Delay Estimation and Optimal Length for Four-Lane Divided Freeway Workzones

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(Reviewed by the Urban Transportation Division)

Abstract: Vehicle delay is one of the most serious impacts of highway workzones on existing traffic operations. The present study provides a comprehensive and detailed estimation of traffic delay due to a freeway workzone where the roadway in one direction is closed and the traffic usually on the closed roadway is diverted to share the roadway in the opposite direction. The total delay was decomposed into speed reduction delay and congestion delay. Different procedures were developed to estimate speed reduction delay for several roadway conditions. A mathematical model was developed to find the length of the queue upstream of a workzone and a procedure was generated to estimate daily congestion delay under any given condition. Alternative roadway closure patterns along the length of a given project were evaluated in terms of traffic control cost and additional road user costs. The optimal workzone length for a given project was studied. Procedures were developed to determine the optimal workzone length.

INTRODUCTION

Today, reconstruction and rehabilitation workzones can be found almost on numerous interstates and freeways. While workzones provide a means to perform reconstruction and rehabilitation without completely shutting down traffic operations, they have significant impacts. These impacts include reduced freeway capacity, increased accident rates, increased fuel consumption and emissions, and additional congestion and delay. Delay is one of the most significant problems associated with a workzone. In some cases, highway traffic operations can completely fail due to congestion caused by workzones, especially in the peak time periods.

Various efforts have been made to mitigate the impacts of workzones (Janson et al. 1987). One such effort involves optimizing workzone traffic control design and practices, including optimal control device design, optimal lane-closure configuration, and optimal workzone length. For four-lane divided rural freeways, there are two basic lane closure configurations (Dudek et al. 1986): 1) Closing one lane in one direction (single-lane closure), resulting in no impact on the traffic in the opposite direction; and 2) completely closing the roadway in one direction and the traffic in that direction is diverted to the roadway in the opposite direction through median crossovers so that the roadway in the opposite direction becomes a two-lane two-way operation.

Although single-lane closures have no impact on the traffic operation in the opposite direction, such arrangements increase construction duration and construction costs of a given reconstruction project. Traffic accident rates could also increase because of the conflict between traffic operations and construction activities. Therefore, single-lane closures are typically preferred only for small projects such as maintenance work. For large reconstruction projects, completely closing the roadway in one direction tends to significantly reduce the required construction duration and cost (Dudek et al. 1986), which, in turn, would reduce overall traffic control costs and road-user costs.

Completely closing a roadway on a four-lane divided freeway segment changes the segment effectively into a two-lane two-way no-passing operation. On the average, the capacity of the segment is reduced to about one-third of its normal capacity (Dudek et al. 1986). As a result, the level of service (LOS) drops. Significant delay is induced because of the reduced average traffic speed on the workzone segment and upstream congestion caused by the reduced capacity in the workzone. The capacity and average traffic speed in a workzone are significantly affected by heavy vehicles since heavy vehicles are likely to move slowly and inhibit the traffic stream on the workzone segments, especially under rolling and mountainous terrain.

Changing the arrangement of workzones along the length of a given project (longitudinal roadway closure pattern), such as changing the number of workzones at the same time or changing the length of a workzone, can mitigate or intensify the effect of heavy vehicles on the capacity and average traffic speed in work areas, and in turn, can reduce or increase traffic delay. The objectives of this study are to evaluate alternative longitudinal roadway closure patterns and determine the optimal workzone length in terms of delay, traffic control, and safety criteria under different roadway and traffic conditions for a given four-lane divided freeway reconstruction project with complete closure of the roadway in one direction.

TRAFFIC DELAY DUE TO WORKZONES

There are two sources of workzone delay: speed-reduction delay and congestion delay.

Speed-Reduction Delay

For a four-lane divided freeway workzone where the roadway in one direction is closed for reconstruction and the roadway in the opposite direction is changed to a two-lane two-way no-passing operation (Fig. 1), the delays due to speed

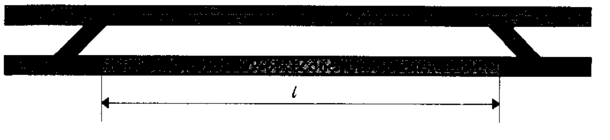

FIG. 1. Typical Four-Lane Freeway Workzone

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reduction can be determined by the difference between average moving speed with two-lane two-way operations and average moving speed with normal four-lane freeway operations.

With normal four-lane freeway operations, the actual capacity, c, of the freeway (Transportation 1985) is

$$c = c_j N f_w f_p f_{HV} \quad (1)$$

where c_j = capacity under ideal conditions with design speed of j; N = number of lanes in one direction; f_w = lane width and lateral clearance factor; f_p = driver population factor; f_{HV} = heavy vehicle factor, where $f_{HV} = [1 + P_T(E_T - 1) + P_B(E_B - 1) + P_R(E_R - 1)]^{-1}$; E_T, E_B, and E_R = passenger car equivalences of trucks, buses, and recreational vehicles, respectively; and P_T, P_B, and P_R = proportion of trucks, buses, and recreational vehicles in traffic, respectively. Since bus and recreational vehicles are only a small proportion of heavy traffic, and their effect on capacity is approximately the same as trucks, it is assumed that $f_{HV} = [1 + P_{HV}(E_{HV} - 1)]^{-1}$.

The service flow rate (SF) of a normal four-lane divided freeway can be determined by (2) (Transportation 1985)

$$SF_i = 2{,}000 \cdot N \cdot f_w \cdot f_p \cdot f_{hv} \cdot (v/c)_i \quad (2)$$

where $(v/c)_i$ = volume-capacity ratio (VCR) at LOS i.

