혼돈지역을 이용한 도로-철도 교차로의 사각차단기 운영시간 설정

Design of Four Quadrant Gate Operation Times using Dilemma Zone at Highway-Rail Intersections

Young-Jun Moon
Research Associate
The Korea Transport Institute

Fred Coleman, III
Dept. of Civil Engineering,
Univ. of Illinois at Urbana-Champaign

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초 록

이 연구는 도로상의 신호사거리에서 신호변환시 발생하는 혼돈지역의 개념을 이용하여 도로-철도 교차로에 설치되는 사각차단기의 운영시간을 결정하는 방법론을 제시한다.

이 방법론은 현재 미국 일리노이주에서 시카고-세인트루이스의 고속전철 노선중 6개의 교차로에 설치중인 사각차단기의 운영시간을 설정하여 제시함으로써 현장검증이 되었다. 특히, 운전자가 교차로 진입시 차단기 사이에 차량이 걸리지 않으며 혹은 정지시 안전하게 교차로 앞에서 정지하도록 하는 의사결정이 이루어 지도록 최적의 차단기 운영시간을 운전자에게 제시하였다.

I. Introduction

Four quadrant gates (QG) are an additional pair of dual gate arms which are lowered on each side of a bi-directional crossing preventing any vehicle from crossing in-between the lowered gates because their travel path is blocked on the front and rear side of the crossing. The implementation of QG, while eliminating gate arm violations, does present the potential for trapping a vehicle. Similar to a signalized highway-highway intersection, vehicles approaching at various speeds must be allowed to clear the intersection during the yellow change interval. The highway-rail intersection must therefore be analyzed relative to a change interval based on the ability of vehicles to clear the intersection. Driver behavior relative to stopping or proceeding at various speeds is the primary determinant of the likelihood of clearing the intersection. If implementation of QG is to be undertaken, there needs to be design criteria which assure the safety of motorists based on human factors related to the operation of QG and the likelihood of clearing the crossing or stopping in an appropriate manner.

Two elements are taken into consideration in the operation of QG: a) gate delay, the time interval after initiation of flashing lights before initiation of entry gate descent, and b) gate interval time, the time interval between entry and exit gates descent to assure that vehicles will have sufficient time to stop or clear the crossing. A primary concern is focused on determining these time operating parameters to ensure a safe system operation in terms of eliminating the possibility of a vehicle becoming "trapped" between the entry and the exit gates.

Recent studies have reported on driver behavior during flashing light operation while a train is approaching the highway-rail intersection. Shinar and Raz (1982) observed 367 drivers at crossings with flashing lights only and flashing lights and gates. Driver behavior was associated with approach speed near the intersection. They found drivers reduced their approach speed before the intersection under all conditions: by 10.7 kph average when the lights were on and the gates lowered and by 5.2 kph average when the lights were off and the gates were raised. Tenkink and Van der Horst (1988) observed more than 900 drivers: 660 while confronted with the red lights and 272 while passing the white lights indicating that no train is approaching. Variables measured were approach speed, vehicle position at the onset of red phase, headway, deceleration rate, and travel time to stop line. Red light compliance was studied by measuring the decision-making behavior at the onset of, and during the red light phase. Drivers with a high degree of compliance were observed to have reactions such as emergency braking and hesitations, which they concluded were the result of an unexpected event. They indicated also in 10% of cases, drivers tended to decelerate more strongly than necessary during the white signal phase. Lerner and Ratte (1990) reviewed driver behavior and identified important human factors issues which contribute to driver error in decision making, such as information limitations and credibility, expectancies regarding trains and crossings, emotional reactions, temporal constraints due to sight distance, vehicle speed, and the complexity of decisions, etc. Moreover, there are several methods (Richards and Heathington, 1988; Aberg, 1988) which have been suggested to estimate human reactions as a probability of stopping at flashing lights.

In summary, these studies suggest that an important variable to be considered in characterizing driver behavior at highway-rail intersections is vehicle approach speed.

The purpose of this research is to determine the operating parameters which insure a safe system operation with the integration of QG into existing crossing safety systems such as flashing lights and signals.

