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DEVELOPMENT OF A MODEL FOR PREDICTING TRAVEL TIME ON AN URBAN FREEWAY

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Research Report 165-8

Development of Urban Traffic Management and Control Systems

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ABSTRACT

This report presents the development of a model for predicting the travel time required by a motorist to travel from any freeway location to the end of the freeway system during freeway incident conditions. Operating speeds and shock wave speeds can also be predicted. Typical incident solutions and travel time results are presented. The mathematical model was developed following the kinematic wave theory of Lighthill and Whitham for use in the operational control strategy of freeway information variable message signs.

Key Words: Freeway control, travel time, freeway incidents, shock waves, traffic diversion.

SUMMARY

Freeway incidents frequently occur and cause congestion on urban freeways even where surveillance and control systems are in operation. Methods are being sought for reducing the effects of these incidents through the use of variable message signs and other driver communication methods whereby motorists would be diverted from the freeway to alternate routes if conditions on the freeway relative to selected alternate routes justified the diversion. Travel times on the freeway and alternate routes are one measure that would be considered. The need existed on the Gulf Freeway surveillance project for travel time prediction capabilities since freeway traffic diversion was being planned.

To satisfy the need previously described, a model was developed for predicting the time required by a motorist to travel from any selected freeway location to the end of the freeway system during freeway incident conditions. The model is predictive in that it computes an estimate of a motorist's travel time if the motorist were to enter the freeway several minutes in the future. Freeway speeds, volumes and shock wave speeds are also predicted. The mathematical model of freeway incident conditions was developed following the kinematic wave theory of Lighthill and Whitham.

A computer program was written to compute the desired travel times when the initial freeway conditions and the characteristics of the incident were known. Four incidents were studied to determine the accuracy of the model when all conditions were known. The model was calibrated to these data with an error in travel time of no greater than

15 percent for 85 percent of the time.

Implementation

The travel time model is to be used as a part of a freeway traffic information and diversion system on the Gulf Freeway in Houston to divert motorists around congestion when an incident occurs on the freeway. The model may also be used to predict the effects of a lane closure in terms of lengths of queue backups and delays. The theory developed in this research with regard to describing incident conditions has already been helpful in describing more effective incident detection systems (1).

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INTRODUCTION

Freeway ramp control systems have proved their effectiveness in relieving freeway congestion when operations are free of incidents. Incident conditions, however, are a frequently occurring phenomena on urban freeways. Goolsby found that within a 6-mile section on the Gulf Freeway in Houston (2) over 13 lane-blocking incidents occur on the average during the time period of 6 a.m. to 7 p.m. from Monday through Friday. Stalled vehicles and accidents were the contributing causes of 97 percent of the incidents observed. Approximately 80 percent of the incidents reduced the capacity of the freeway by about one-half or more.

Freeway operational improvements have been proposed and/or implemented for improving the level of service provided during incidents. Several of these systems have consisted of some form of variable message signs (3, 4, 5, 6). One of the chief operational objectives of these signs is to increase the effective capacity of the freeway corridor during incidents on the freeway by achieving a higher utilization of the adjacent frontage road and surface street system. Driver preference questionnaire studies indicate that drivers will divert around congestion if accurate, reliable, and timely traffic information is provided to them. This diversion could occur from the freeway, at the frontage roads, or at major intersections located within the freeway corridor (7). One measure of the likelihood and desirability of diversion is the travel time saving that may occur to motorists if they were diverted (7, 8). This evaluation requires an estimate of the travel times along the alternate route and along the freeway during the incident conditions.

This paper presents the development of a method for predicting the time a motorist will require to travel from selected freeway locations to the end of the freeway system during incident conditions on the freeway. It is predictive in that it computes an estimate of what a motorist's travel time would be if he were to enter the freeway at a selected location some time in the future after the incident has occurred. Speeds, volumes, and other operational measures together with the speed and location of shock waves can also be predicted. Previous methods for calculating travel times have been based on measured or average speeds in fixed subsections (9, 10) rather than by predicting the changing traffic flow conditions.

DEVELOPMENT OF METHOD

Traffic Flow Theory

The deterministic theory of traffic flow has been shown to be very useful in describing freeway traffic conditions and in providing a basis for a rational explanation of certain observed traffic phenomena (11, 12, 13). The results of several approaches to the deterministic theory of traffic flow have been summarized by Drew (14) in his textbook on traffic flow theory and control. In general, the traffic flow theory has presented several methematical models that relate the traffic flow variables of volume, speed, and density.

One of the more frequently used deterministic theories of traffic flow is Greenshields' well-known linear speed-density model (11)

$$u = u_f - \frac{u_f}{k_j} k \qquad (1)$$

or

$$k = k_j - \frac{k_j}{u_f} u \qquad (2)$$

where

u = speed of the traffic stream

u_f = free speed as defined in Figure 1-a

k = density of the traffic stream

k_j = jam density as defined in Figure 1-a

Using the general equation of the traffic stream, $q = ku$, where q is the mean rate of traffic flow, the resulting parabolic relationships between traffic speed u and volume are formulated. Substituting from equation 2 for density k into $q = ku$ yields

$$q = k_j u - \frac{k_j}{u_f} u^2 \qquad (3)$$

A similar relationship exists between volume q and density k. Substituting from equation 1 for speed u in $q = ku$ yields

$$q = u_f k - \frac{u_f}{k_j} k^2 \qquad (4)$$

Equations 1, 3 and 4 are presented in generalized form in Figures 1-a, b and c, respectively. Also shown is the point on each of the respective curves that represents an assumed traffic flow condition existing on a section of freeway during normal operating conditions.