If the directional design hourly flow rate (DDHV), and peak hour factor (PHF) are given, the demand service flow rate (DSF) is DSF = DDHV/PHF. If we let SF = DSF, then the VCR (v/c) can be expressed by (3)

$$V/C = DDHV/PHF \cdot 2{,}000 \cdot N \cdot f_w \cdot f_p \cdot f_{hv} \quad (3)$$

With two-lane two-way no-passing operations, the actual capacity for general two-lane two-way segments is given by (4) (Transportation 1985)

$$c = 2{,}800 f_d f_w f_{HV} \quad (4)$$

where f_d = traffic direction distribution factor. For specific upgrade segments, the actual capacity is given in (5) (Transportation 1985)

$$c = 2{,}800 f_d f_w f_g f_{HV} \quad (5)$$

where $f_g = (1 + P_p I_p)$; and P_p = proportion of passenger cars, with $I_p = 0.02(E - E_o)$. E, E_o = PCEs at any given steepness of grade and at 0% grade, respectively, $f_{HV} = [1 + P_{HV}(1 + E_{HV} - 1)]^{-1}$, where $E_{HV} = 1 + (0.25 + P_T/P_{HV})(E - 1)$ = heavy traffic factor.

The service flow rate for general two-lane two-way segments is as follows (Transportation 1985):

$$SF_i = 2{,}800 \cdot f_w \cdot f_d \cdot f_{hv} \cdot (v/c)_i \quad (6)$$

If the total two-way design hourly volume (DHV) and peak hour factor are known, the demand flow rate (DSF) for a two-lane two-way segment is DHV/PHF. The PHF should be the same as in freeway segments because traffic arrival patterns within an hour do not change with respect to closing the roadway.

If we let SF = DSF, then the VCR for a general two-lane two-way segment can be found as shown in (7)

$$(v/c) = DHV/PHF \cdot 2{,}800 \cdot f_w \cdot f_d \cdot f_{hv} \quad (7)$$

using (3) and (7), and the tables given in the Highway Capacity Manual (HCM), (Transportation 1985) the LOS and average traffic speeds can be found for both normal four-lane freeway operations and two-lane two-way operations.

Tables 1, 2, and 3 present the reduction in LOS and average traffic speed due to a workzone over a range of traffic volumes and truck percents under different types of terrain. With normal four-lane divided freeway operations, when VCR is less than 0.35 (LOS = A), traffic speed depends on driver preference, average traffic speed is 60 mph, and the speed will not increase with further decreases in VCR. With two-lane two-way no-passing operations, when the VCR is greater than 1.0, the average traveling speed reduces to a crawl within the workzone segment (approximately 44 mph for level terrain, 40 mph for rolling terrain, and 30 mph for mountainous terrain). The average moving speed will not be reduced with further increases in VCR since only the volume equal to workzone

TABLE 1. Operational Analysis of Workzones under Level Terrain

Two-way traffic volume (vph) (1)	Truck per-cent (%) (2)	Four-Lane Divided Freeway (v/c) (3)	LOS (4)	Speed (mph) (5)	Two-Lane Two-Way Highway (v/c) (6)	LOS (7)	Speed (mph) (8)	Speed reduction (mph) (9)
1,000	5	0.136	A	60.0	0.398	D	51.5	8.5
1,000	15	0.145	A	60.0	0.444	D	50.9	9.1
1,000	25	0.155	A	60.0	0.489	D	50.3	9.7
2,000	5	0.272	A	60.0	0.797	E	46.8	13.2
2,000	15	0.291	A	60.0	0.887	E	46.0	14.0
2,000	25	0.309	A	60.0	0.977	E	45.3	14.7
3,000	5	0.409	B	59.0	1.195	F	44.0	15.0
3,000	15	0.436	B	58.6	1.330	F	44.0	14.6
3,000	25	0.464	B	58.2	1.466	F	44.0	14.2
4,000	5	0.545	C	57.0	1.594	F	44.0	13.0
4,000	15	0.582	C	56.5	1.774	F	44.0	12.5
4,000	25	0.618	C	56.0	1.955	F	44.0	12.0

TABLE 2. Operational Analysis of Workzones under Rolling Terrain

Two-way traffic volume (vph) (1)	Truck per-cent (%) (2)	Four-Lane Divided Freeway (v/c) (3)	LOS (4)	Speed (mph) (5)	Two-Lane Two-Way Highway (v/c) (6)	LOS (7)	Speed (mph) (8)	Speed reduction (mph) (9)
1,000	5	0.151	A	60.0	0.451	D	50.8	9.2
1,000	15	0.191	A	60.0	0.601	E	48.9	11.1
1,000	25	0.230	A	60.0	0.752	E	47.2	12.8
2,000	5	0.303	A	60.0	0.902	E	45.9	14.1
2,000	15	0.382	B	59.5	1.203	F	40.0	19.5
2,000	25	0.461	B	58.2	1.503	F	40.0	18.2
3,000	5	0.454	B	58.4	1.353	F	40.0	18.4
3,000	15	0.572	C	56.6	1.805	F	40.0	16.6
3,000	25	0.691	C	55.0	2.256	F	40.0	15.0
4,000	5	0.605	C	56.2	1.805	F	40.0	16.2
4,000	15	0.763	C	54.1	2.406	F	40.0	14.1
4,000	25	0.921	E	46.5	3.008	F	40.0	6.5

TABLE 3. Operational Analysis of Workzones under Mountainous Terrain

Two-way traffic volume (vph) (1)	Truck per-cent (%) (2)	Four-Lane Divided Freeway (v/c) (3)	LOS (4)	Speed (mph) (5)	Two-Lane Two-Way Highway (v/c) (6)	LOS (7)	Speed (mph) (8)	Speed reduction (mph) (9)
1,000	5	0.178	A	60.0	0.583	E	49.1	11.9
1,000	15	0.270	A	60.0	0.996	E	45.2	14.8
1,000	25	0.362	B	59.8	1.409	F	30.0	29.8
2,000	5	0.355	B	59.9	1.165	F	30.0	29.9
2,000	15	0.539	B	57.0	1.992	F	30.0	27.0
2,000	25	0.724	C	54.6	2.820	F	30.0	24.6
3,000	5	0.533	B	57.1	1.748	F	30.0	27.1
3,000	15	0.809	D	52.0	2.988	F	30.0	22.0
3,000	25	1.086	F	30.0	4.229	F	30.0	0.0
4,000	5	0.711	C	54.8	2.331	F	30.0	0.0
4,000	15	1.079	F	30.0	3.985	F	30.0	0.0
4,000	25	1.447	F	30.0	5.639	F	30.0	0.0

capacity is served and the excess volume is transferred to the next hour.