II. Dilemma Zone Concepts

1. Dilemma Zone Concept at Highway-Highway Intersections (HHI)

The dilemma zone concept refers to the research and methodology pioneered in traffic engineering related to drivers decisions to stop or proceed at the onset of the yellow change interval. Sheffi and Mahmassani (1981) state "The dilemma refers to the drivers decision to proceed through the intersection or to stop when the signal indication changes from green to amber". The concept of a dilemma zone was recognized in the work of Gazis et al. (1960), Olson and Rothery (1972), Crawford (1962) and Herman (1963). It is further defined by Sheffi and Mahmassani (1981) ''as that zone within which the driver could neither come to a stop nor proceed through the intersection before the end of the amber phase". Figure 1 illustrates the definition of dilemma and option zones which a driver faces when he/she is approaching a highway-highway intersection. An option zone is defined as the zone within which the driver could either come to a stop or proceed through the intersection before the end of the yellow interval. Continued work on this concept has led to a probabilistic approach to a driver stopping, with Zegeer (1977) defining a dilemma zone as "the road segment where more than 10% and less than 90% of the drivers would choose to stop." Sheffi and Mahmassani (1981) indicate "The approach consists of developing dilemma zone curves of percent drivers stopping versus distance from stop bar at the instant when the signal indication changes from green to amber."

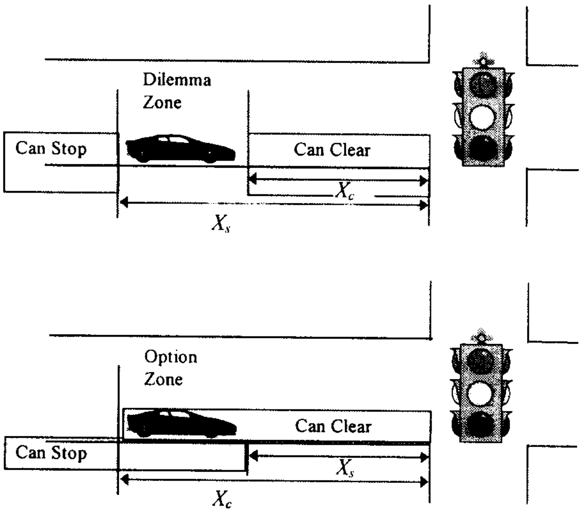

FIGURE 1 Definition of Dilemma and Option Zones at Signalized Intersections (*Xs* = Stopping Distance; *Xc* = Continuation Distance).

2. Basis for Dilemma Zone at Highway-Rail Intersections (HRI)

Drivers approaching a highway-rail intersection are faced with a similar scenario in which a visual signal in the form of flashing lights informs the driver of the need to stop, before the descent of the gates. The similarity to a signalized highway-highway intersection is that when drivers are some distance away from an intersection with gates when commencement of flashing lights occur, they must make a decision to stop or proceed. Therefore, determination of the zone boundaries for highway-rail intersections based on "safe stopping distances" and the "continuation distance" given the approach speed and the width of the intersection using typical values of acceleration and deceleration rates are possible, similar to work performed by Gazis et al. (1960).

Figure 2 shows the definition of dilemma and option zones which a driver faces when he/she is approaching a highway-rail intersection with four quadrant gates (QG).

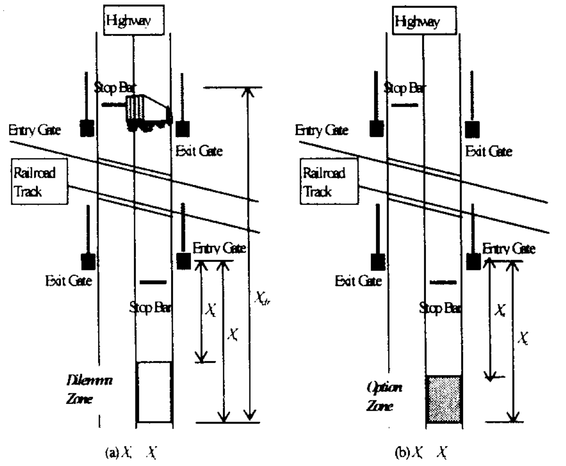

FIGURE 2 Definition of Dilemma and Option Zones: (a) Dilemma Zone; (b) Option Zone.

III. Design of Gate Operation Times

1. Distance Variables

In order to design the gate delay and gate interval time at highway-rail intersections with four quadrant gates, a dilemma zone is established in terms of the relationship between stopping distance (Xs), continuation distance (Xc), and clearance distance ($Xclr$). A driver approaching a highway-rail intersection during gate delay, the time that the flashing light signals are operated before the entry gates are activated, will either have to stop or proceed to clear the intersection. Figure 3 shows the QG system of an intersection and includes all geometric data for calculating stopping distance as well as clearance distance.