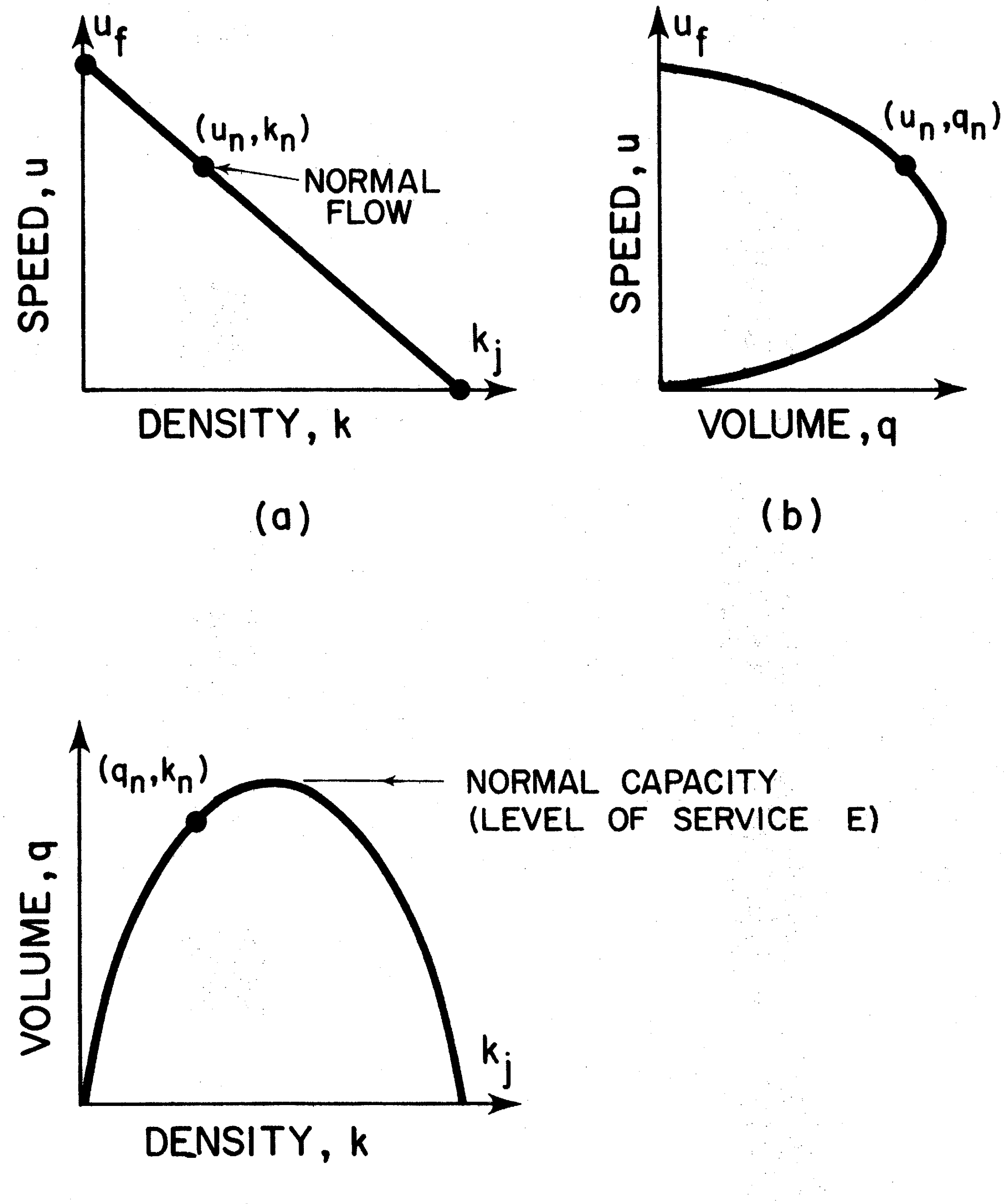

Figure 1 - Speed, Volume, and Density Relationships Using Greenshield's Model

Normal flow conditions exist when freeway traffic demand is less than capacity and there is no congestion or incidents on the freeway.

Initial Effects of Incident

When an accident occurs on a high volume freeway, it has been widely observed that a queue forms at the location of the accident. The queue and its resulting congestion then begin backing upstream from the scene of the bottleneck, often for several miles during peak hour operations. Whitson (15) has presented volume-density plots of freeway operations in Houston during an incident that clearly illustrates this upstream progression of the queueing area and its corresponding congestion. The frontal boundary of this queue, as it moves upstream, is commonly called the shock wave. Freeway surveillance of traffic operations during incidents has indicated that the shock wave commonly travels from 10 to 20 miles per hour during moderate to heavy traffic conditions.

Whitson (15) also noted that a wave moves downstream from the incident location. This wave denotes the change that occurs downstream of the incident from normal traffic flow to a much lighter flow. The reduction in the capacity of the freeway caused by an accident, or other lane blocking incident, thus meters the freeway flow downstream from the site of the incident, but causes a queue and congestion to form upstream of it.

Figure 2 presents a graphic summary of freeway traffic conditions upstream and downstream of the incident location while the incident blocks the freeway. The congested queue is bounded by the shock wave and the incident location with the queue having a nearly saturated density k_q

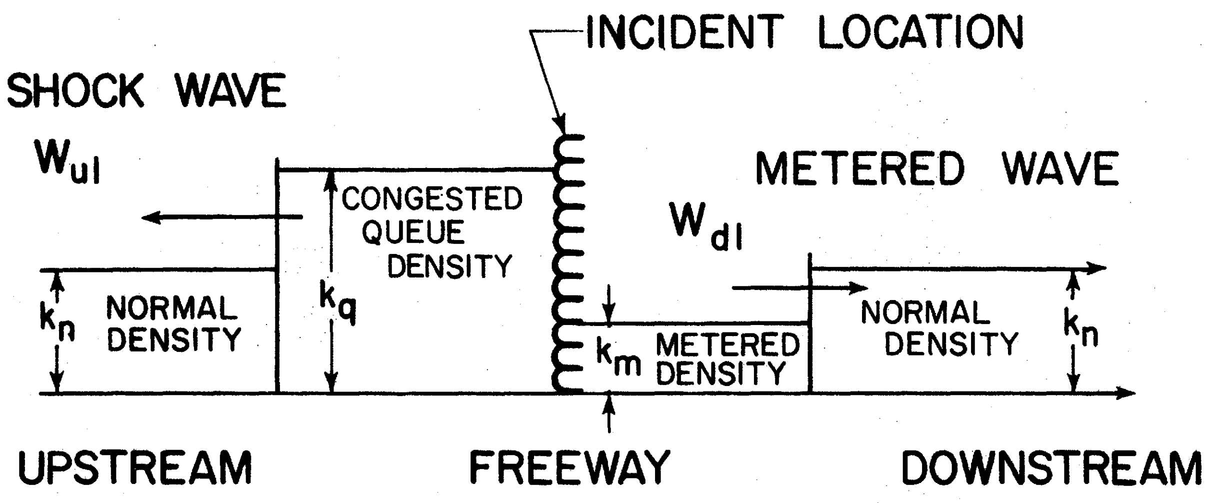

Figure 2 - Traffic Conditions While
Incident Exists on Freeway

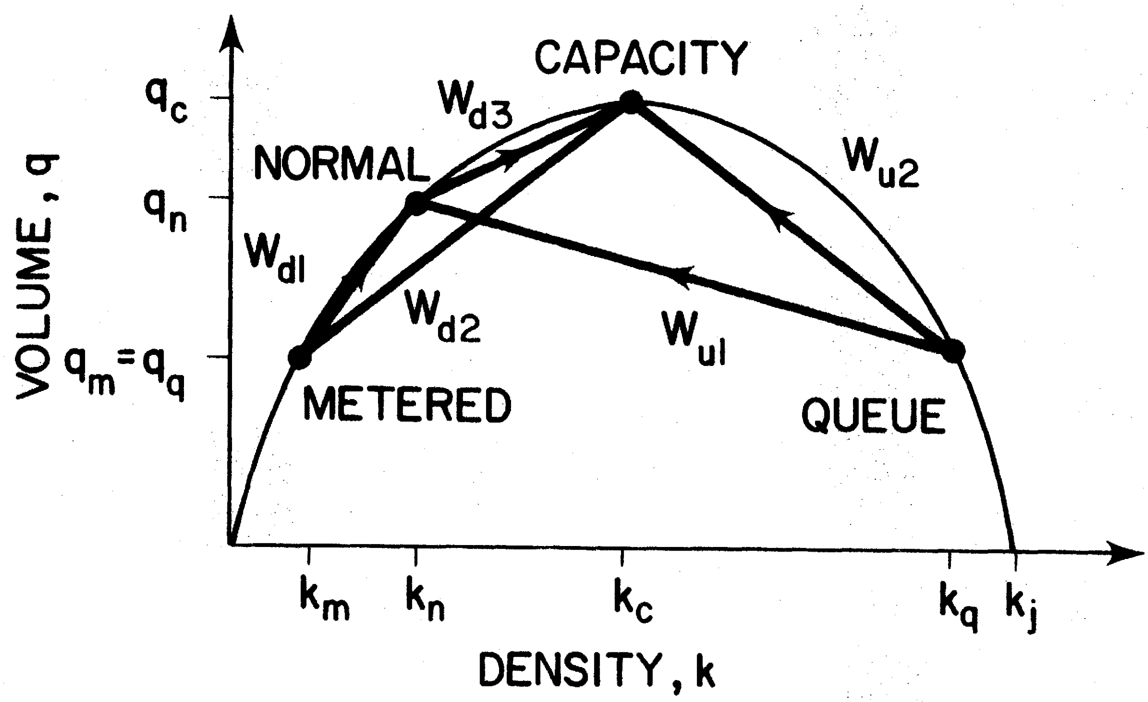

Figure 3 - Relationships of the Five Waves
Developed Due to an Accident

which is much higher than the normal density k_n (Also see Figure 3). Downstream of the incident in the metered flow region, the density is reduced from the normal density k_n existing before the incident to a much lighter metered density k_m, reflecting a higher mean traffic speed. The location of the clearing wave defines the boundary between the metered flow and the as yet undisturbed normal flow region.

