DEVELOPING USER COSTS FOR BRIDGE MANAGEMENT SYSTEMS

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ABSTRACT

Evaluation of bridges for improvement in bridge management systems to meet expectations of ISTEA legislation and AASHTO guidelines depends on accurate estimates of various user and agency costs associated with both the existing structure and the improved or replaced structure. This paper summarizes methods developed for determining the user costs associated with deficiencies in load capacity, deck, approach and vertical clearance geometry.

BACKGROUND AND OBJECTIVES

Consideration of user costs is essential in Bridge Management Systems (BMSs) if functional deficiencies are to be eliminated. If agency costs alone are considered, the alternatives would tend to favor maintenance only to extend life until permanent closure. The objective of this paper is to outline an approach for estimating user costs generated by deficient bridges. This effort was initiated in 1983 when North Carolina began the development of methodologies for evaluating alternatives for bridge maintenance and improvement based upon economic analysis (*1,2*). These concepts are embodied in the OPBRIDGE analysis program (*3,4,5*), a major component of the North Carolina BMS. Due to length constraints, this paper will primarily reference summary reports of the authors (*1,6,7*). The reader is encouraged to refer to those reports for a more detailed development of each topic and a more thorough citation of other studies and sources of data. Some user costs involve parameters that must be periodically updated. One example is the operating costs of vehicles. In such cases, a priority was placed on identifying a source that could be easily referred to for an update. Usually, improvements in the methodology can be made by research that could provide more accurate data for individual parameters, or which could better define the parameters in a manner tailored to the user traffic of the individual states or other owning agencies. Nevertheless, the efforts summarized here have proved valuable in quantifying user costs for North Carolina and have provided a guide to others trying to conduct similar analyses.

TYPES OF USER COSTS

User costs can be generated by such bridge deficiencies as narrow width, low clearance, poor alignment and low load capacity. Bridges with narrow width, low clearance, or poor alignment induce vehicle accidents. Bridges with low clearance and low load capacity cause some vehicles to be detoured. The costs accumulate independently for both the over-route and the under-route roadway. If user costs incurred are assumed to be proportional to traffic volume and the level of service deficiency of the bridge, the user costs in any given year, *t*, can be derived as follows:

$$AURC(t) = 365\, ADT(t)\, [C_{WDA} U_{AC} + C_{ALA} U_{AC} + C_{CLA} U_{AC} + C_{CLD} U_{DC} DL + C_{LCD}(t)\, U_{DL} DL] \qquad (1)$$

where:

AURC(t) = annual user cost of the bridge at year *t*, \$;

ADT(t) = average daily traffic using the bridge at year *t*;

C_{WDA} = coefficient for proportion of vehicles incurring accidens due to width deficiency;

C_{ALA} = coefficient for proportion of vehicles incurring accidents due to poor alignment;

C_{CLA} = coefficient for proportion of vehicles incurring accidents due to a vertical clearance deficiency;

C_{CLD} = coefficient for proportion of vehicles detoured due to a vertical clearance deficiency;
$C_{LCD}(t)$ = coefficient for proportion of vehicles detoured due to a load capacity deficiency at year t;
U_{AC} = unit cost of vehicle accidents on bridges, \$/accident;
U_{DC} = unit cost for average vehicle detours due to a vertical clearance deficiency, \$/mile, (\$/km);
U_{DL} = unit cost for average vehicle detours due to a load capacity deficiency, \$/mile (\$/km); and
DL = detour length, miles (km).

For bridges with the same level-of-service deficiency, the one having greater ADT would generate, proportionally, higher user costs because of the higher probabilities of causing detours and accidents. For some user costs, the traffic affected is only the truck traffic. However, since average daily truck traffic, ADTT, is usually not in the bridge data file, the various coefficients are used to estimate the appropriate segment of traffic affected based on total ADT. The coefficients C_{WDA}, C_{ALA}, C_{CLA}, C_{CLD}, and C_{LCD}, of Equation 1 are assumed constant during the service life of a bridge unless action is taken to reduce the deficiencies. However, C_{LCD} may vary with time; if load capacity of the bridge deteriorates, the proportion of vehicles detoured increases. The coefficients, ADT and DL vary for the over-route and the under-route computations. C_{LCD} is zero (0) for the under-route. This paper describes the efforts to quantify the coefficients in Equation 1, the factors that influence the ADT increase rates for different functional classification routes, and the derivations of user costs due to the load capacity, deck width, alignment and vertical clearance deficiencies.