The significant drop in LOS after closing one roadway is due to the smaller capacity of the workzone. The total two-way capacity of the workzone is only about one-third of the original capacity. The speed reduction after closing one roadway depends on type of terrain, truck percent, and hourly traffic volume. Under low traffic volume and level terrain, VCR is less than 1.0 both before and after closing the roadway and VCR increases with volume far faster with two-lane two-way operations than with four-lane freeway operations. Therefore, speed reduction increases with hourly traffic volume. Under a higher traffic volume and mountainous terrain, the VCR with two-lane two-way operations is greater than 1.0 and the speed remains constant with further increases in VCR while the average speed with four-lane freeway operations continues to decrease with increases in VCR. Thus, speed reduction decreases with increases in hourly traffic volume. The effect of truck percent on the drop in LOS and speed reduction varies with the type of terrain. Under level terrain, when truck percent was increased from 5% to 25%, the drop in LOS remained the same with all given volume levels. This is because trucks have little impact on traffic streams under level terrain. However, under rolling or mountain terrain, trucks tend to move slowly and their effects become significant.

For a vehicle traveling through a workzone, the delay due to speed reduction can be expressed as $d_s = (1/s_l - 1/s_0)\cdot l$ where d_s = speed reduction related delay per vehicle due to a workzone (h); s_l = hourly average traffic speed with two-lane two-way operation (km); s_0 = hourly average traffic speed with normal freeway operation (km); and l = length of workzone (km).

Hourly speed-reduction delay is the product of hourly traffic volume and delay per vehicle $d_{hs} = \text{DHV}\cdot d_s$ where d_{hs} = hourly speed reduction related delay (veh-h).

Since traffic volume is usually given in average daily traffic (ADT) rather than in average hourly volume (DHV), a given ADT has to be converted into equivalent hourly volumes according to a given or a typical traffic-flow pattern. An hourly delay is estimated for each hour of a day. Daily speed-reduction delay is calculated by cumulating hourly delay over the 24 hours of a day.

Since the procedure to determine average moving speed for general highway segments is different from that for specific upgrade highway segments, estimation of daily speed reduction delay should be separated into two cases: 1) for general segments; and 2) for specific upgrade segments.

Procedure 1 shows the steps to calculate daily speed-reduction delay per unit length of work zone for general segments.

Procedure 1

1. Step 1. Input ADT, truck percent, type of terrain, and roadway conditions.
2. Step 2. Convert ADT into hourly volumes.
3. Step 3. Find PCEs for trucks according to type of terrain. PCEs with normal freeway operations are different from that with two-lane two-way operations. The specific data are given as follows:
   - Level terrain: E_T = 1.7 for freeway, and 2.2 for two-way operation
   - Rolling terrain: E_T = 4.0 for freeway, and 5.0 for two-way operation
   - Mountainous terrain: E_T = 8.0 for freeway, and 12.0 for two-way operation
4. Step 4. Find capacity adjustment factors and actual capacity. Use (1) with normal freeway operations and (4) with two-lane two-way operations.
5. Step 5. Find hourly average speeds for each hour with normal freeway operations and with two-lane two-way operations. This includes the following:
   - Determine VCR with normal freeway operations.
   - Find average speed with normal four-lane freeway operations. Using HCM tabulated values (Transportation 1985) and performing linear interpolation, the following equations can be used.

     When $v/c > 1.0$, speed = 30 mph.
     When $1.0 >= v/c > 0.93$, speed $= 46 - 229\cdot(v/c - 0.93)$ mph.
     When $0.93 >= v/c > 0.77$, speed $= 54 - 50\cdot(v/c - 0.77)$ mph.
     When $0.77 >= v/c > 0.54$, speed $= 57 - 13\cdot(v/c - 0.54)$ mph.
     When $0.54 >= v/c > 0.35$, speed $= 60 - 15.8\cdot(v/c - 0.35)$ mph.
     When $v/c <= 0.35$, speed = 60 mph.

   - Determine the VCR for two-lane two-way segments.
   - Find average moving speed with two-lane two-way operations. Through regression analysis of HCM tabulated values (Transportation 1985), average moving speed can be determined as follows:

     If $v/c < 1.0$, speed $= 58.3 - 15.9\cdot(v/c) + 6.36\cdot(v/c)^2$ mph, where $R^2 = 0.99$.
     If $v/c > 1.0$, speed = 44 mph for level, 40 mph for rolling, and 30 mph for mountainous terrain.
6. Step 6. Find hourly delay for each hour with the average hourly speed with normal freeway operations and that with two-lane two-way operations.
7. Step 7. Integrate hourly delays (d_{hs}) into daily delay (d_{ds}).

Fig. 2 shows the relationship between daily speed-reduction delay and traffic volume under different types of terrain. Under level terrain, daily speed reduction delay increases linearly with ADT. Truck percent does not have a significant effect on the delay.

Under rolling terrain, daily speed reduction delay is more significant than under level terrain. Truck percent has a significant effect on the relationship between the delay and ADT. When truck percent is less than 15%, the delay increases with ADT and the relationship is nearly linear. When truck percent is 25%, the delay increases with ADT only if ADT is less than 50,000.

Mountainous is the terrain where the delays are most sig-

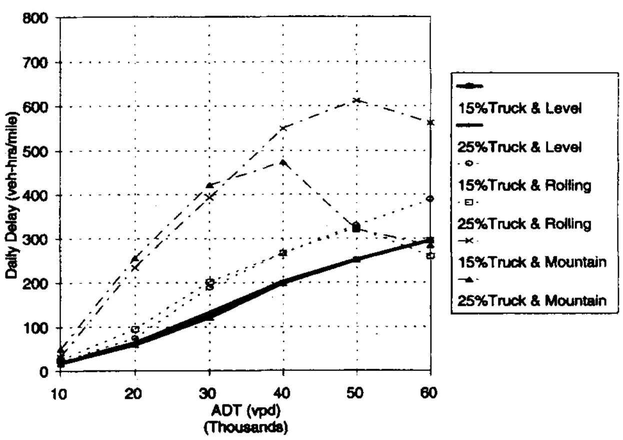

FIG. 2. Relationship between Daily Speed Reduction Delay and ADT

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nificant. When the truck percent is 25%, the delay does not significantly increase with ADT if ADT > 30,000. It actually decreases with increase in ADT if ADT is greater than 40,000 vehicles per day (vpd). This is because, under mountainous terrain and high truck percent, the freeway is operated near or exceeding its capacity and there is no significant difference between the average speed with normal freeway operations and that with two-lane two-way operations.