The stopping distance is the distance required for vehicles to stop at the stop bar, which is usually 2 - 3 meters (8 ft) in front of the gate arm. This stopping distance for highway-rail intersections is the same as for signalized intersections, except that it includes the distance between stop bar and gates. It is formulated as follows:

$$X_s = \Delta T \cdot v + \frac{v^2}{2 \cdot (d + G \cdot g)} + D \qquad (1)$$

where

Xs = stopping distance (m);

ΔT = driver perception-reaction time (PRT) (sec);

v = approach speed (m/sec);

d = typical deceleration rate for stopping on level pavement (m/sec^2);

G = acceleration due to gravity (m/sec^2);

g = grade of approach lanes (percent/100); and

D = distance between stop bar and gates (m).

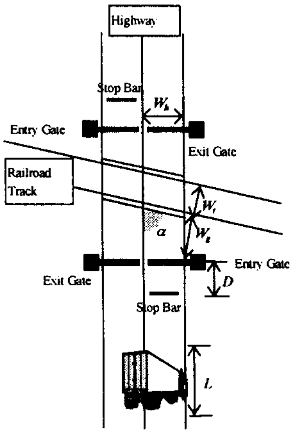

FIGURE 3 Geometric Data for Calculating Stopping Distance and Clearance Distance.

The stopping distance formula utilized is same as that employed by ITE Technical Committee 4A-16 (1985) in the development of the dilemma zone concept. The only difference in the stopping distance formula between the highway dilemma zone concept and its application here is that it includes the distance D between stop bar and gates. AASHTO (1990) uses a slightly different stopping distance formula which includes f a pavement friction factor based on assumed conditions of speed and wet pavement. The AASHTO formulation for stopping distance is recommended for implementation of this methodology. However, consistent with prior research and methodological work on the concept of dilemma zone by Gazis et al. (1960), Olson and Rothery (1972), Crawford (1962), Herman (1963), and Sheffi and Mahmassani (1981), their formulation for signalized intersections will be applied in this methodology.

The continuation distance is defined as the distance a vehicle travels during initiation and/or the interval of gate delay prior to descent of the entry gates, with no change in speed that results in the vehicle position perpendicular to the entry gate. The continuation distance is then the vehicle position at the start of gate delay (or within this interval) and a position attained immediately prior to initiation of gate descent. Figure 2 indicates where the continuation distance, Xc, could occur. The clearance distance is defined as the distance a vehicle travels to clear the intersection through the exit gates, including vehicle length. The clearance distance is a function of the distance within which the vehicle can clear the intersection before the end of the quad gate operation time which consists of gate delay *plus* gate interval time. Figure 3 presents the clearance distance, $Xclr$, in relation to the stopping distance, Xs, and continuation distance, Xc.

2. Mathematical Relationships between Distance Variables

If a vehicle were to proceed to clear the intersection during the quad gate operation time, the vehicle would cover a clearance distance which consists of: (1) the continuation distance, (2) the distance between entry and exit gates, and (3) the length of the vehicle. The QG system of an intersection which includes all geometric data for calculating stopping distance as well as clearance distance is shown in Figure 4. Considering the approach speed, the clearance distance required for a vehicle to clear the intersection is formulated as follows:

$$X_{clr} = T_G \cdot v \tag{2}$$

where

$Xclr$ = clearance distance (m);
TG = quad gate operation time (sec); and
υ = approach speed (m/sec).

TG represents the total quad gate operation time, which contains both a gate delay component and a gate interval time component. The gate delay time component contains the vehicle travel time for the continuation distance, which effectively places the vehicle at the entry gate prior to entry gate descent. The gate interval time component contains the travel time for initiation of gate descent through the intersection geometry, which includes the distance between entry and exit gates, as well as the length of vehicle to clear the exit gates. Referring to Figure 3 and considering the intersection angle between the railroad and highway, as well as road segments immediately adjacent to the railroad track, the distance between entry and exit gates is formulated as follows:

$$\begin{aligned} W_{ght} &= \frac{W_t}{\sin\alpha} + \frac{2 \cdot W_h}{\tan\alpha} + \frac{2 \cdot W_g}{\sin\alpha}, \quad \alpha \le 90^\circ \\ &= \frac{W_t}{\sin(180-\alpha)} + \frac{2 \cdot W_h}{\tan(180-\alpha)} + \frac{2 \cdot W_g}{\sin(180-\alpha)}, \quad \alpha > 90^\circ \end{aligned} \tag{3}$$

where

$Wght$ = distance between entry and exit gates (m);
Wt = width of railroad track (m);
Wh = width of approaching lane of the highway (m);
Wg = distance from track edge to gate (m);
α = intersection angle (deg).