DETOUR LENGTH AND DETOUR UNIT COSTS

The route detour length listed in the National Bridge Inventory (NBI) is the bypass detour distance that a vehicle must travel for a closed and detour-posted bridge. However, the actual detour may be more for a load- or clearance-posted bridge. If the driver is not aware of the low capacity or clearance bridge, the detour would be longer since the posting is usually only placed at the bridge and not at the possible detour turnoff. If the detour route involves a posted bridge, the detour could increase even further. There are many possible permutations that would vary with drivers' knowledge of the route, destination, layout of the roadways, possibility of a posted bridge on the detour route, etc. For this analysis, the actual detour length, DL, is nevertheless assumed to be the detour length recorded in the inventory file. However, one could argue that this value is an underestimate.

Vehicle operating costs can vary due to vehicle characteristics and operator wage rates. Recognizing that the values would have to be updated periodically, easily referred to sources were desired. To estimate operating costs for all vehicles, two limiting extremes were established. The upper end vehicle was assumed to be a truck tractor semi-trailer (TTST) vehicle at the legal load limit of 36.7 tons (329 kN) and the lower end was assumed to be a vehicle weighing less than 3 tons (27 kN). Operating cost variations were then assumed linear with weight between these values since weight reflects both fixed costs and energy requirements and also need for operator skill.

Reliable data on operating costs for trucks in different weight ranges are limited (*1*). The trucking industry is regulated and truckers do not publish their actual costs since they are a part of the negotiations. The U.S. Department of Agriculture regularly compiles cost data on long distance haul fruit and vegetable trucks having a tractor-trailer configuration. The cost report for the fruit and vegetable trucks consists of fixed and variable costs and the total estimated operating cost per vehicle-mile. According to the cost report of May 1991, the estimated operating cost was \$1.28/mile (\$0.80/km), including the driver salary. The FHWA Office of Planning, Highway Performance Monitoring Branch, periodically publishes data on operating costs for various truck types and weights. A similar value for trucks at the legal weight limit can be deduced from this information, as shown by Abed-Al-Rahim and Johnston (*6,7*).

The average operating costs for passenger cars, small pickup trucks, and other vehicles weighing up to 3 tons (27 kN), were assumed to be equal. This lower end cost includes two components: vehicle and operator costs. The vehicle cost was assumed to be the same as the Federal IRS tax allowance for business use of passenger cars, currently \$0.28 per mile (\$0.17/km). The light truck operator cost was assumed to be the wage rate of a North Carolina State Government employee level one vehicle operator. Including fringe benefits and assuming a 48-week work year, 40 hours per week, and a speed of 40 mph (64 km/hr), this results in an operator cost of \$0.18 per mile (\$0.11/km) and a total average operating cost of \$0.46 per mile (\$0.28/km).

If the relationship between the vehicle operating cost and the vehicle weight is assumed to be linear, the following equation for vehicle operating cost could be deduced:

$$U_{DW} = U_{D3} + (U_{DNP} - U_{D3})\,\frac{(W - 3tons)}{(NP - 3tons)} \quad (2)$$

where:

U_{DW} = operating cost for vehicle of weight W, \$/mile (\$/km);
U_{D3} = operating cost for vehicle weighing 3 tons (27 kN) or less, \$/mile (\$/km);
U_{DNP} = operating cost for vehicle weighing the maximum legal load, \$/mile (\$/km);
NP = maximum legal load or non-posted capacity of bridge, tons (kN); and
W = weight of vehicle, tons (kN).

One method for calculating the total cost of the vehicle detours due to load capacity deficiency is to multiply the average operating cost of the detoured vehicles by the detour length and the number of vehicles detoured, as indicated in Equation 1. If the distribution of vehicles above 3 tons (27 kN) is about uniform by weight, the average operating cost for the detoured vehicles could be calculated by averaging the smallest and the largest operating costs of the vehicles detoured. The average operating cost of the detoured vehicles is then given by:

$$U_{DL} = (U_{DP} + U_{DNP})/2 \quad (3)$$

where:

U_{DL} = average operating cost for the detoured vehicles; and
U_{DP} = operating cost for a vehicle weighing the posted bridge capacity (smallest operating cost among the detoured vehicles).

LOAD CAPACITY DETOURS

If a bridge is posted for load capacity, some proportion of the vehicles using the bridge must detour. The vehicles detoured are those that weigh more than the bridge posting. The number of vehicles detoured depends on the posted load capacity of the bridge, and the number and weight distributions of the vehicles encountering the bridge. Different functional classification routes have different patterns of vehicle weight distributions. Thus, the proportion of the vehicles detoured due to the bridge load capacity deficiency would be different for bridges on the different functional classifications. Current bridge policy requires that bridges with load capacities less than 3 tons (27 kN) be closed. Thus, if a bridge is open to the public, its load capacity is 3 tons (27 kN) or greater. Usually, passenger cars, pickup and panel trucks weigh less than 3 tons. Therefore, if a bridge is posted for load capacity, the vehicles detoured would be trucks and similar vehicles that weigh more than 3 tons (27 kN).