Wave Theory

Lighthill and Whitham have presented a theoretical model for computing the speed of a shock wave based on changes in volume and density. The speed of the shock wave is given by (16)

$$W_{ul} = \frac{q_q - q_n}{k_q - k_n} \tag{5}$$

where

W_{ul} = the speed of the shock wave

k_q = traffic density in the congested queue

k_n = traffic density during normal operations

q_q = flow rate (traffic volume) in the congested queue

q_n = flow rate (traffic volume) during normal operations

The wave subscript notation refers to the direction of travel of the wave and the position number. That is, W_{ul}, the shock wave, is the speed of the first wave that travels upstream during incident conditions. W_{dl} would be the first wave traveling downstream. As was noted in Figure 2, the density k_q in the congested queue is greater than the normal density k_n. The incident is assumed to reduce the capacity of the freeway to q_q that is less than the normal flow q_n, which is a requirement if congestion is to form. Thus, the speed of the shock wave W_{ul} will be negative indicating the wave is moving upstream.

As illustrated in the volume-density curve in Figure 3, the speed of W_{ul}, the shock wave moving upstream from the location of the incident, is the slope of the chord that connects the point characterizing the traffic condition within the congested queue with the point characterizing normal traffic conditions. The negative speed of W_{ul} is also indicated in Figure 3 since the slope of the chord that defines W_{ul} from equation 5 is negative.

As shown in Figure 3, the traffic flow rate q_m in the clearing metered section downstream of the bottleneck incident is the same as the bottleneck flow rate q_q, but the density k_m within the metered area is much lower than the density k_q in the congested queueing section. The speed of the metered wave, which is the boundary between the metered and normal traffic operation, is

$$W_{d1} = \frac{q_m - q_n}{k_m - k_n} = \frac{q_q - q_n}{k_m - k_n} \qquad (6)$$

where

W_{d1} = speed of the clearing metered wave being the first wave moving downstream from the incident

q_m = flow rate (traffic volume) in the metered section

q_q = flow rate (traffic volume) in the queue and equals q_m

q_n = normal flow rate (traffic volume)

k_m = density in metered section

k_n = normal density

Since both the numerator and denominator of equation 6 are negative, W_{d1} is positive, indicating that the clearing metered wave is traveling downstream from the site of the incident bottleneck.

After a time T has elapsed since the incident occurred, the incident is assumed to be completely removed from the freeway, as shown in Figure 4. When the incident is removed, the capacity of the freeway is increased and the vehicles stored upstream of the site of the incident then begin to travel downstream. The flow of these vehicles out of the downstream end of the congested queue also begins to shorten or clear-up the queue upstream of the site of the incident. Figure 4 presents a summary of the traffic operating conditions along the affected sections of freeway from the time the incident begins until the freeway traffic operations return to normal sometime after the incident is removed. The shock wave W_{u1} and the clearing metered wave W_{d1} are depicted as the boundary vectors emanating upstream and downstream, respectively, from point A in Figure 4, which defines the beginning of the incident. The equations given in Figure 4 for the wave speeds are developed on pages 12-16.

The freeway traffic flow in the high density, high flow region, denoted as region c (capacity) in Figure 4, may be described as generally being unstable flow at or slightly under the maximum flow at normal capacity. For the purposes of this analysis, the average flow and density within this high density, high volume section is assumed to be at capacity, noted as the capacity point in Figure 3. As soon as the incident bottleneck is removed from the freeway, this unstable near capacity region of flow begins to travel both upstream from the incident location

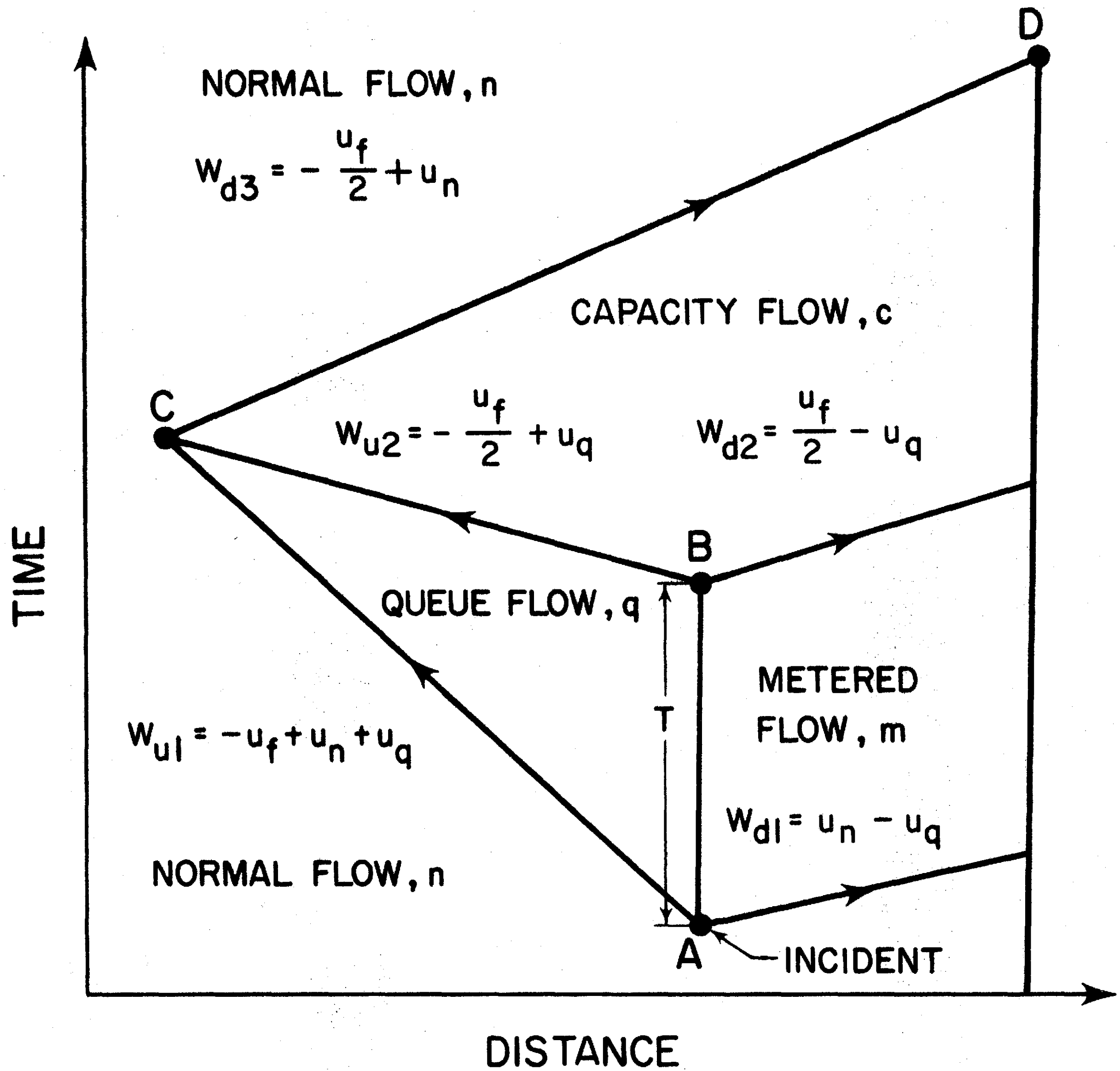

Figure 4 - Model of Freeway Traffic Conditions Due to an Accident

(point B in Figure 4), reducing the queue length, and downstream from the incident, increasing the flow and density downstream.