Then, the continuation distance required for a vehicle to continue its approach during the initiation and/or gate delay interval to arrive at the stop bar is formulated as follows:

$$X_c = X_{clr} - (W_{ght} + L) \tag{4}$$

where

Xc = continuation distance (m); and
L = length of the vehicle (m).

As shown in Figure 2(a), when a vehicle approaches an intersection during gate delay, if $Xs > Xc$ and the vehicle is positioned between Xs and Xc such that $Xs > X > Xc$, then a dilemma zone exists where a vehicle could neither stop nor clear the intersection. As shown in Figure 2(b), if $Xs < Xc$ and if the vehicle is positioned between Xs and Xc such that $Xs < X < Xc$, then an option zone exists where a driver can choose between stopping and clearing the intersection. If $Xs = Xc$, the dilemma and option zones are eliminated and a point or distance is obtained which assures the likelihood of drivers stopping or clearing the intersection. This distance is called a "Safe Decision Location".

3. Determining Gate Delay and Gate Interval Time

When $Xs = Xc$, the two distances are equal, meaning that the point where this occurs eliminates the option zone and the dilemma zone.

Based on the "Safe Decision Location", where $Xs = Xc$, the simplification results as follows:

$$\Delta T \cdot v + \frac{v^2}{2(d + G \cdot g)} + D = v \cdot T_G - (W_{ght} + L) \tag{5}$$

This gives

$$T_G = \left\{\Delta T + \frac{v}{2(d + G \cdot g)} + \frac{D}{v}\right\} + \left\{\frac{1}{v}\left(W_{ght} + L\right)\right\}. \qquad (6)$$

The determination of an equal distance for stopping or continuing connotes that the driver has an opportunity to stop or an opportunity to continue up to the stop bar. When the total gate operation time (*TG*) is determined, the first term is the optimal gate delay time, while the second is the optimal gate interval time for the vehicle to clear with an assumption of constant speed. From Equation (6),

$$\begin{aligned} T_G &= T_D + T_I, \\ T_D &= \left\{\Delta T + \frac{v}{2(d + G \cdot g)} + \frac{D}{v}\right\}, \\ T_I &= \left\{\frac{1}{v}\left(W_{ght} + L\right)\right\} \end{aligned} \qquad (7)$$

where,

TD = gate delay (sec); and
TI = gate interval time (sec).

The gate delay (*TD*) is independent of the intersection angle, however the gate interval time (*TI*) must include the intersection angle. Gate delay is based on human factors and driver behavior. With a typical deceleration rate of 3.05 m/sec^2 (10 ft/ sec^2), similar to the highway-highway intersection studies, and with an approach speed of 56.4 kph (35 mph), 3.7 sec of gate delay is required. As a whole, the gate delay should be approximately 3.0 - 4.0 sec at ΔT (PRT) = 1 sec, or 4.5 - 5.5 sec at ΔT (PRT) = 2.5 sec based on AASHTO (1990) PRT values recommended for stopping distance. However, slightly longer times may be justified if vehicle approach speeds are over 64.4 kph (40 mph).

The gate interval time is the *total* gate interval time available. If the interval is assumed to begin at the start of entry gate descent, and a vehicle at the entry gate decides to clear the intersection only 2 - 3 seconds is available before the gate would come into contact with the vehicle. Utilization of this 2 - 3 seconds does not diminish the total gate interval time for this vehicle since the same amount of time to avoid contact with the exit gate is available.

In order to determine both gate delay and gate interval time for actual sites, six highway-rail intersections under consideration for four quadrant gates in Illinois were evaluated. Each sites operating and geometric data are shown in Table 1.

Table 2 shows the gate delay needs at the six sites utilizing a deceleration rate of 3.05 m/sec2 (10 ft/sec2). Gate delay with 1 second of PRT is approximately 4 seconds at an approach speed of 56 kph (35 mph) and 5 seconds with an approach speed of 72 kph (45 mph). For 2.5 seconds of PRT, approximately 5 seconds of gate delay at 56 kph and 6 seconds at 72 kph are required. Gate delay is independent of the intersection angle and depends only on the approach speed and deceleration rate, as indicated in the first term of Equation (7).