From Equation 1, the number of vehicles detoured in a given year for a posted bridge is calculated as follows:

$$N_{DET}(t) = 365ADT(t)C_{LCD}(t) \quad (4)$$

where:

$N_{DET}(t)$ = number of vehicles detoured in a given year for a posted bridge; and
$C_{LCD}(t)$ = coefficient for the proportion of vehicles detoured due to load capacity deficiency in year t.

The total number of trucks detoured includes single unit trucks (or single vehicle trucks, SV) and TTSTs. Thus,

$$C_{LCD}(t) = R_{SV}(t) + R_{TT}(t) \quad (5)$$

where:

$R_{SV}(t)$ = ratio of the number of single-unit trucks heavier than the bridge's single vehicle posting to the total vehicles using the bridge; and
$R_{TT}(t)$ = ratio of the number of trailer combinations heavier than the TTST posting to the total vehicles using the bridge.

Vehicle classification distribution, in terms of vehicle configurations, varies with route functional classification. Literature and data in this area were summarized and new data added from North Carolina and then synthesized (*1*). Since the North Carolina Department of Transportation (NCDOT) posts bridges for load limit considering SV and TTST configurations, the analysis was designed to estimate detours in these two categories. Some sources were categorized by number of axles, others by single-tired, dual-tired and TTST. Some sources separated buses and special vehicles, others did not. In the end, the goal became to define the percentage of major vehicle types on the different roadway functional classifications and to define the typical actual weight distributions of those vehicles. Since cars and light trucks typically weigh less than 3 tons (27 kN), they are not detoured by load posting. Thus, the vehicles of interest are the SV Duals and the TTSTs. Based on the analysis and synthesis of the data available, the values proposed for use as the vehicle

TABLE I VEHICLE DISTRIBUTIONS ON NORTH CAROLINA ROADWAYS BY FUNCTIONAL CLASSIFICATION

Functional Classification	Proportion of Total Vehicles (%)		
	Cars & Light Trucks	SV Duals	TTST
Interstate	83.1	4.4	12.5
Principal Arterial	87.3	6.0	6.6
Minor Arterial	92.1	4.6	3.3
Major Collector	96.3	2.6	1.1
Minor Collector	96.5	2.6	0.8
Local	97.0	2.4	0.6

classification distributions on the different functional classifications of North Carolina bridges are presented in Table I.

The actual truck weight distribution for each type of vehicle classification was needed for determining the number of vehicles detoured for a posted bridge. Weigh-In-Motion data (*8*) for bridges on Interstates, U. S. routes, and State routes were analyzed for this purpose. The truck configurations included 2-axle, 3-axle, and 4-axle single-unit trucks and most semi-trailer combinations. The trucks counted and weighed did not include pickup trucks, recreational vehicles, house trailers, or cars pulling trailers, but included buses. The single-unit trucks recorded in the study were about equivalent to the duals of North Carolina data. The loading distributions by truck type were then multiplied by the corresponding vehicle classification distributions in Table I to determine the percentage of each truck weight range out of the total vehicles encountering the bridge. Instead of showing the percentage for each weight range, Table II shows the cumulative percentage of trucks out of the total vehicles that are heavier than each weight listed. Thus, the values indicate the percentage of ADT detoured by the particular posting level.

On local routes with a low ADT, the detours calculated by this method may not adequately represent the need to provide essential access. If a bridge is posted for less than 16 tons (143 kN), most public service vehicles such as fire trucks, school buses, garbage trucks, heating oil trucks, etc., have to detour (*9*). For each school day, at least six trips may be generated by school buses (two for the elementary school, two for the middle school, and two for the high school). On average, there are about 180 school days in a year. Thus, the average is about three school bus trips every day of the year. For the rest of the public service vehicles, the trips are generated periodically and assumed to average one trip per day. Therefore, if a bridge on the local route is posted for less than 16 tons (143 kN), the number of detours (four per day) generated by the public service vehicles is compared with detours calculated from the results of Table II, and the larger is taken as the number of vehicles detoured by the local route bridge.

BRIDGE LOAD CAPACITY DETERIORATION

Bridge load capacity may deteriorate due to section loss or material degradation. Causes include spalling, cracking, scouring, rotting, infestation or corrosion of reinforcing steel or structural steel, sometimes aggravated by deicing chemicals. Load capacity deterioration is also influenced by the environment of the bridge. Bridges in different weather environments may have different load capacity deterioration rates. Bridges over water or in marine environments may have more severe substructure problems. High volumes of traffic may result in fatigue and overloads may cause damage. Materials and quality of construction are also factors influencing load capacity deterioration. However, such loss rates have not been quantified, and no helpful research results were found in the literature. When a bridge is maintained in good condition, there is virtually no reason to expect load capacity loss with increasing age. However, when deterioration is allowed to start, loss can occur. Experienced engineers note that load capacity decreases with severe deterioration, especially for timber superstructures and substructures. To determine the load capacity deterioration rate, a variety of analysis approaches were tried (*1*). North Carolina posts bridges for load capacity based on the operating rating. Regression analyses of bridge operating rating versus age were conducted using inspection data from