Associated with the movement upstream of the capacity flow region is the wave W_{u2} noted in Figure 4. Likewise, the wave W_{d2} moves downstream from the site of the incident (when it is removed) that defines the boundary between the capacity flow and the metered regions. Using Figure 3, it follows that

$$W_{u2} = \frac{q_c - q_q}{k_c - k_q} \tag{7}$$

where W_{u2} is the speed of the capacity boundary wave moving upstream, (q_c, k_c) and (q_q, k_q) define the volume-density operating conditions in the capacity flow region, c, and congested queue region, q, noted in Figure 3. Note in Figure 4 that W_{u2} is the second wave that travels upstream.

The boundary of the high density, capacity flow region travels downstream at a speed of

$$W_{d2} = \frac{q_c - q_m}{k_c - k_m} = \frac{q_c - q_q}{k_c - k_m} \tag{8}$$

where W_{d2} is the boundary wave speed, (q_c, k_c) and (q_m, k_m) define the volume-density operating conditions in the capacity flow region, c, and the clear metered region, m, respectively, noted in Figure 3 and Figure 4. Note again that $q_m = q_q$.

As indicated in Figure 4, one remaining wave occurs before the freeway traffic conditions return to normal. Sometime after the incident is removed, the capacity flow wave W_{u2} will catch the shock wave W_{u1} and the congested queue will have been dissipated. At this point, the final clearing wave W_{d3} forms and begins to move downstream. This wave defines the boundary between the high density capacity flow region and normal traffic flow. The speed of the wave is

$$W_{d3} = \frac{q_c - q_n}{k_c - k_n} \tag{9}$$

where W_{d3} is the speed of the last clearing wave and (q_c, k_c) and (q_n, k_n) define the volume and density in the capacity flow and normal regions, respectively, shown in Figures 3 and 4.

<u>Computing Shock Waves from Speed</u>

Freeway surveillance of incidents on the Gulf Freeway in Houston has indicated that a very useful and reliable method for readily detecting the occurrence of an incident on the freeway is to measure the change that occurs in the stream speed (or occupancy) in the queueing area immediately upstream of the scene of the incident. This suggests that it would be desirable if the entire freeway traffic flow existing during incident conditions (in essence, a mathematical description of Figure 4) could be related to the normal speed u_n existing before the incident occurred and the average speed within the congested queue, u_q. The average speed in the queue could be determined from the incident bottleneck capacity q_q using equation 3.

Figure 4 indicates that a description of freeway traffic conditions during an incident depends heavily on knowing the speeds and locations of the various waves in time and space and on knowing the duration of the incident. The following development is directed toward relating the previously discussed wave speeds to the normal traffic speed u_n and the queue speed u_q.

The two wave speeds W_{u1} and W_{d1} are of primary interest while the incident forms a bottleneck on the freeway. Note that the shock wave W_{u1} in equation 5 can be written as a function of only the normal traffic speed u_n and the speed u_q in the congested queue since $q = f(u)$ from equation 3 and $k = f(u)$ from equation 2. Since the speed of the shock wave is

$$W_{u1} = \frac{q_q - q_n}{k_q - k_n}$$

from equation 5 and substituting for $k = f(u)$ and $q = f(u)$ from equations 2 and 3 yield

$$W_{u1} = \frac{k_j u_q - \frac{k_j}{u_f} u_q^{\,2} - k_j u_n + \frac{k_j}{u_n} u_n^{\,2}}{k_j - \frac{k_j}{u_f} u_q - k_j + \frac{k_j}{u_f} u_n} \tag{10}$$

Subtracting the k_j's and rearranging yields

$$W_{u1} = \frac{k_j \ (u_q - u_n) - \frac{k_j}{u_f} (u_q^{\,2} - u_n^{\,2})}{- \frac{k_j}{u_n} (u_q - u_n)} \tag{11}$$

Dividing by $-k_j/u_f$ and by $(u_q - u_n)$ leaves

$$W_{u1} = -u_f + u_n + u_q \tag{12}$$

where W_{u1} is the speed of the shock wave, u_f is the free speed, and u_n and u_q are the normal and queue speeds, respectively. For the Greenshields linear speed-density model being used, the speed-volume curve of Figure 1-b is symmetrical about the speed at capacity. Thus, the sum of $u_n + u_q$ will be less than u_f as long as the bottleneck capacity flow q_q is less than the normal flow q_n that existed before the incident occurred.

The speed of the clearing metered wave W_{d1}, progressing downstream from the scene of the incident, can be developed in a similar manner since

$$W_{d1} = \frac{q_q - q_n}{k_m - k_n}$$

from equation 6. However, k_m must first be related to traffic conditions existing within the queueing section. Referring to Figure 3 and using equation 4 which related $q = f(k)$, it follows that $k_m = f(q)$ is

$$k_m = \frac{k_j}{2} - \sqrt{\frac{k_j^2}{4} - \frac{k_j}{u_f} q_q} \qquad (13)$$

Substituting $q_q = f(u_q)$ from equation 3 into equation 13 yields

$$k_m = \frac{k_j}{2} - \sqrt{\frac{k_j}{4} - \frac{k_j}{u_f} (k_j u_q - \frac{k_j}{u_f} u_q^2)} \qquad (14)$$

which reduces to

$$k_m = \frac{k_j}{u_f} u_q \qquad (15)$$

Returning to the equation for the metered wave speed of equation 6,

$$W_{d1} = \frac{q_q - q_n}{k_m - k_n}$$

the results of equation 15 are then substituted for k_m which yields

$$W_{d1} = \frac{q_q - q_n}{\frac{k_j}{u_f} u_q - k_n} \tag{16}$$

Next, the volume and density relationships as a function of speed, equation 2 and 3, are then substituted into equation 16, yielding

$$W_{d1} = \frac{k_j u_q - \frac{k_j}{u_f} u_q^{\;2} - k_j u_n + \frac{k_j}{u_f} u_n^{\;2}}{\frac{k_j}{u_f} u_q - k_j + \frac{k_j}{u_n} u_n} \tag{17}$$

or

$$W_{d1} = \frac{k_j (u_q - u_n) - \frac{k_j}{u_f} (u_q^{\;2} - u_n^{\;2})}{\frac{k_j}{u_f} (u_q + u_n) - k_j} \tag{18}$$

Dividing by u_f/k_j yields

$$W_{d1} = \frac{u_f (u_q - u_n) - (u_q - u_n)\ (u_q + u_n)}{u_q + u_n - u_f} \tag{19}$$

Factoring $-(u_q - u_n)$ results in

$$W_{d1} = \frac{-(u_q - u_n)\ (u_q + u_n - u_f)}{u_q + u_n - u_f} \tag{20}$$

and dividing out $(u_q + u_n - u_f)$ yields

$$W_{d1} = u_n - u_q \tag{21}$$

where W_{d1} is the clearing metered wave speed, u_n is the normal speed on the freeway before the incident, and u_q is the speed in the congested queue.

As has been noted in Figure 4, when the bottleneck incident is removed, three additional waves are generated. The equations for computing these waves have also been presented. The procedures used to relate the wave speeds to the normal speed u_n and the speed in the congested queue u_q follow the two previous examples. Hence, only the result of these three analyses will be presented.

$$W_{u2} = -\frac{u_f}{2} + u_q \tag{22}$$

$$W_{d2} = \frac{u_f}{2} - u_q \tag{23}$$

$$W_{d3} = -\frac{u_f}{2} + u_n \tag{24}$$

<u>Discussion of Model</u>

The results of the previous equations are summarized in Figure 4. The bottleneck incident occurs at point A in time and space and it lasts until the time of point B is reached. The maximum queue backup along the freeway from the location of the incident is noted as point C in Figure 4.