The steps to calculate speed reduction related delay for specific upgrade segments is similar to that for general segments. However, it is a more complex procedure to estimate average speed with two-lane two-way operations. This is because the average speed depends on PCEs, while, at the same time, PCEs vary with average speed. In order to choose PCEs, an iterative procedure is required. First, assume an average speed and find PCEs under the assumed speed. Using the chosen PCEs, an average speed is then calculated. The calculated speed is always different from the assumed one. Different trials must be carried out until the calculated speed is close enough to the assumed one. This process is further complicated by the effect of the steepness and length of grade on PCEs.

Procedure 2 shows how to determine the average moving speed with two-lane two-way operations for specific upgrade segments.

Procedure 2

1. For a given steepness and length of grade, select PCEs under the highest possible upgrade speed tabulated in HCM.
2. Calculate f_{HV} and actual work zone capacity with the selected PCEs using (5).
3. Calculate VCR and traffic speed with the following relationship obtained by regression analysis of tabulated values (Transportation 1985) ($R^2 > 0.99$ in all cases):
   - At 3% grade: speed = $57 - 15.8 \cdot (v/c)$ mph
   - At 4% grade: speed = $56.8 - 15.8 \cdot (v/c)$ mph
   - At 5% grade: speed = $50.9 - 19.2 \cdot (v/c)$ mph
   - At 6% grade: speed = $54.2 - 6.6 \cdot (v/c) - 17 \cdot (v/c)^2$ mph
4. Compare the calculated speed with the assumed speed. If the calculated speed is greater than the assumed speed, use the calculated speed, stop. Otherwise, select PCEs under the next highest given speed and go through step 2, step 3, and step 4. Repeat the process until the condition to stop is satisfied.

When PCEs under the lowest tabulated speed is selected and calculated speed is less than the lowest tabulated speed, set the actual speed equal to the lowest speed, that is, 40 mph for 3–4% grade, 35 mph for 5% grade, and 30 mph for 6% grade.

Procedure 3 shows the steps for estimating speed-reduction delay for specific upgrade segments given as follows:

Procedure 3

1. Step 1. Input ADT, truck percent, steepness and length of grade, and roadway conditions.
2. Step 2. Convert ADT into hourly volumes over the 24 hours.
3. Step 3. Find PCEs for normal freeway operation according to steepness of grade and truck percent (PT). The specific data are given as follows:
   - When grade = 3%, $E_T = 7.0$ if PT < = 5% and $E_T = 5.0$ if PT > 5%
   - When grade = 4%, $E_T = 9.0$ if PT < = 5% and $E_T = 7.0$ if PT > 5%
   - When grade = 5%, $E_T = 11.0$ if PT < = 5% and $E_T = 8.0$ if PT > 5%
   - When grade = 6%, $E_T = 12.0$ if PT < = 5% and $E_T = 9.0$ if PT > 5%
4. Step 4. Find capacity adjustment factors and actual capacity for normal freeway operations using (1).
5. Step 5. Find VCR, average hourly speed for each hour with normal freeway operations (same as step 4 in procedure 1).
6. Step 6. Find average hourly speed for each hour with two-lane two-way operations using procedure 2.
7. Step 7. Find hourly delay for each hour with the average hourly speed with normal freeway operations and that with two-lane two-way operations.
8. Step 8. Integrate hourly delay into daily delay.

Congestion Related Delay

When traffic volume is less than workzone capacity, there is no congestion if the arrivals are uniformly distributed within a given hour. However, if the arrivals are not uniformly distributed, congestion could occur. The distribution of arrivals depends on traffic conditions upstream of a workzone. Studies indicate that under a low directional flow rate (e.g., 1,800 vph on four-lane divided freeways), there is little interaction between vehicles, the arrivals are random, and a Poisson-based model can be used (May 1990). According to the HCM (Transportation 1985), the directional capacity of a workzone is about 1,400 vph, which is less than 1,800 vph. Therefore, the arrivals can be assumed to be random if traffic volume is less than workzone capacity.

If congestion exists, the traffic waiting to enter a workzone is similar to waiting in a service queue. For a low flow rate (e.g., 500 vph), the distribution of time headway can be approximated by a negative exponential distribution. The average time headway, about 3.9 s (May 1990), can be considered as average service time which is the average time required to empty the first position in the workzone segment for the following car to enter. The average queue length over the hour was found to be 0.4 vehicles using the following equation (Hillier and Lieberman 1991), $q = x^2/[u(u - x)]$ where q = length of queue (veh); x = average number of vehicle arrivals per unit of time (veh/s); and u = number of vehicles serviced per unit time (veh/s).

For a higher flow rate (e.g., 1,200 vph), a minimum time headway (2.0 s usually), is likely to be maintained within the work zone. The average queue length over the hour is estimated to be 0.67 vehicles using the following equation (Hillier and Lieberman 1991): $q = x^2/[2u(u - x)]$. This queue analysis indicates that there is little congestion when the demand is less than workzone capacity.

Congestion tends to occur during certain time periods of the day when demand is likely to exceed workzone capacity. The portion of demand exceeding capacity in one time period will form a queue and be transferred to the next time period. If the demand exceeds capacity in several consecutive time periods, the queue will grow over the time periods. The queue decreases only with the time periods where the demand is less than capacity. For a given time period, if existing, the queue length is a function of time and is expressed in (8)

$$q(t) = q(0) + (v_t - c/T) \cdot t \qquad (8)$$

where $q(0)$ = initial queue length, equal to number of vehicles left from previous time period [$q(0) = v_{Lt-1}$]; v_t = demand or volume in current time period; c = workzone capacity; t = time; and T = length of time period.