TABLE II PERCENTAGE OF ADT DETOURED BY BRIDGE LOAD POSTING LEVEL, FUNCTIONAL CLASSIFICATION AND VEHICLE TYPE

Bridge Posting (tons)	Interstate		Princ. Art.		Minor Art.		Major Coll.		Minor Coll.		Local	
	SV	TT ST	SV	TT ST	SV	TT ST	SV	TT ST	SV	TT ST	SV	TT ST
3	4.40	12.50	6.00	6.60	4.60	3.30	2.60	1.10	2.60	0.80	2.40	0.60
4	3.87	12.45	5.21	6.57	4.11	3.29	2.32	1.09	2.32	0.80	2.14	0.60
5	3.35	12.40	4.41	6.54	3.61	3.28	2.04	1.09	2.04	0.79	1.88	0.60
6	2.82	12.36	3.62	6.50	3.12	3.26	1.76	1.08	1.76	0.79	1.63	0.59
7	2.30	12.31	2.82	6.47	2.62	3.25	1.48	1.08	1.48	0.78	1.37	0.59
8	1.77	12.26	2.03	6.44	2.13	3.24	1.20	1.07	1.20	0.78	1.11	0.59
9	1.52	12.24	1.70	6.33	1.78	3.19	1.00	1.05	1.00	0.77	0.92	0.58
10	1.26	12.02	1.36	6.23	1.43	3.14	0.80	1.04	0.80	0.76	0.74	0.57
11	1.10	11.65	1.22	5.97	1.28	3.01	0.72	0.99	0.72	0.73	0.67	0.54
12	0.95	11.28	1.08	5.70	1.13	2.87	0.64	0.95	0.64	0.69	0.59	0.52
13	0.82	10.74	0.97	5.39	1.02	2.71	0.57	0.90	0.57	0.66	0.53	0.49
14	0.71	10.04	0.90	5.02	0.94	2.53	0.53	0.84	0.53	0.61	0.49	0.46
15	0.60	9.34	0.82	4.66	0.86	2.35	0.48	0.78	0.48	0.57	0.45	0.42
16	0.51	8.89	0.76	4.41	0.79	2.22	0.45	0.73	0.45	0.54	0.41	0.40
17	0.42	8.35	0.69	4.16	0.73	2.09	0.41	0.69	0.41	0.51	0.38	0.38
18	0.35	8.04	0.63	3.95	0.66	1.99	0.37	0.66	0.37	0.48	0.34	0.36
19	0.30	7.71	0.58	3.78	0.60	1.90	0.34	0.63	0.34	0.46	0.31	0.34
20	0.24	7.37	0.52	3.61	0.55	1.82	0.31	0.60	0.31	0.44	0.28	0.33
21	0.21	7.06	0.44	3.50	0.47	1.76	0.26	0.58	0.26	0.43	0.24	0.32
22	0.18	6.75	0.37	3.39	0.39	1.71	0.22	0.56	0.22	0.41	0.20	0.31
23	0.16	6.46	0.30	3.28	0.32	1.65	0.18	0.55	0.18	0.40	0.17	0.30
24	0.15	6.17	0.25	3.17	0.26	1.60	0.15	0.53	0.15	0.39	0.14	0.29
25	0.13	5.89	0.20	3.06	0.21	1.54	0.12	0.51	0.12	0.37	0.11	0.28
26	0.11	5.61	0.16	2.96	0.17	1.49	0.10	0.49	0.10	0.36	0.09	0.27
27	0.09	5.32	0.13	2.86	0.13	1.44	0.08	0.48	0.08	0.35	0.07	0.26
28	0.08	5.01	0.10	2.75	0.10	1.39	0.06	0.46	0.06	0.33	0.05	0.25
29	0.07	4.68	0.07	2.64	0.08	1.33	0.04	0.44	0.04	0.32	0.04	0.24
30	0.06	4.35	0.05	2.52	0.05	1.27	0.03	0.42	0.03	0.31	0.03	0.23
31	0.05	3.95	0.03	2.38	0.04	1.20	0.02	0.40	0.02	0.29	0.02	0.22
32	0.04	3.56	0.02	2.25	0.02	1.13	0.01	0.37	0.01	0.27	0.01	0.20
33	0.04	3.11	0.01	2.09	0.01	1.05	0.00	0.35	0.00	0.25	0.00	0.19
33.6	0.00	2.81	0.00	1.98	0.00	1.00	0.00	0.33	0.00	0.24	0.00	0.18
34		2.60		1.91		0.96		0.29		0.23		0.16
36		1.74		1.56		0.78		0.24		0.19		0.14
36.6		0.00		0.00		0.00		0.00		0.00		0.00

North Carolina bridges. These analyses excluded the bridges with known reconstruction in past years. The analyses were categorized based on original design loads as an indicator of original capacity. They were also categorized by the material combinations of bridge superstructures and substructures as variables possibly affecting deterioration. The results were found to have poor correlation due to severe scatter and other factors. Nevertheless, the loss rates shown in Table III, compiled with engineering judgement from the regression results and multi-year averaging results, have been used in absence of better information to represent the effect that occurs at low condition states.