Comparisons of different wave speeds are made in the interest of providing additional insight and information of the models description of the freeway's operation during the incident. Since it is proposed that W_{u2} must catch the initial shock wave W_{u1}, the difference between them yields the rate of queue dissipation, or

$$W_{u2} - W_{u1} = \frac{u_f}{2} - u_n \qquad (25)$$

This difference will be negative as expected since the normal speed u_n is greater than the speed at normal capacity flow $u_f/2$ using Greenshields linear model of traffic flow. The expected negative difference also follows from the initial assumption that the normal flow was stable before the incident occurred with operating speeds above the speed at capacity (See Figure 1). Equation 25 confirms the expectation that the lighter the normal traffic flow before the incident (a larger u_n) the quicker the queue is dissipated.

For the three waves traveling downstream, the differences indicate each subsequent wave is slower than the previous one. This suggests that these waves never intersect downstream of the incident as if all three waves were rays emanating from a common point source. These results are based on the differences between

$$W_{d1} - W_{d2} = u_n - \frac{u_f}{2} \qquad (26)$$

and

$$W_{d2} - W_{d3} = u_f - u_n - u_q \qquad (27)$$

Both of these differences are positive indicating W_{d1} is faster than W_{d2} which in turn is faster than W_{d3}, the third and final wave traveling downstream. These results are reflected in the respective slopes of the waves shown in Figure 3.

PREDICTION OF FREEWAY TRAVEL TIMES

The procedure for computing the travel times of vehicles on the freeway during incident conditions requires a knowledge of freeway traffic speeds as a function of time and distance. Figure 4 has been shown to define the time and space locations of the four different freeway traffic flow conditions that exist during incident conditions. The average volumes and densities existing within each of these flow regions has been noted in Figure 3. Thus, the average traffic speed within each of the flow regions can be determined using equation 1. The traffic speed within each region and two examples of vehicles traveling through a congested freeway section during an incident are presented in Figure 5. Again, all speeds are being computed from only two traffic variables, the normal speed u_n and the speed within the congested queue u_q. Recall that u_f is the free speed parameter in Greenshields' linear speed-density model.

The procedure for computing the travel times of two vehicles will be illustrated. As shown as point A in Figure 5, one vehicle is assumed to be at an entrance ramp at t_o, the time the incident occurs. This vehicle would travel at a speed u_n until it intercepts the shock wave backing up the freeway at B. The speed of the vehicle would then drop considerably

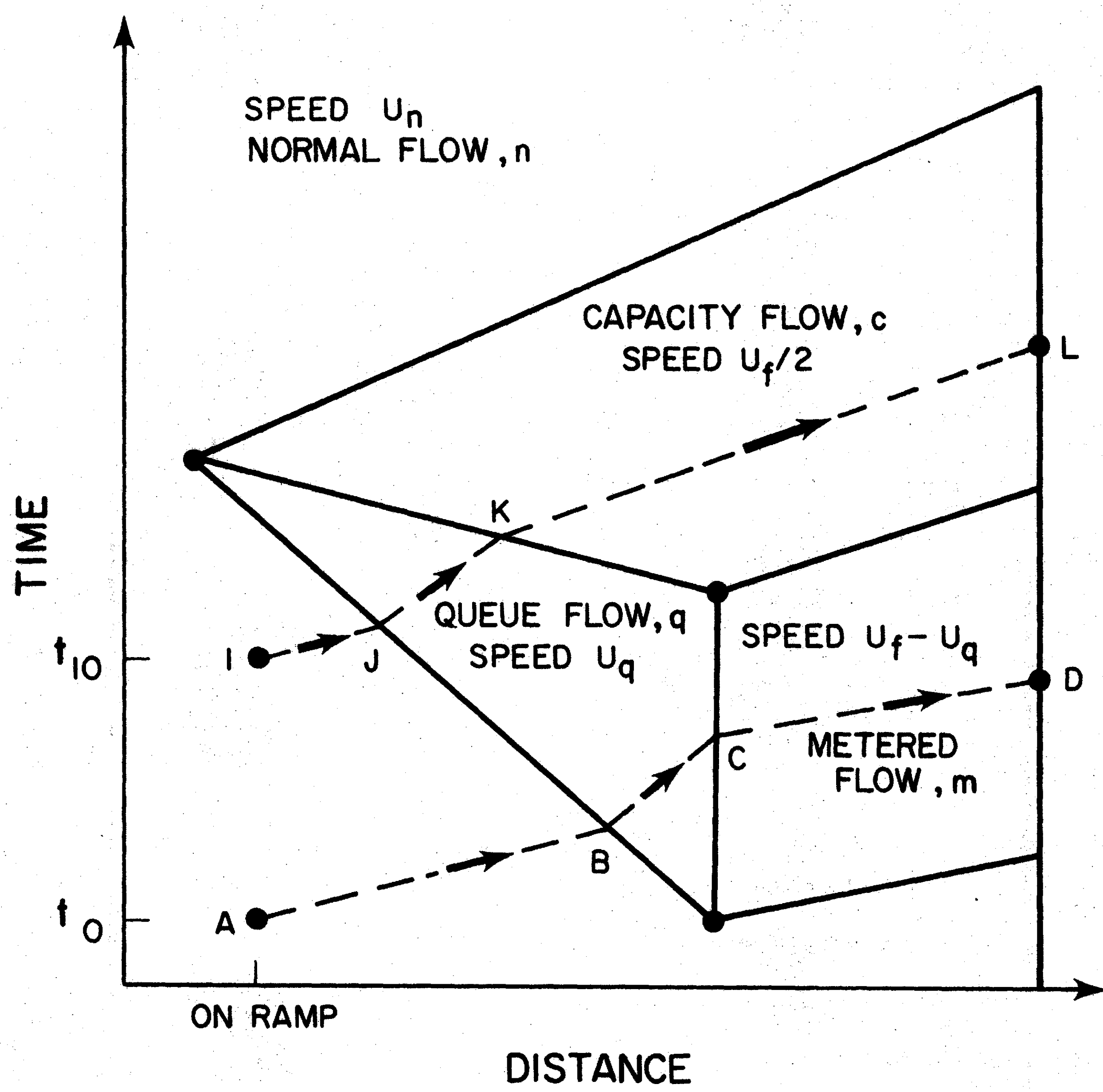

Figure 5 - Method of Predicting Travel Times of Vehicles Traveling Through Incident Conditions

to u_q while the vehicle travels through the congested queue. When it passes the incident location at C, the vehicle then enters the high speed metered region, having a speed $u_n = u_f - u_q$. The vehicle is assumed to leave the freeway system at D. The travel time for this vehicle would be $t_D - t_o$.

One feature of the travel time model is that it permits an immediate prediction, as soon as the incident is detected, of the travel times of vehicles that may enter the freeway some time after the incident occurs. Assume that a vehicle were to enter the freeway at the on-ramp, point I in Figure 5, ten minutes after an incident occurred. Entering the freeway, the vehicle then intercepts the shock wave at J and remains in the queue until the capacity flow wave at K is reached. The vehicle then remains in the capacity flow region, leaving the system at L. The travel time on the freeway from point I to L would be $T_L - T_{10}$ minutes.

The time-distance path that a vehicle would trace along Figure 5, e.g., path IJKL, is not known initially for an incident and must be computed in a trial and error manner. A computer program, which requires only a few seconds to execute, was written to compute these travel times. The listing of the travel time computer program and an output of travel times computed for an incident are included in Appendix A.