The congestion delay over the time period is given in (9).

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$$d_c = \int_0^{T_1} \left[q(0) + \frac{v_t - c}{T} \cdot t \right] \cdot dt \qquad (9)$$

where T_1 = duration for which the queue lasts over the time period T. There are two possible cases concerning T_1:

1. $v_{Lt-1} + v_t < c$, that is, the sum of current volume v_t and traffic left from previous time period (v_{Lt-1}) is less than workzone capacity (c). In this case, the queue remaining from the previous time period will disappear within the current time period. Therefore, $T_1 < T$ and T_1 was found to be $T_1 = v_{Lt-1}/(c - v_t)$.

   Let T = 1 h. By performing the integration, hourly congestion delay was found to be $d_{hc} = v_{Lt-1}(v_{Lt-1})/[2 \cdot (c - v_t)]$, where d_{hc} = hourly delay (veh-hours). By the end of this hour, no queue is transferred to the next hour.
2. $v_{Lt-1} + v_t >= c$. In this case, the queue will last for the entire time period, that is, $T_1 = T$. If $v_t < c$, the queue length will be reduced by $(c - v_t)$ in the current time period. The rest of the queue will be further transferred into the next time period. If $v_t > c$, the length of the queue from the previous time period will be increased in this time period. Hourly congestion delay in this case was found to be $d_{hc} = v_{Lt-1} - (v_t - c)/2$. By the end of this hour, the length of the queue to be transferred to the next hour is $v_{Lt} = v_{Lt-1} + v_t - c$.

Procedure 4 shows the steps to determine the daily congestion delay for general segments.

Procedure 4

1. Step 1. Input data, including ADT, truck percent, type of terrain, and roadway conditions.
2. Step 2. Convert ADT to hourly volumes.
3. Step 3. Determine workzone capacity using (4) and given conditions.
4. Step 4. Determine hourly congestion delay using (9).
5. Step 5. Integrate the hourly congestion delays (d_{hc}) into daily congestion delay (d_{dc}).

The relationship between daily congestion delay and ADT under different types of terrain was presented in Fig. 3. As shown in the figure, daily congestion delay is much more significant under mountainous terrain than under level terrain. Under level terrain, there is no daily congestion delay if ADT is less than 30,000 vpd. The effect of truck percent is not significant. Under rolling terrain, the relationship is affected by truck percent. There is not much daily congestion delay if ADT is less than 30,000 and truck percent is less than 15%. However, when truck percent is 25%, daily congestion delay begins to emerge at ADT = 20,000 and increases with ADT. Under mountainous terrain, daily congestion delay occurs even with a low ADT when truck percent is greater than 15%. For a given specific upgrade segment, the actual workzone capacity varies with the steepness and length of grade. The procedure to determine daily congestion delay for a specific upgrade segment is similar to that for a general segment except the difference in step 4, determining workzone capacity. The workzone capacity should be determined with (5). PCEs were selected under the minimum moving speed with two-way two-lane operations. By regression analysis of the tabulated values in the HCM (Transportation 1985), the following equations can be used to calculate PCEs for any given steepness and length of grade (for l > 1 mi, R^2 > 0.99 in all cases).

- At 3% grade: PCEs = $1.1 + 0.93 \cdot l$ where l = length of grade

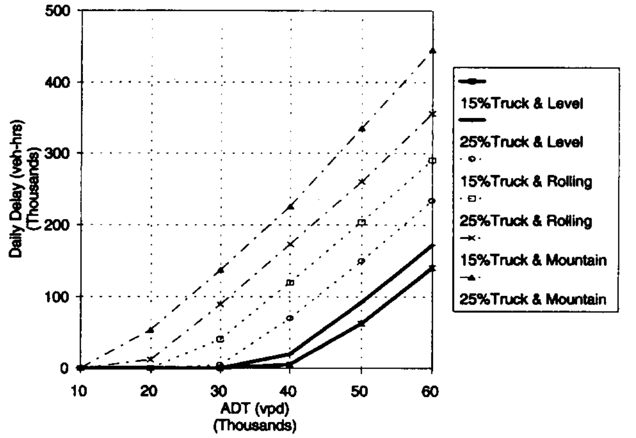

FIG. 3. Relationship between Daily Congestion Delay and ADT

- At 4% grade: PCEs = $0.6 + 1.67 \cdot l$
- At 5% grade: PCEs = $1.94 + 0.13 \cdot l + l^2$
- At 6% grade: PCEs = $2.2 + 0.2 \cdot l + 0.94 \cdot l^2$

EVALUATION OF LONGITUDINAL ROADWAY CLOSURE PATTERNS

For a given four-lane divided freeway reconstruction project with a total length of L, (e.g. 15 mi). There are three possible alternative configurations for closing the roadway in one direction along the length of the project (Fig. 4). They are as follows:

Alternative I—Full Length Closure

The roadway in one direction is closed along the entire length of the project and construction work is undertaken at one time [Fig. 4(a)]. The advantages of this roadway closure pattern include less traffic control devices required and less disturbance to the construction work because there is little conflict between construction activities and traffic operations. The disadvantages of this alternative are more traffic delay since speed reduction delay is directly proportional to the length of the two-lane two-way segment; and higher accident cost due to severer segment accidents on the long two-lane two-way segment.

Alternative II—Partial Length Closure with Multiple Workzones

The project is divided into several segments, (e.g. five segments each three miles long). Construction work is undertaken in every other segment at the same time. Thus, there are two construction periods, as shown in Fig. 4(b). The first period includes segments 1, 3, and 5. The second period includes segments 2 and 4.

The advantages of this roadway closure pattern are the following: (1) Possible higher capacity under mountainous terrain or specific upgrade segments because when a long workzone is broken into a number of short workzones and there is a normal freeway segment between workzones for light vehicles to pass the heavy vehicles, the effect of heavy traffic is reduced so that the capacity and average traffic speed are increased; and (2) less delay in comparison with alternative I.