When applied, the lowest of the substructure or superstructure condition ratings is assumed to control the deterioration. Analysis of the database shows that

TABLE III ESTIMATED BRIDGE LOAD CAPACITY DETERIORATION RATES

Lower Rating of Superstructure and Substructure	Deterioration Rate (Tons/Year)		
	Timber	Concrete	Steel
6 - 9	0.00	0.00	0.00
5	0.30	0.20	0.20
4	0.60	0.30	0.30
3 or less	1.00	0.50	0.50

1 ton = 8.964 kN

the deck rarely controls the load capacity. The load capacity loss is subtracted from the operating rating; however, the SV posting and TTST posting is similarly reduced only when the resulting operating rating is less than the legal load. Considering the rates of condition deterioration, the values estimated in Table III would result in a capacity loss of approximately 3 tons (27 kN) as the bridge passes through conditions five and four.

VERTICAL CLEARANCE DETOURS

At a bridge with a vertical restriction, some vehicles passing through or under the bridge must detour, i.e., those whose heights are higher than the vertical clearance. The proportion of vehicles detoured depends on the truck height distribution, which may vary with roadway functional classifications (*1*). User costs are generated due to accidents and vehicles that must be detoured at bridges with low vertical clearance. Most trucks on highways are less than 13.5 feet (4.11 m) in height, the legal height for many states. According to Kent and Stevens (*10*), about 0.067 percent of the duals and 0.444 percent of the trailer combinations are more than 13.5 feet (4.11 m) high. If the heights of duals are assumed to be well distributed between 8.0 and 13.5 feet (2.44 and 4.11 m) and if trailer combinations are well distributed between 10 and 13.5 feet (3.05 and 4.11 m), the truck height distributions would correspond to those listed in Table IV, using the vehicle classification distributions in Table I.

The detour length for the vertical clearance deficiency was also assumed to be the appropriate under- or over-route inventory detour length following the same approach used for load capacity detour. Although the operating cost may vary with height, no data were available indicating the variation. The number of bridges with a vertical clearance of less than 13.5 feet (4.11 m) in North Carolina is very small. Thus, it was assumed adequate to use the TTST legal load limit operating cost, U_{DNP}, as a reasonable estimate of the vertical clearance detour unit cost, U_{DC}

ACCIDENT UNIT COSTS

In a study of North Carolina accident data from 1984 to 1989, the annual number of all accidents and the annual number of bridge-related accidents was uniform, averaging 161,922 and 2,710 (1.7 percent) respectively (*6,7*). Although bridge-related accidents represent only 1.7 percent of all traffic accidents, it is important to evaluate these accidents to try to minimize them with appropriate bridge improvements. The severity of bridge-related accidents is usually higher than the severity of other roadway traffic accidents. However, the degree of severity will vary depending on the approach used for measuring the severity. In various published studies, the severity of bridge-related accidents has been estimated to be from 2-to-50 times the severity of general roadway traffic accidents (*6*).

The NCDOT classifies vehicular accidents as fatal, injury, and property-damage-only accidents. An A-B-C injury scale is used to describe the severity level of the injuries, where A is the most severe and C is the least severe. The pattern of bridge-related accident severity is summarized and is compared to other accidents in Table V. The average number of people killed in bridge-related accidents in North Carolina was determined to be 0.019 persons/accident. However, the average number of people killed for other traffic accidents was 0.009 persons/accident. Taking this as a measure of accident severity, it implies that bridge-related accidents are roughly twice as severe as general roadway traffic accidents. The ratio comparing the severity of bridge-related accidents to other traffic accidents decreased as the injury severity decreased. However, for all injury types except C, bridge-related

TABLE IV PERCENTAGE OF ADT DETOURED BY BRIDGE VERTICAL CLEARANCE POSTING LEVEL, FUNCTIONAL CLASSIFICATION AND VEHICLE TYPE