A travel time solution will be presented for a typical lane blockage incident that occurred on the inbound Gulf Freeway in Houston. A vehicle stalled on the median lane at 8:16 a.m., reducing the capacity by about one-half, and was removed six minutes later at 8:22, is shown in Figure 6. This figure also shows the predicted operating speeds, wave speeds and average traffic conditions during incident conditions. The incident

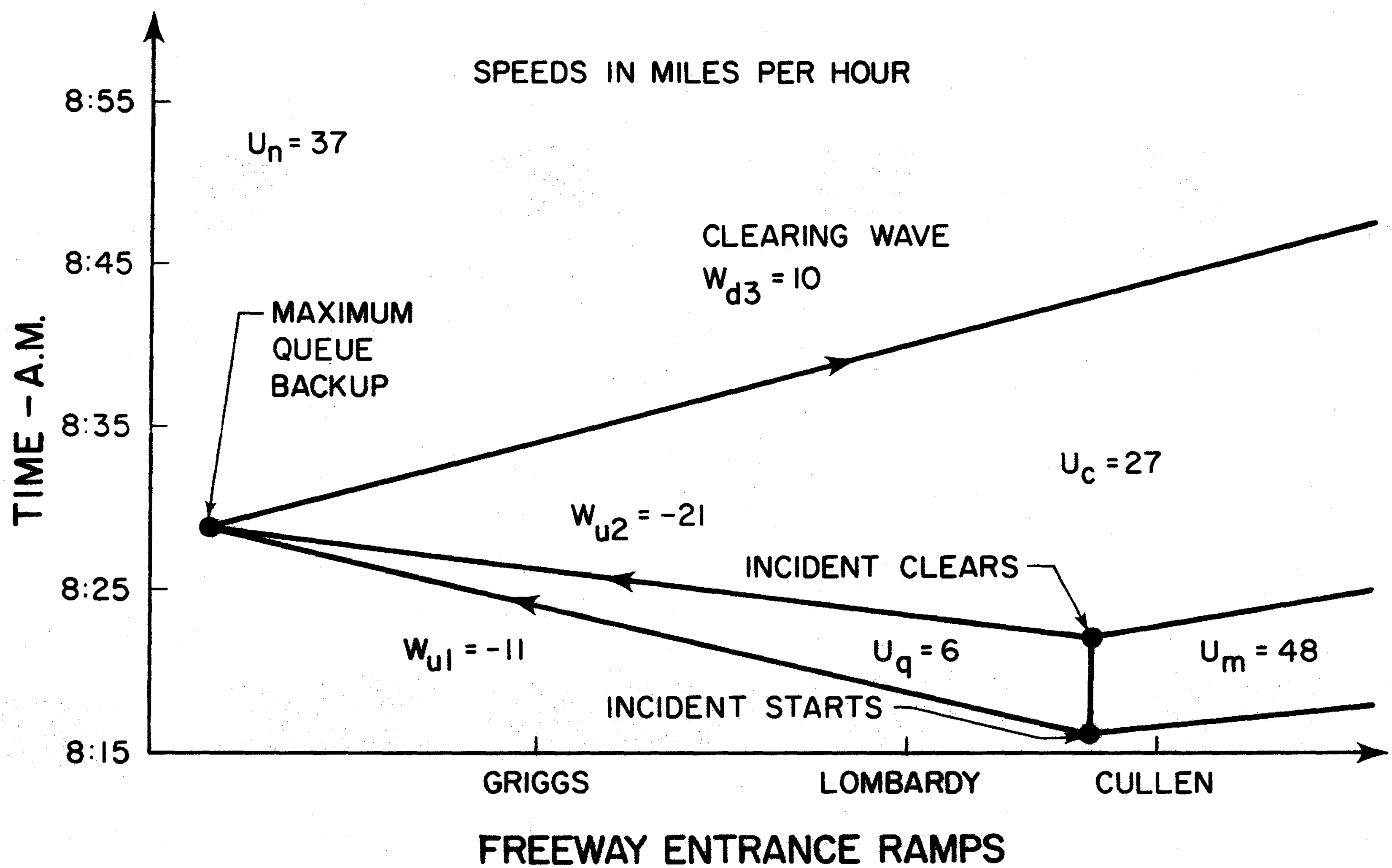

Figure 6 - Time-Space Diagram of Traffic Conditions During Incident

generated a shock wave having a speed of 11 miles per hour. It moved upstream for 13 minutes, until 8:29, resulting in a queue backup of about two and one-half miles. The shock wave was predicted to arrive at the Griggs ramp at 8:24 and was observed to arrive at 8:25 a.m.

Figure 7 shows the predicted travel time from any freeway location shown to the end of the system if the vehicle were to begin its trip at the time shown. The predicted travel times at 8:16, the time the incident occurred, are higher than the travel times expected just before the incident occurred. Note that the predicted travel times at the Griggs and Lombardy ramps located upstream of the incident increase for about ten minutes, four minutes after the blockage was removed.

Feasibility Study

The method presented for predicting travel times requires estimates for several variables and parameters. The location, duration, and severity of the incident must be established in addition to the normal average operating speed, speed in queue, and free speed. During real-time operations, all of these would have to be estimated within a short period of time based on real-time traffic data. The accuracy of these estimates would directly affect the accuracy of the travel time prediction model. Based on the literature available and freeway operational experience, it would appear that an accurate prediction of incident duration would be the most difficult variable to determine (2). Research is currently being conducted in this area to develop the necessary detection and estimation techniques.