The disadvantages of this alternative include the following: (1) Higher traffic-control costs because it requires five crossover systems for which signs, channelization, and other traffic control devices must be purchased and installed; (2) more accidents at the ends of the workzones; and (3) higher additional

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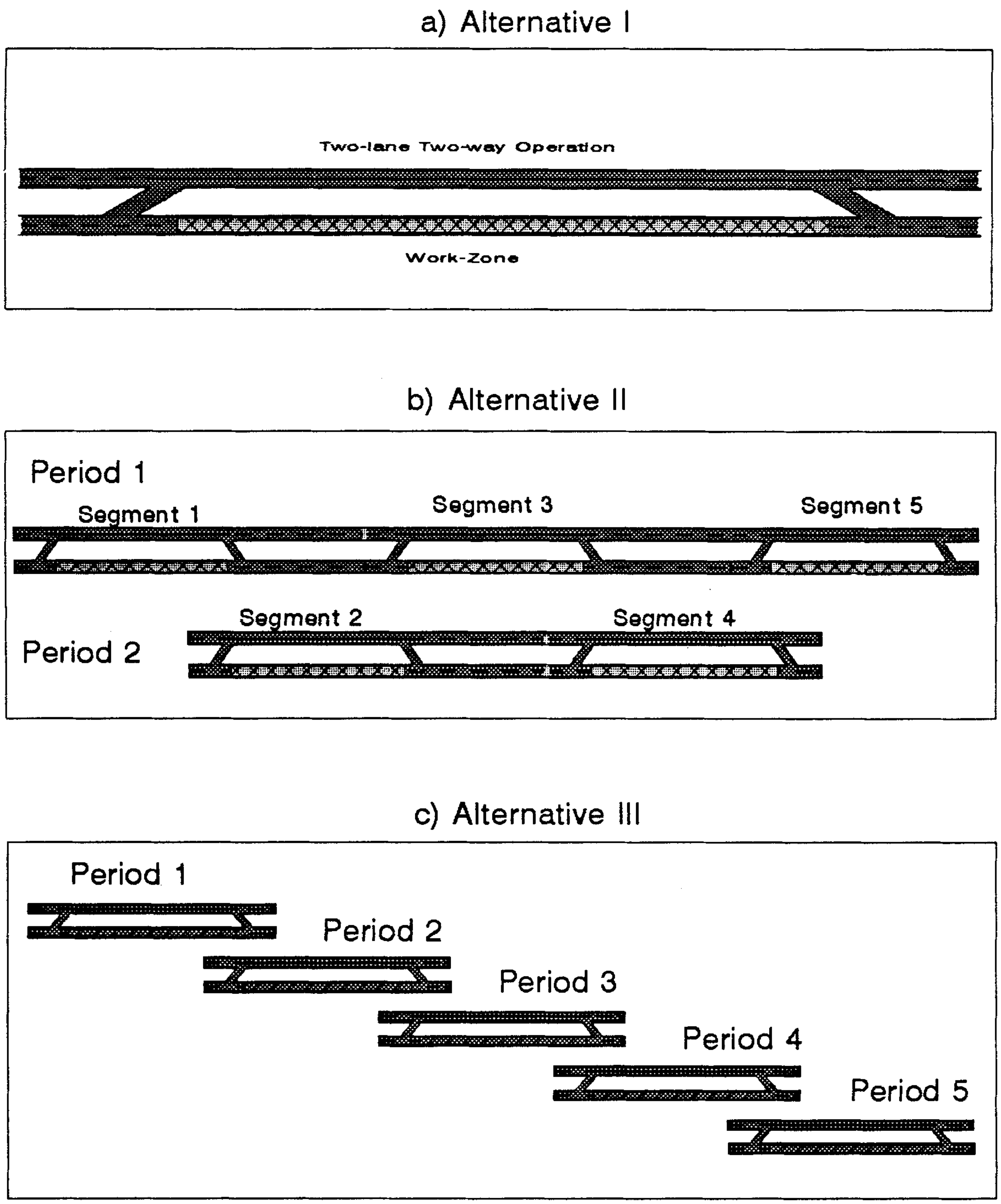

FIG. 4. Alternative Roadway Closure Patterns

operation cost due to more acceleration-deceleration cycles at the ends of workzones.

Alternative III—Partial Length Closure with Single Workzone

The project is divided into several segments, as shown in Fig. 4(c). Reconstruction work is undertaken only on one segment at a time. Thus, there are five construction periods. The traffic needs to travel only a three mile workzone throughout the entire reconstruction duration.

The significant advantages of this alternative include the following: (1) Shortened workzone and less speed reduction delay; (2) higher workzone capacity under upgrade conditions; (3) less traffic control cost in comparison with Alternative II because only one workzone is in operation at a time and the devices can be reused for the next workzone; and (4) low accident rates due to shortened workzones and only one active pair of crossovers. The disadvantages of this alternative appear to be higher traffic control cost (because it requires five pairs of crossovers and associated control facilities), and possible longer construction duration (because the project is divided into several time periods and it requires additional time to move the workzone from one segment to another).

In order to find the optimal longitudinal roadway closure pattern among the possible alternatives, the alternatives were evaluated based on the following attributes:

- Total delay cost due to construction work with the project
- Additional operating cost with the project
- Total traffic-control cost, including control devices, their

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installation, channelization, and median crossover construction

- Additional accident cost with the project

Total delay cost at the project level can be estimated with (10).

$$C_{DP} = c_T \cdot D \cdot (d_{ds} \cdot l + d_{dc}) \tag{10}$$

where C_{DP} = total delay cost at project level (\$); c_T = time value (\$/veh-hr) and assuming c_T = \$8/veh-hr; l = total length of workzones at same time (miles); d_{ds} = daily speed-reduction–related delay (veh-hrs/mile); d_{dc} = daily congestion delay (veh-hrs); and D is project duration (d).