Vertical Clearance (feet)	Interstate		Princ. Art.		Minor Art.		Major Coll.		Minor Coll.		Local	
	SV	TT ST	SV	TT ST	SV	TT ST	SV	TT ST	SV	TT ST	SV	TT ST
8.0	4.40	12.50	6.00	6.60	4.60	3.30	2.60	1.10	2.60	0.80	2.40	0.60
8.5	4.00	12.50	5.45	6.60	4.18	3.30	2.36	1.10	2.36	0.80	2.18	0.60
9.0	3.60	12.50	4.91	6.60	3.76	3.30	2.13	1.10	2.13	0.80	1.96	0.60
9.5	3.20	12.50	4.36	6.60	3.35	3.30	1.89	1.10	1.89	0.80	1.75	0.60
10.0	2.80	12.50	3.82	6.60	2.93	3.30	1.66	1.10	1.66	0.80	1.53	0.60
10.5	2.40	10.72	3.27	5.66	2.51	2.83	1.42	0.94	1.42	0.69	1.31	0.51
11.0	2.00	8.94	2.73	4.72	2.09	2.36	1.18	0.79	1.18	0.57	1.09	0.43
11.5	1.60	7.17	2.18	3.78	1.67	1.89	0.95	0.63	0.95	0.46	0.87	0.34
12.0	1.20	5.39	1.64	2.85	1.26	1.42	0.71	0.47	0.71	0.34	0.66	0.26
12.5	0.80	3.61	1.09	1.91	0.84	0.95	0.47	0.32	0.47	0.23	0.44	0.17
13.0	0.40	1.83	0.55	0.97	0.42	0.48	0.24	0.16	0.24	0.12	0.22	0.09
13.5	0.00	0.06	0.00	0.03	0.00	0.01	0.00	0.00	0.00	0.00	0.00	0.00
14.0	0.00	0.01	0.00	0.01	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
14.5	0.00	0.01	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00

1 ft = 0.3048 m

TABLE V AVERAGE NUMBER OF INJURIES BY TYPE (1984-1989)

Severity of Accident	Total Average Bridge-Related Accident Injuries per Year	Average Number of Injuries per Accident		
		Bridge-Related Accidents	Other Roadway Traffic Accidents	Ratio of Bridge-Related to Other Roadway Accidents
Fatal	52	0.02	0.01	2.00
Injury A	352	0.13	0.10	1.30
Injury B	555	0.20	0.19	1.05
Injury C	910	0.34	0.38	0.87

injuries were more severe than other traffic accident injuries. Furthermore, the number of injuries per accident was greater for bridge-related accidents.

Chen and Johnston (*1*) used similar data on relative severity of non-bridge to bridge-related accidents to determine the average cost of a bridge-related accident. In 1985 dollars, the estimated cost was $14,710 based upon a Human Capital Approach and $31,919 based upon a Willingness-to-Pay Approach. These amounts need annual updating due to inflation and changing relative costs in the economy. One method is to use the bridge-related accident injury data (Table V), which can be updated periodically by analysis of NCDOT accident data, and to combine it with injury costs published periodically by available sources. Injury costs based upon a Human Capital Approach are published annually (*Accident Facts*) by the National Safety Council (NSC). Injury costs based upon the Willingness-to-Pay Approach are published about every three to four years by the Federal Highway Administration (*11*). Table VI shows

TABLE VI BRIDGE-RELATED ACCIDENT AVERAGE COST

Injury Severity	Average Number of Injuries per Accident	Human Capital Approach (1990 Dollars)		Willingness-to-Pay Approach (1988 Dollars)	
		Average Cost per Injury, $	Cost per Bridge-Related Accident, $	Average Cost per Injury, $	Cost per Bridge-Related Accident, $
Fatal	0.02	410,000	8,200	1,500,000	30,000
Injury A	0.13	38,200	5,000	39,000	5,100
Injury B	0.20	8,900	1,800	12,000	2,400
Injury C	0.34	2,900	900	6,000	2,000
Property Damage			3,900		3,900
Total			19,800		43,400

the 1990 injury costs from NSC, the 1988 injury costs from FHWA, and the average property damage reported in bridge related accidents in 1990 in North Carolina. When extended, this results in an average bridge-related accident cost, U_{AC}, of $19,800 (1990 dollars) based on the Human Capital Approach and a cost, U_{AC}, of $43,400 (1988 dollars) based upon the Willingness-to-Pay Approach.