An initial feasibility study was conducted, however, to determine the

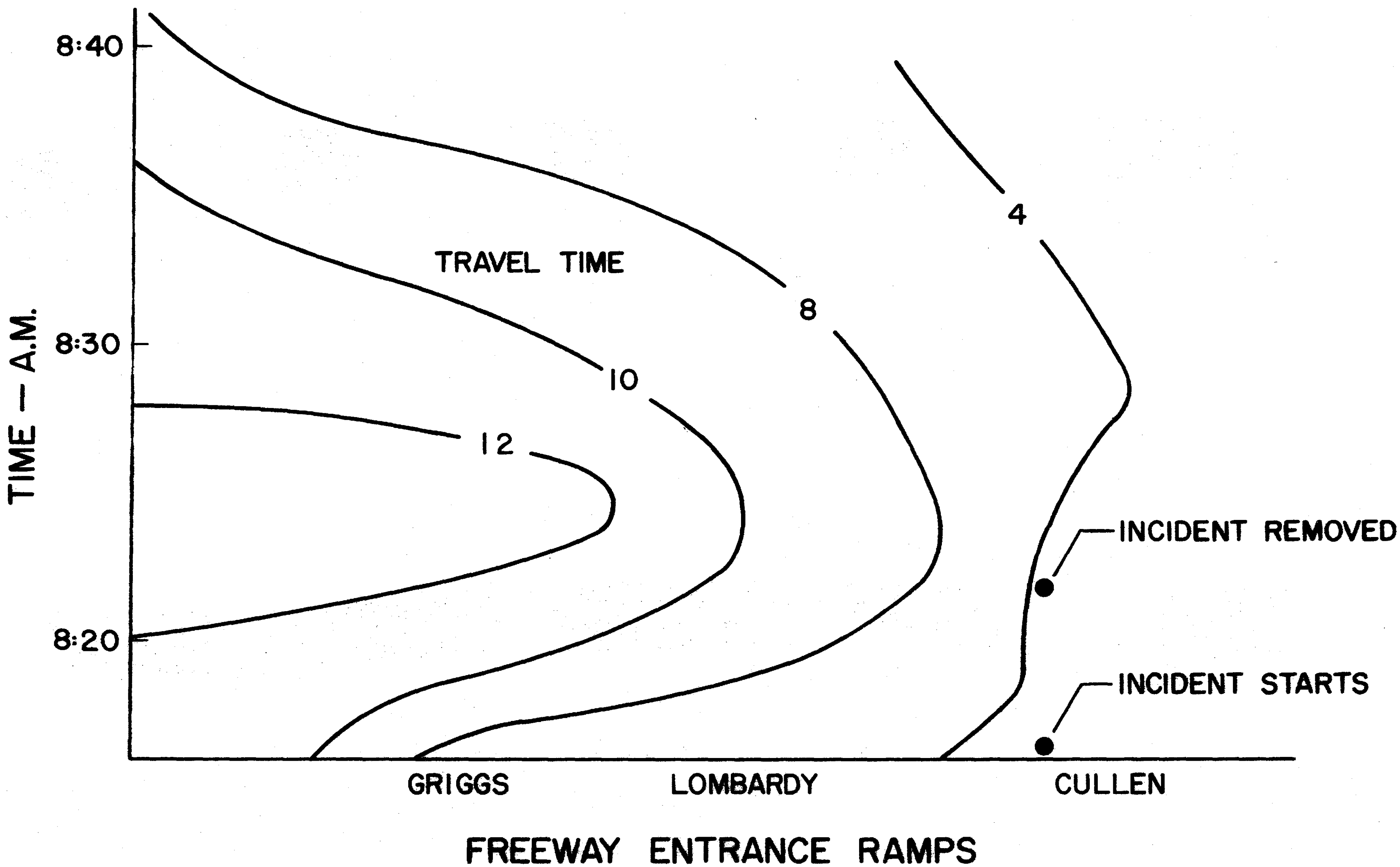

Figure 7 - Predicted Travel Times to End of Freeway System From Entrance Ramps

accuracy of the method in predicting travel times if all the necessary variables and parameters were accurately determined. One off-peak and three peak period incidents that occurred on the Gulf Freeway in Houston were evaluated. Freeway traffic flow was normal and not congested before the lane blockages occurred. The incident data were accurately recorded from television surveillance available in the freeway surveillance center, together with the resulting freeway traffic flow data available from computer printout. Ten auto travel times were manually recorded from the television surveillance for each incident. All travel time computations were made at a later date. Since each incident occurred at a different location on the freeway, the free speed, U_f, used in the method was adjusted slightly to provide the best possible fit of the recorded data.

Figure 8 shows the cumulative percentage of the relative percent error between the 40 samples of the auto travel times taken during the four incidents and the computer travel times. Two-thirds of the observed travel times were within ten percent of the computer travel times, a level felt satisfactory for consideration as reliable information. Most of the larger errors arose when travel times were being predicted for times 10-20 minutes after the incidents occurred. Again, based on the available data, this is the highest accuracy that could be expected to be obtained with accurate estimates of the incident variables under ideal conditions.

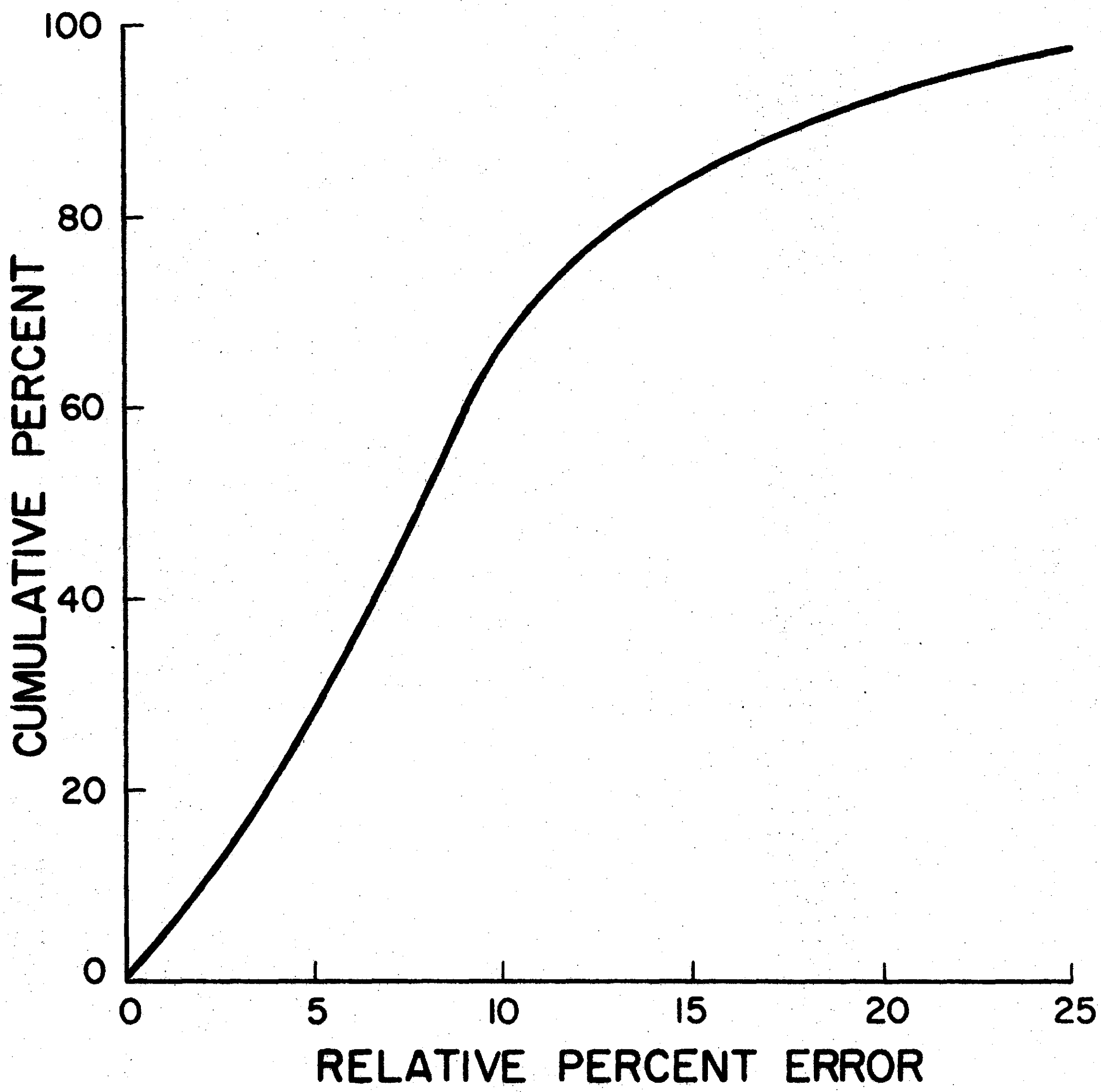

Figure 8 - Accuracy of Calibrated Travel Time Model When All Conditions Are Known

