col. (n) Incremental average time to repair is the product of the incremental average percent damage to the embankment times the time to repair for a given percent damage.

Col. (o) Incremental average detour cost is the product of the incremental average time to repair times the cost per day of detour.

Col. (p) Incremental detour cost is the product of the incremental probability times the incremental average detour cost.

Col. (p) = Col. (l) x Col. (o).

Total annual expected detour cost is the sum of all figures in Col. (p) plus the annual expected cost of detour while the flood causing damage is receding before repair (recession detour).

The recession cost was assumed to be the product of the detour costs during the time the flood recedes multiplied by the annual probability of having a flood of magnitude to cause damage. The recession time is the time for the flood water to recede from flood detour elevation to an elevation where embankment repair can begin. This time was assumed as constant in the examples.

$$\begin{pmatrix}\text{Recession}\\ \text{Cost}\end{pmatrix} = \begin{pmatrix}\text{time for}\\ \text{recession}\end{pmatrix}\begin{pmatrix}\text{cost/unit for}\\ \text{detour time}\end{pmatrix}\begin{pmatrix}\text{annual probability of}\\ \text{damage occurring}\end{pmatrix}$$

For instance, in Example 1 of this paper, the time to recede is assumed as $1\frac{1}{2}$ days at a cost for detouring of $10,000 per day. Thus the expected annual cost of detouring is $15,000 times the probability of a flood of stage above the embankment roadway elevation. For example, the $2,698 shown in Col. 5 under Item (L) in Table 6 equals the sum of $1,198 from Col. (15) of Table 8 and $15,000 x 0.10.

Annual expected traffic detour costs during floods. – The annual cost of detouring during floods is the product of the annual probability of having a flood equal to or higher than the flood routing stage times the cost per occurrence of detouring for the days above this stage. The number of days a flow has been exceeded (see Fig. 7) and the number of times a flow has been exceeded (see Fig. 8) and the detour cost per day will be available.

The computation form for annual expected detour cost during floods is shown in Table 4 and Table 9. The columns are described as follows:

Col. (c) Flood routing stage is the flow when detouring begins. This detouring begins when the water surface at the embankment is some assumed distance below the elevation of the embankment.

Col. (d) Flow is taken from the water surface elevation at the upstream embankment slope curve (Fig. 4).

Col. (e) Days above stage is taken from Figure 7 for the flow given in Col. (d).

Col. (f) Times above stage is taken from Figure 8 for the flow given in Col. (d).

Col. (g) Average days per time is the ratio of days exceeded per flow to times exceeded per flow.

Col. (g) = Col. (e) ÷ Col. (f).

Col. (h) Cost per time is the product of detour days per time (Col. g) times the cost of detouring per day.

Col. (h) = c times Col. (g),
where c = cost/day of detouring.

Col. (i) Probability of occurrence is taken from the flood frequency curve (Fig 6) for the flow in Col. (d).

Col. (j) Expected annual cost of detouring traffic because of flood is the product of the cost per time (Col. h) times the probability of occurrence.

Col. (j) = Col. (h) x Col. (i).

Annual expected incremental backwater damage cost. – This cost is the difference in damage costs between the annual expected flood damage that would occur with a given bridge and approach embankment and the annual expected flood damage in the natural stage without the bridge.

Calculation of the backwater damage cost is another evaluation of the annual expected damage by numerical integration. The calculation form for backwater damage cost is shown in Table 5 (see Table 10 also). The various columns in Table 5 are described as follows:

Col. (c) Stage without bridge is the normal stage (see Fig. 9).

Col. (d) Flow is for the stages found in Col. (c).

Col. (e) Increment of backwater to cause damage is found from Figure 5. After the flood stage reaches embankment elevation, the backwater effect will follow the receding curve for increased flows.

Col. (f) Incremental backwater damage is the difference between the damage for the stage with incremental backwater Col. (e) plus normal stage Col. (c) and the damage at normal stage Col. (c). These damages are found from the stage-damage curve (Fig. 10) for the respective stages.

Col. (g) The average incremental damage is the average of successive rows in Col. (f).

$$\text{Col. } (g)_{1-2} = \frac{\text{Col. } (f)_1 + \text{Col. } (f)_2}{2}$$

Col. (h) Probability of occurrence is the probability of the flows in Col. (d). This is taken from the frequency curve (Fig. 6) for the respective flows.

Col. (i) Incremental probability is the difference between successive rows of flow probabilities.

$$\text{Col. } i = (\text{Col. } h)_1 - (\text{Col. } h)_2$$

Col. (j) Incremental damage is the product of the average incremental damage (Col. g) times the incremental probability (Col. i).

Col. (j) = Col. (g) x Col. (i).

The total incremental backwater damage caused by the bridge and embankment is the summation of incremental damages found in Col. (j).

$$\text{Backwater damage} = \sum_{i=1}^{n} \text{Col. } (j)_i$$

Method for Finding the Most Economical Embankment Height

The procedure for finding the most economical embankment height for a given bridge length is to choose the embankment heights to be compared, evaluate the various embankment costs for these heights (see Table 6), and plot embankment height vs cost (see Fig. 12). The most economical embankment will be the minimum point on the summation curve. If the most economical embankment height is not included in those for which costs have been developed it may be necessary to compute the costs of other embankment heights. Finally, the costs of the most economical embankment height are evaluated as a check, by using the normal calculation methods for finding embankment costs. To illustrate, for the 1,100-ft bridge in Example 1, embankment costs for an elevation of 892.2 can be computed to be $12,860, which checks Figure 12.