In order to estimate traffic-control cost, it was assumed that a traffic-control system consists of signs, channelization devices, flashing arrow panel, pavement marking, and crossovers. Using (*Means* 1993) Construction Cost Guides, the cost for each component was estimated as follows:

- Warning signs, total \$3,280
- Flashing arrow panel, \$1,200 each
- Channelizing devices on the crossovers and their approaches, \$10,735 total, where cones and portable concrete barrier are assumed to be used
- Pavement marking \$600, not including two-lane two-way separation marking
- Crossover and taper construction cost, total \$34,737

The total traffic control costs at the project level are Alternative I: \$50,547; Alternative II: \$232,961; and Alternative III: \$213,087. It was assumed that traffic-control cost within the workzone segment is not affected by the different alternatives.

Additional daily accident cost consists of crossover accident cost and additional segment accident cost. According to national statistics and empirical data (McCoy et al. 1980; U.S. 1987), the current accident rates are estimated to be 34.3 segment accidents per 100 million veh-mi with normal freeway operations, 11.5 segment accidents per 100 million veh-mi with two-lane two-way operations, and 90.4 crossover accidents per 100 million entering vehicles. Additional accident cost was calculated to be \$4,160 per crossover accident and \$52,839 additional segment accident cost per 100 million veh-mi.

Additional daily operating cost consists of cost due to deceleration and acceleration at the ends of a workzone, estimated to be \$0.0461/(per 60 mph-49 mph-60 mph speed change cycle) and saving due to a lower travel speed through a workzone, estimated to be −\$0.007/veh-mi.

In order to compare total cost at the project level, the duration of the project should be estimated. The construction rate was assumed to be 1/6 mi per day. Therefore, the duration of the project is 90 d. Assuming that a three-mile workzone is long enough to layout all the construction work, the construction rate will not be affected by selecting different alternatives. Project duration can be assumed to be the same for the three alternatives. Total costs for the alternatives at the project level under level terrain are presented in Table 4. It was found that for ADT < 10,000 and level or rolling terrain, the full-length closure, Alternative I, is the best choice. This is because under these conditions, delay cost is not significant, traffic control cost dominates the total cost, and Alternative I has the lowest total cost. If ADT is greater than 20,000, partial length closure with single workzones, Alternative III, is the best choice regardless of any other conditions because the delay cost is more significant than others when ADT is greater than 20,000. When ADT is greater than 40,000, total costs of the three alternatives are all high. This is because congestion delay cost is far more than any other cost and it is the same in each alternative. However, congestion delay does not affect the best choice. Regardless of traffic conditions, Alternative II is not the best choice. Similar results were obtained under rolling and mountainous terrain.

TABLE 4. Cost Comparison of Alternative Roadway Closure Patterns

Truck percent (1)	ADT (2)	ALT I (\$) (3)	ALT II (\$) (4)	ALT III (\$) (5)
5	10,000	220,377	415,513	282,801
	20,000	557,337	678,337	386,211
	30,000	1,136,307	1,070,323	537,897
	40,000	2,688,195	2,355,330	1,530,129
	50,000	23,749,707	23,200,770	22,115,109
	60,000	76,523,727	75,839,113	74,547,381
15	10,000	234,417	422,814	285,609
	20,000	609,177	705,294	396,579
	30,000	1,235,667	1,121,990	557,769
	40,000	5,870,019	5,469,244	4,598,769
	50,000	47,465,787	46,903,372	45,808,725
	60,000	104,500,767	103,820,819	102,532,197
25	10,000	247,377	429,553	288,201
	20,000	658,857	731,127	406,515
	30,000	1,401,339	1,234,267	634,449
	40,000	16,329,531	15,909,057	15,025,449
	50,000	68,810,907	68,243,826	67,146,069
	60,000	127,283,367	126,604,456	125,316,525

OPTIMAL WORKZONE LENGTH

The optimal workzone length is that which minimizes the sum of traffic control and road user costs, in order to determine the optimal workzone length, the relationship between the workzone length and the sum of traffic control, traffic delay cost, additional traffic operation cost, and additional accident cost at the project level should be established. Total traffic delay cost includes total speed reduction delay cost and total congestion delay cost and was already formulated in (10).

Traffic control facilities of a reconstruction project were classified as the end approach control system and the segment control system. The end approach control system consists of median crossovers, signs, flashing-arrow panels, the channelization devices that are installed at the ends of a workzone. The segment control system includes separation devices or pavement marking and signs used within the two-lane two-way segment. Eq. (11) formulates total traffic control cost

$$C_{TC} = (c_c + c_{el}) \cdot N + c_s \cdot l \cdot N + c_{em} \tag{11}$$

where C_{TC} = total traffic control cost at project level; c_c = cost of constructing crossovers for workzone; c_{el} = labor cost of installing and removing end traffic control facilities; c_{em} = material cost of end traffic-control facilities, which can be reused; N = number of workzones, $N = L/l$; L = length of the project; l = length of a workzone; and c_s = segment traffic control cost per unit length of two-lane two-way segment.

Additional traffic operating costs include the cost of speed change cycle at the ends of a workzone and the saving of travelling through a workzone. It is formulated as

$$C_{AO} = (c_{ad} - s_s \cdot l) \cdot ADT \cdot D \tag{12}$$

where C_{AO} = total additional operating cost at project level; c_{ad} = cost per speed change cycle; and s_s = saving per unit length of workzone traveled by a vehicle.

Additional accident costs include crossover accident cost and additional segment accident cost, as shown in (14). Additional segment accident cost is the difference between segment accident cost with normal freeway operations and that with two-lane two-way operations.

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$$C_{AA} = (c_{ax}a_x + (c_{al}a_l - c_{an}a_n)\cdot l)\cdot ADT\cdot D \quad (13)$$

where C_{AA} = total additional accident cost at project level; c_{ax} = cost per crossover accident; a_x = crossover accident rate; c_{an} and a_n = cost per segment accident and segment accident rate with normal freeway operations, respectively; and c_{al} and a_l = cost per segment accident and segment accident rate with two-lane two-way operations, respectively.