Table 1. Operational and Geometric Data for Selected Sites in Illinois

Location of Intersection (City Name in Illinois)	McLean	Spring-field	Hartford	Gardner	Pontiac	Chenoa
AADT (veh)	2800	18500	8600	3200	2900	360
No. of Tracks	2	1	2	1	1	1
Veh. Approach Speed (85th-Percentile), kph (mph)	72 (45)	56 (35)	64 (40)	56 (35)	40 (25)	40 (25)
Roughness	Good	Good	Good	Rough	Rough	Good
Assumed Min. Veh. Speed in Track Zone, *vt*, kph (mph)	8 (5)	8 (5)	8 (5)	5 (3)	5 (3)	8 (5)
Angle, α (Deg)	85	70	95	80	90	80
% Heavy Truck	16	3	0	5	0	5
Train Type	HS / Freight	Freight	HS / Freight	HS / Freight	HS / Freight	HS / Freight
Wt, m (ft)	6.1 (20)	1.5 (5)	16.8 (55)	1.5 (5)	1.5 (5)	1.5 (5)
Wg, m (ft)	3.7 (12)	6.7 (22)	4.6 (15)	3.7 (12)	4.3 (14)	3.7 (12)
Wh, m (ft)	3.4 (11)	9.2 (30)	5.5 (18)	3.1 (10)	3.7 (12)	2.8 (9)
D, m (ft)	2.5 (8)	2.5 (8)	2.5 (8)	2.5 (8)	2.5 (8)	2.5 (8)

Table 2. Gate Delay for Six Intersections in Illinois

Street of the Intersection, City	Approach Speed, kph (mph)	Gate Delay at ΔT (PRT) = 1	Gate Delay at ΔT (PRT) = 2.5
U.S. Route 136, McLean	72 (45)	**4.5**	**6.0**
N. Grand Ave., Springfield	56 (35)	**3.7**	**5.2**
Hawthorn St., Hartford	64 (40)	**4.1**	**5.6**
Main St., Gardner	56 (35)	**3.7**	**5.2**
Main St., Pontiac	40 (25)	**3.1**	**4.6**
Trunk Route 35A, Chenoa	40 (25)	**3.1**	**4.6**

Table 3. Gate Interval Time for Six Intersections in Illinois

Street of the Intersection, City	Min. Spd in Track Zone, kph (mph)	Gate Interval for Auto	Gate Interval for Truck
U.S. Route 136, McLean	8 (5)	**8.9**	**15.1**
N. Grand Ave., Springfield	8 (5)	**14.3**	**20.5**
Hawthorn St., Hartford	8 (5)	**14.7**	**21.1**
Main St., Gardner	5 (3)	**11.4**	**21.9**
Main St., Pontiac	5 (3)	**11.8**	**22.3**
Trunk Route 35A, Chenoa	8 (5)	**7.0**	**13.3**

Table 3 indicates the gate interval time needs at these sites. Five of the intersections are not at a right angle. In addition, the Gardner and Pontiac site intersection surfaces have been evaluated as "rough" (IDOT 1994), suggesting that minimum vehicle speeds are appropriate. All other sites evaluated as "good" have assumed crossing speeds of 8 kph (5 mph). All the intersections are level (0% grade).

Utilizing the geometry and assumed minimum speed in the track zone, the overall gate interval time varies from approximately 7 - 15 seconds for a passenger vehicle. For a WB-12 truck defined by AASHTO (1990) which has 19.8 meters (65 ft.) of vehicle length, approximately 14 - 22 seconds of gate interval time would be required for clearing the vehicle safely at the intersection. It should be noted that the major factors which influence the gate interval time are the minimum vehicle speed in the track zone, width of the intersection (*i.e.* distance between entry and exit gates), the length of the vehicle, and the angle of the intersection.

IV. Conclusions

The approach suggested along with the findings presented . utilizing site data from crossings in Illinois indicate that utilizing the concept of a dilemma zone provides a design procedure for gate delay and gate interval time. Further, it is demonstrated that by incorporating the geometry of a site, approach speed, width of crossing, and minimum assumed critical speed in the track zone, a sufficient gate interval time for automobiles could be provided. This overall design approach was validated through comparison of the Safe Decision Location and was found to provide essentially the same distance for a driver to stop or continue through the crossing, since the gate operation time included the necessary gate interval time to allow clearance of the crossing.

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