Costs Affecting the Most Economical Combination of Bridge Length and Embankment Height

There are two bridge costs that will be common to all economy studies: (a) annual capital recovery cost of bridge and (b) annual bridge maintenance cost. Methods for evaluating these have already been outlined.

Total costs for a given alternative bridge length are the sums of embankment costs plus bridge costs. Because the total length of bridge plus embankment roadway for the compared alternatives must be the same, a length of roadway must be added to the shorter alternatives to make the compared project lengths equal. The capital recovery cost of extra pavement is added to the other bridge costs. Often the added pavement cost is small and may be ignored.

Selection of the Most Economical Alternative

The lower portion to Table 6 summarizes total and annual costs for the most economical combination of bridge and embankment. In it the annual costs of the least costly embankment for each bridge length is combined with the annual costs associated with the bridge (and added pavement length). The combination with the least total annual cost is most desirable from an economy point of view.

ANALYSIS

Findings of this study favor the 800-ft bridge in Example 1 and the 150-ft bridge in Example 2. As stated earlier, there well may be "irreducibles" that cannot be put in money terms. The final choice of bridge length will be made by weighing both the "dollar considerations" outlined here along with other important factors.

The graphs for embankment costs (Figs. 12 and 19) show that costs increase quite slowly with small departures from the economical embankment height. This indicates, for these examples at least, that embankments a foot or so higher or lower than the "most economical" represent acceptable alternatives. Table 11 summarizes the effects of embankment height on the individual cost items that make up total embankment costs. Such a table may prove useful in selecting embankment height for the final design.

Bridge length, the other principal variable in the analysis, makes a significant difference in total annual cost. This is indicated clearly by Table 12. Results of both examples favor short rather than long bridges. It would seem that in spite of the many uncertainties in the data on which the analysis is based, such a study warrants the time and effort it requires, particularly if it questions present practices.

In the two examples, the effects of channel scour resulting from high velocities were not considered in the calculation of bridge backwater nor in the economy study. This might be an important design or cost factor in some instances. For example, velocities under a short bridge with high approach embankments might be so great as to require expensive channel and slope protection. The overtopping of low approach fills reduces the velocities under the bridge and therefore reduces scour. Even so, where velocities are high enough to threaten stream bed or embankment erosion, the analysis must be modified to recognize design changes and cost factors. Bradley (1) has a discussion on the effects of scour and how to allow for it in backwater computations.

The authors have concentrated on developing an economy-study procedure. They recognize that this procedure involves a considerable amount of routine computation. However, with electronic computers readily available to carry out such manipulations, computation time becomes of little importance.

CONCLUSIONS

An economy study, basically the same as the one in this paper, could be used to good advantage in the design of many bridge and approach embankment combinations. The writers acknowledge that some of the methods proposed here for evaluating costs are at best approximate. Often they were assumed without supporting data. It is to be presumed that other more direct and accurate ways of obtaining them are available to engineers in the various highway agencies; if so, these better methods should be used. However, the principles for the economy study remain the same.

A literature search indicates that research is needed at least in three areas before reliable cost data will be available for studies such as these:

1. A sound basis on which to evaluate flood damages so that reliable stage-damage curves can be constructed. Joint efforts with other agencies concerned with this problem should be fruitful.
2. More knowledge of the behavior of embankments when they serve as spillways so that reliable estimates of first cost and damage can be made.
3. Better measures for determining the market and extra market costs that accompany rerouting of or delays to traffic. Considerable work is currently under way in this field and results should be forthcoming in the near future.

It is to be observed that the importance of items 1 and 3 is minimized on low-volume highways in rural areas. Thus, an analysis such as proposed here, supported by the underlying hydrologic and hydraulic studies, seems particularly appropriate for major bridges on rural farm to market and other secondary roads.

All things considered, efforts towards collecting the supporting data and in making economy studies such as proposed in this paper should lead to better grounds for decision making by highway engineers.

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Discussion

GENE E. WILLEKE, Hydraulic Research Engineer, Division of Hydraulic Research, Bureau of Public Roads. —It is refreshing to have a highway problem in which hydrology is less uncertain than some of the other factors.

One point that stands out very clearly is the insensitivity of change in backwater to a change in bridge length. A considerable change in bridge length has a small effect on the amount of backwater. The experimental errors inherent in the development of the procedure for computation of backwater would lead one to question a bridge length determination based on such a procedure. This is especially true in the case of the examples given in this paper in which all costs other than capital recovery and routine maintenance for the long bridge amount to less than 11 percent of the total cost. The same figure for the short bridge is less than 27 percent.

Although all the figures are quite fictitious, the evidence presented would certainly indicate that backwater computations are a poor criterion for bridge length determination and that a search for better criteria is in order.

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