ACCIDENTS DUE TO DECK AND APPROACH ROADWAY GEOMETRY

One of the difficulties in developing prediction models for bridge related accidents has been that the accident data files cannot currently be linked directly to the bridge inventory file. This may be accomplished in the future either by complete mileposting or by GIS technique; however, for the present, alternate approaches were necessary. In one effort (*1*), average annual statewide bridge-related accidents were determined with accident data files. The accidents also could be tabulated by functional classification. However, since the individual accidents could not be linked to particular bridges, only an empirical approach could be used in developing a prediction equation. Assuming the accidents were primarily due to deck width and approach roadway alignment deficiencies, a trial-and-error approach was used evolve an equation that would predict about the same total accidents statewide and by functional classification. For the comparison, resulting accidents for each bridge were calculated and summed by the respective classifications. The resulting equation was:

$$ACCR_{CDW,ALI} = 6.28 x 10^{7.5} CDW^{-6.5}[1 + 0.5(9 - ALI)/7] \quad (6)$$

and

$$C_{WDA} + C_{ALA} = ACCR_{CDW,ALI} \ x \ 10^{-6} \quad (7)$$

where:

$ACCR_{CDW,ALI}$ = Accident rate of bridge, accidents per million vehicles;
CDW = Clear deck width, feet (m/3.28); and
ALI = Alignment appraisal rating.

In a more recent effort (*6,7*), the accidents from 1983 to 1989 in five of the North Carolina's 100 counties were studied. Over 2,000 accident records indicating a bridge as a feature on the over-route were manually matched to the actual bridge. Various forms of regression analysis were conducted considering a variety of bridge data file parameters, such as deck width, alignment, ADT, bridge length, functional classification, etc. From this process, the annual number of accidents on a bridge was estimated to be

$$NOACC = 0.783(ADT^{0.073})(LENGTH^{0.033})(WDIFACC + 1)^{0.05} - 1.33 \quad (8)$$

where:

ADT = Average daily traffic;
LENGTH = Bridge length, feet (m/3.28);
NOACC = Number of accidents per year; and

WDIFACC = Width difference between the goal clear deck width for an acceptable level of service and the actual bridge clear deck width, but not less than zero, feet (m/3.28).

Although developed from only five counties, Equation 8, when applied statewide, predicts the current average number of bridge-related accidents happening in North Carolina per year. The investigators noted with some surprise that the analysis did not find alignment significant for the accident data set studied. This may be because poor alignment is generally not associated with high ADT routes. When using this equation, the number of accidents for low ADT approaches zero. However, negative values for the number of accidents may be generated for very low values of the independent variables, particularly ADT. For example, the number of accidents at an ADT of less than 200 vehicles per day would be expected to be very low, when considering only bridge related factors. It is therefore reasonable to assume that at such low variable combinations the number of accidents would be zero. It is also important to interpret the results in combination with the width and lane goals that are simultaneously increasing with ADT (*4,9*). To date, Equation 6 has been the basis for predictions in OPBRIDGE; however, Equation 8 is being implemented simultaneous with other updates and improvements.

ACCIDENTS DUE TO VERTICAL CLEARANCE

Although low vertical clearance has been recognized as one of the contributing factors to accidents on bridges, neither definitive data nor studies relating accidents to vertical clearance deficiency could be found in the literature. Thus, data were obtained from NCDOT traffic accident records and analyzed. Vehicle accidents associated with underpasses of bridges in North Carolina consistently averaged approximately 440 per year. The data available for accidents involving bridge underclearance divided the accidents by roadway functional classification, but it did not show the actual bridge or clearance involved. Therefore, a direct analysis by regression or other means was not possible. Therefore, an empirical relationship was assumed, fitted and then tested to see if it could predict the accident trends. For analysis, the accidents were assumed to have occurred because of underclearance deficiency. Although some accidents may have involved under-route width problems, these could not be separated. Most of the underpass accidents reported to the NCDOT Bridge Maintenance Unit appear to involve vertical clearance. The accident rate was assumed to be linearly increasing with the amount of the vertical deficiency in relation to the desirable level of service goals (*9*) and the under-route ADT. Distributing the number of accidents to the bridges having vertical clearance deficiency in proportion to the deficiency, the accident rates for various functional classifications were calculated. From this approach, the accident rate generated due to a bridge vertical clearance deficiency, C_{CLA}, for a bridge can be estimated (*1*) as follows:

$$C_{CLA} = \frac{UG - UCL}{ACCRU} \tag{9}$$

where:

C_{CLA} = coefficient for proportion of vehicles incurring accidents due to a vertical clearance deficiency;
UG = underclearance desirable goal, feet (m);
UCL = bridge underclearance height, feet (m);
ACCRU = accident rate by functional classification due to vertical clearance deficiency;
= 7.4×10^6 veh./acc./ft. deficiency (2.25×10^6 veh./acc./m deficiency) for Interstates;
= 37.3×10^6 veh./acc./ft. deficiency (11.4×10^6 veh./acc./m deficiency) for Arterials;
= 8.0×10^6 veh./acc./ft. deficiency (2.44×10^6 veh./acc./m deficiency) for Collectors; and
= 1.1×10^6 veh./acc./ft. deficiency (0.34×10^6 veh./acc./m deficiency) for Locals.

Due to the insufficient data on the costs for vertical clearance accidents, the average vehicular bridge-related accident cost, U_{AC}, presented previously has been used as the average cost for bridge underpass accidents.