Adding all the cost components together, the relationship between the sum of the costs and the workzone length is given in (14).

$$C_p = \{c_T\cdot(d_{dc} + d_{ds}\cdot l) + [(c_{ad} - s_s\cdot l) + c_{ax}a_x + (c_{al}a_l - c_{an}a_n)\cdot l]\cdot ADT\}\cdot D + (c_c + c_{el})\cdot L/l + c_s\cdot L + c_{em} \quad (14)$$

The optimal workzone length can be determined by minimizing the objective function C_p and subjecting l to the constraints of $l = L/[\mathrm{int}(L/l)]$, and $l \geqq l_{\min}$ where $l_{\min}$ is required by construction work.

For general freeway segments, daily congestion delay (d_{dc}) and speed reduction delay per unit length of workzone (d_{ds}) are not related to the length of a workzone. The optimal workzone length can be solved by letting the derivative of (14) with respect to workzone length l equal zero. The optimal workzone length is found to be

$$l_{\mathrm{opt}} = \sqrt{\frac{(c_c + c_{el})\cdot L}{D\cdot[c_t\cdot d_{ds} - ADT\cdot(s_s - c_{al}\cdot a_l + c_{an}\cdot a_n)]}} \quad (15)$$

For general freeway segments, the procedure to find the optimal workzone length of a given project consists of the following steps:

Procedure 5

1. Step 1. Given ADT, truck percent, type of terrain, roadway conditions, and project length, estimate crossover construction cost of a workzone, labor cost of installation, and removal of traffic-control facilities at the ends of a workzone, saving when traveling through a unit length of workzone, segment accident rate, and average cost per segment accident both with normal freeway operations and with two-lane two-way operations, and the duration of the project.
2. Step 2. Find daily speed reduction-related delay per unit length of workzone using procedure 1.
3. Step 3. Determining the optimal workzone length using (15).
4. Step 4. Let N = integer(L/l_{op}), actual workzone length $l = L/N$.
5. Step 5. If $l < l_{\min}$ let $l = l_{\min}$, stop.

For specific upgrade segments, both daily congestion delay d_{dc} and speed reduction delay per unit length of workzone d_{ds} in (14) depend on the length of a workzone. An explicit relationship between the sum of costs and the workzone length cannot be obtained. Therefore, the derivative of (14) with respect to workzone length l cannot be carried out and it is impossible to use analytical method to determine the optimal workzone length. However, the objective function C_P still has a minimum value. With the help of computers, procedures and programs can be developed to find an approximately optimal workzone length. The following presents a procedure for determining the optimal workzone length for a specific upgrade freeway reconstruction project.

Procedure 6

1. Step 1. Given ADT, truck percent, grade, roadway conditions, and project length, estimate traffic control cost, additional operating cost, and additional accident cost-related indices shown in (14).
2. Step 2. Given an initial workzone length l_o, and increment Δl arbitrary, $\Delta l > 0.8$ km.
3. Step 3. Calculate congestion delay and speed reduction delay with $l - \Delta l$, l, and $l + \Delta l$, respectively, using procedures 3 and 4.
4. Step 4. Calculate $C_p(l - \Delta l)$, $C_p(l)$, and $C_p(l + \Delta l)$ using (14).
5. Step 5. If $\Delta l < 0.01$ mi, let l_o be the optimal length. Let N = integer(L/l_o) and $l = L/N$, if $l < l_{\min}$, let $l = l_{\min}$, stop. Otherwise, do step 6.
6. Step 6.

   If $C_p(l - \Delta l) > C_p(l) > C_p(l + \Delta l)$, let $l = l + \Delta l$ and go to do steps 3, 4, and 5.
   If $C_p(l - \Delta l) < C_p(l) < C_p(l + \Delta l)$ let $l = l - \Delta l$ and go to do steps 3, 4, and 5.
   If $C_p(l - \Delta l) > C_p(l) < C_p(l + \Delta l)$, let $\Delta l = \Delta l/2$ and go to do steps 3, 4, and 5.

If a reconstruction project involves roadways in both directions, to use (14) or (15), the construction duration should be doubled. The crossover systems constructed for one direction can be reused for the other direction except those at the two ends of the project. Therefore, the number of crossover systems for both directions is (N + 1).

CONCLUSIONS

- Traffic delays due to workzones can be classified as speed reduction delay (moving delay) and congestion delay (stopping delay). Highway capacity and level of service analyses appear to be a proper technique to estimate these two types of delay.
- Speed-reduction delay results from the fact that a vehicle moves at a lower speed on a workzone than on a normal freeway segment. Actual moving speed of a vehicle on a workzone segment or on a normal freeway segment varies with ADT, traffic composition, and terrain condition.
- An effective procedure for estimating speed reduction delay is to convert a given ADT to hourly volume, estimate the delay on an hourly basis, and cumulate hourly delay into daily delay.
- For general freeway segments, speed reduction does not depend on the length of a workzone. So, speed reduction related delay is a linear function of the length of a workzone. For specific upgrade segments, speed reduction depends on the length of a workzone. Therefore, the relationship between speed reduction delay and the length of a workzone is more complicated. Speed reduction delay increases with ADT, percent truck and varies with terrain condition.
- Congestion occurs when the hourly traffic volume is greater than the capacity of a workzone for a significant period of time. The amount of congestion delay depends on the length of the queue upstream of a workzone and the duration of congestion. Congestion delay is directly proportional to the hourly volume and increases with truck percent. For general freeway segment, congestion delay does not depend on the length of a workzone; however, it does for specific upgrade freeway segments.
- Traffic delay due to a workzone can be reduced by reducing the length of the workzone or changing the roadway closure pattern because total delay increases with the length of a workzone. However, reducing the length of a workzone will increase traffic control cost at the project level. There is a best trade-off between traffic control cost

and additional road user cost in respect to the length of a workzone.

- For a given project, the best roadway closure pattern is either closing the full length of the project or closing a segment of the project at one time, depending on the optimal workzone length under given conditions.
- The optimal workzone length is that which would minimize the sum of traffic control costs and the road user costs and can be determined for any given condition utilizing procedures developed in the present study.

Studies to determine whether or not the construction duration and cost at the project level will be affected by changing the length of a workzone are valuable. If overall construction cost depends on the length of a workzone, it should be considered in the study of the optimal workzone length. Traffic-control cost, additional accident cost, and additional operating cost need to be updated to find the best trade-off between traffic-control cost and road-user cost.

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