TRAFFIC GROWTH

Due to many factors, such as population growth, economic prosperity, the traffic volumes using most roadways increase year by year. Although there have been occasional drops, the national vehicle-miles have increased at an annual rate of about 3.7 percent while the population increase rate has averaged about 1.2 percent. The growth occurs partly on existing roadways and partly on newly added roadways. Different functional classification highways have different service purposes. The Interstate highways provide interstate traffic services. The arterial systems provide traffic services between major points within a state. The collector systems provide services for intracounty traffic. And the local systems usually provide the essential access to residences, farms, and other abutting properties. Since growth factors may affect these

TABLE VII EXAMPLE ANNUAL TRAFFIC GROWTH RATES FOR NORTH CAROLINA DIVISION 12 COUNTIES

NC Division 12 Counties (6 of 100 NC counties)	ADT Increase Rates (Percent per Year)			
	Local	Collector	Arterial	Interstate
Alexander	1.29	1.62		
Catawba	0.92	1.43		
Cleveland	0.30	1.12	1.94	4.06
Gaston	1.62	1.78	(Divisionwide)	(Statewide)
Iredell	0.74	1.34		
Lincoln	1.13	1.53		

systems in different ways, the ADT increase rates of different functional classification routes may be different.

In North Carolina, there were 59 automatic traffic recording (ATR) stations in operation for continuous traffic counting at the time of the study. The locations of these ATR stations were spread over the state and distributed on most of the highway functional classifications. Of the 59 ATR stations, 20 were on arterial systems, including principal and minor arterials; 18 were on collector systems, including major and minor collectors; and seven were on the Interstate System. The remaining 14 stations were in urban areas. The ADT data collected at each ATR station over 10 years were used to analyze the ADT increase rates of North Carolina highways. Based on this data, the ADT increase rates of different functional classification highways were predicted.

The average yearly ADT increase rate of the 7 Interstate ATR stations was considered as the ADT increase rate of the North Carolina Interstate System. Although in some urban areas the interstate highways might serve as an expressway, the Interstate highways are mainly for long distance trips. Thus, a single number as the ADT increase rate was used for the entire Interstate System in a state. The yearly ADT increase rate of the 7 Interstate ATR stations was about 4.06 percent. Unlike the Interstate System, the arterial system connects several important towns located in several adjacent counties. The ADT increase rates of the arterial system would be influenced by regional factors. The state's 100 counties are divided into 14 highway divisions. Thus, the ADT increase rates of the arterial systems were predicted on a division basis. The arterial ADT increase rate of a particular division was found by calculating the average ADT increase rate of the ATR arterial stations in the division.

Due to the low volume of traffic, no ATR stations were located on local routes. Thus, the local route ADT increase rate was estimated on a different basis. Most of the traffic is locally initiated and is closely related to the local population. If the population of a local area increases, the number of local activities also would increase. Thus, the yearly ADT increase rate of the local route was assumed to be equal to the population growth of the local area or county. The North Carolina Office of State Budget and Management makes yearly estimates of the 20-year population growth in each county of the state. The county population growth rates were assumed as the local route traffic growth rate, except that a few negative growth rates were adjusted to zero.

The traffic volume of collectors in a region is between that of the arterial and the local systems. Similarly, the ADT increase rate of collectors might be between the increase rates of these two systems. Because of the nature of its traffic, the collector ADT increase rate also would be appropriately predicted by county. However, the data from the 18 collector ATR stations were not sufficient for predicting the ADT increase rates of the collector systems for each of the 100 North Carolina counties. Thus, the average ADT increase rates of the local system and the arterial system of each county were used as the ADT increase rate of its collectors. The resulting statewide collector ADT increase rate was about 1.92 percent percent compared to 2.03 from the 18 collector ATR stations. Table VII shows the ADT increase rate, calculated by these methods, for the roadway classifications in six of 100 NC counties constituting Highway Division 12. Similar data were developed for the other 94 counties (*1*).

SUMMARY

This paper has provided a summary of the efforts to estimate bridge generated user costs for North Carolina bridges. The methods developed were based on varying degrees of available data. Some parameters, such as accidents due to lateral underclearance, could not be defined due to a lack of data. Other parameters were defined for this stage of the state-of-the-art. With the newly expanding interest in economic assessment, it is hoped that more national and state efforts will be made in the future to collect data and allow improvement of such prediction methodologies. Nevertheless, the approaches and parameters developed should provide a starting point to those desiring to estimate bridge-related user costs.

Some have wondered if economic assessment of bridge improvement alternatives can be made sufficiently accurate. However, it is important to remember that although we engineers can calculate stresses to many insignificant figures, we only know the real loads, and thus the stresses, to one or sometimes two significant figures. Achieving the same level of accuracy in estimating costs may still be a goal, but it is probably attainable.

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