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APPENDIX A

```
C
C*******************************************************************************
C
C***** PROGRAM TO PREDICT TRAVEL TIMES FOR INCIDENT CONDITIONS
C**     WRITTEN BY MESSER ---------- DOCUMENTED AND MODIFIED BY FRIEBELE
C
C*******************************************************************************
C
      COMMON UW(6),PT(9),PD(9),SIGNL(10),UT(4),TT(10,10)
C
C***** UT(1)= SPEED OF TRAFFIC BEFORE INCIDENT  (FPS)
C
C***** ZLOC = LOCATION OF INCIDENT W/ RESPECT TO CULLEN STA. 500. -- TO BE INPUT
C
C***** TD = DURATION OF INCIDENT IN SECONDS/100   **** TO BE INPUT**********
C
C     RATIO = INCIDENT CAPACITY/NORMAL FREEWAY CAPACITY
C
      DLL=0.
      DUL=500.
      READ(2,2000)TB,TD,ZLOC,UT(1),RATIO
 2000 FORMAT(5F10.2)
      CALL CHECK (DUL,ZLOC,TB,TD,F)
      CALL COORD (ZLOC,TB,TD,DLL,DUL,F,RATIO)
      CALL SOLVE (ZLOC,DLL,DUL,TB)
      CALL EXIT
      END
C
      SUBROUTINE CHECK (DUL,ZLOC,TB,TD,F)
C
C***** THIS SUBROUTINE IS TO COMPILE INPUT INFORMATION
C
      COMMON UW(6),PT(9),PD(9),SIGNL(10),UT(4),TT(10,10)
C
      SBEF=UT(1)/1.47
      ATD=TD*100
      BTD=ATD/60.
      IZLOC=ZLOC
      WRITE(3,990)SBEF,UT(1),          IZLOC,BTD,ATD
  990 FORMAT('1',//10X,'SPEED BEFORE INCIDENT =',F5.1,' MPH  (',F5.1,
     *' FPS)',/10X,'LOCATION OF INCIDENT IS STA.',I5,'+00',/10X,
     *'DURATION OF INCIDENT IS',F4.1,' MIN.  (',F6.0,' SEC.)',//)
C
C
C***** SIGNL(I)= DETECTION LOCATIONS  (500.00= CULLEN)
C
      SIGNL(1)=280.00
      SIGNL(2)=300.00
      SIGNL(3)=305.98
      SIGNL(4)=331.66
      SIGNL(5)=353.52
      SIGNL(6)=373.88
      SIGNL(7)=393.04
      SIGNL(8)=412.63
      SIGNL(9)=441.47
      SIGNL(10)=470.00
      RETURN
      END
C
      SUBROUTINE COORD (ZLOC,TB,TD,DLL,DUL,F,RATIO)
C
C***** THIS SUBROUTINE COMPILES ALL OUTPUT INFOR
C
      DIMENSION MM(10)
      COMMON UW(6),PT(9),PD(9),SIGNL(10),UT(4),TT(10,10)
      UF=82.
      UF1=UF/1.47
      WRITE(3,991)UF1,UF
  991 FORMAT(/10X,'FREE SPEED =',F5.1,' MPH   (',F5.1,' FPS)',/)
      IF(UT(1)-UF) 70,70,71
C
C***** UT(I)= QUEUE SPEEDS
C
   71 UT(1)=UF
C
   70 UC=UF/2.
      NUM = 0
      X1=.5
      A=1.0-RATIO
  161 X2 = X1+A/X1
      X2 = X2/2.
      NUM=NUM+1
      IF (NUM-4) 162,163,163
  162 X1 = X2
      GO TO 161
  163 X2 = 1.0-X2
      UT(2) = 0.5*UF*X2
      IF(UC-UT(2)-5.) 31,30,30
C
C
C***** UW(I)= WAVE  SPEEDS
C
   31 UT(2)=UC-5.
   30 IF(UT(1)-UC-3.) 32,33,33
   32 UT(1)=UC+3.
   33 IF(UF-UT(1)-UT(2)) 35,35,34
   35 UT(1)=UF-UT(2)-1.
   34 UT(4)=UC
      UT(3)=UF-UT(2)
      UW(1)=-UF+UT(1)+UT(2)
      UW(2)=-UC+UT(2)
      UW(3)=UT(1)-UC
      UW(4)=UT(1)-UT(2)
      UW(5)=UC-UT(2)
C
C
C***** PT(I) AND PD(I) = SPEED INTERCEPTS
C
      PT(1)=TB
      PD(1)=ZLOC
```

```
      PT(2)=TB+TD
      PD(2)=ZLOC
      SUM=-PD(1)+PD(2)+UW(1)*PT(1)-UW(2)*PT(2)
      DIF=UW(1)-UW(2)
      IF(DIF) 5,6,5
    6 DIF=1.
    5 T=SUM/DIF
      D=PD(1)+UW(1)*(T-PT(1))
      IF(D-DLL) 11,12,12
   11 PT(3)=PT(1)+(DLL-PD(1))/UW(1)
      PD(3)=DLL
      PT(4)=PT(2)+(DLL-PD(2))/UW(2)
      PD(4)=DLL
      PT(5)=T+(DLL-D)/UW(3)
      PD(5)=DLL
      PT(6)=PT(5)+(DUL-PD(5))/UW(3)
      PD(6)=DUL
      GO TO 13
   12 PT(3)=T
      PD(3)=D
      PT(4)=T
      PD(4)=D
      PT(5)=T
      PD(5)=D
      PT(6)=T+(DUL-D)/UW(3)
      PD(6)=DUL
   13 PT(7)=PT(1)+(DUL-PD(1))/UW(4)
      PD(7)=DUL
      PT(8)=PT(2)+(DUL-PD(2))/UW(5)
      PD(8)=DUL
      PT(9)=32000.
      PD(9)=DUL
      UW(6)=0.
C
      WRITE(3,54)
   54 FORMAT(/16X,'OPERATING SPEEDS AND WAVE SPEEDS',/)
      WRITE(3,55)(UT(I),I=1,4)
      WRITE(3,55)(UW(I),I=1,6)
      WRITE(3,56)
   56 FORMAT(/35X,'SHOCKWAVE INTERCEPTS',/)
      WRITE(3,55)(PT(I),I=1,9)
      WRITE(3,55)(PD(I),I=1,9)
      MM(1)=0
      DO 88 I=2,10
   88 MM(I)=MM(I-1)+2
      WRITE(3,57)(MM(I),I=1,10)
   57 FORMAT(//43X,'T R A V E L   T I M E S',//,'  FROM TIME*********',
     *10(8X,I2),/)
   55 FORMAT(10F10.1)
      RETURN
      END
C
      SUBROUTINE SOLVE ( ZLOC,DLL,DUL,TB)
      COMMON UW(6),PT(9),PD(9),SIGNL(10),UT(4),TT(10,10)
      TX=PT(5)+(ZLOC-PD(5))/UW(3)
      DO 10 L=1,10
      TIME=TB-2.
      DO 12 M=1,10
      TIME=TIME+2.
      TV=TIME
      DVEH=SIGNL(L)
      TT(L,M)=0.
      NU=1
      IF(ZLOC-SIGNL(L)) 60,60,13
   13 JUT=3
      JLT=1
      JW=1
      CALL EQCK (NU,DVEH,TV,JUT,JLT,JW,JLT,ITEST,T,D,ADDT)
      IF(ITEST) 70,40,15
   40 NU=1
      JUT=9
      JLT=6
      JW=6
      CALL EQCK (NU,DVEH,TV,JUT,JLT,JW,JLT,ITEST,T,D,ADDT)
      IF(ITEST) 41,42,41
   41 TT(L,M)=TT(L,M)+(DUL-DVEH)/UT(NU)
      GO TO 12
   42 JUT=6
      JLT=5
      JW=3
      CALL EQCK (NU,DVEH,TV,JUT,JLT,JW,JLT,ITEST,T,D,ADDT)
      IF(ITEST) 44,41,43
   43 TT(L,M)=TT(L,M)+ADDT
      DVEH=D
      TV=T
   44 NU=4
      JUT=6
      JLT=8
      JW=6
      CALL EQCK (NU,DVEH,TV,JUT,JLT,JW,JLT,ITEST,T,D,ADDT)
      IF(ITEST) 41,46,41
   46 JUT=8
      JLT=2
      JW=5
      CALL EQCK (NU,DVEH,TV,JUT,JLT,JW,JLT,ITEST,T,D,ADDT)
      IF(ITEST) 48,41,47
   47 TT(L,M)=TT(L,M)+ADDT
      DVEH=D
      TV=T
   48 NU=3
      JUT=8
      JLT=7
      JW=6
```

```
      CALL EQCK (NU,DVEH,TV,JUT,JLT,JW,JLT,ITEST,T,D,ADDT)
      IF(ITEST) 41,50,41
   50 JUT=7
      JLT=1
      JW=4
      CALL EQCK (NU,DVEH,TV,JUT,JLT,JW,JLT,ITEST,T,D,ADDT)
      IF(ITEST) 52,52,51
   51 TT(L,M)=TT(L,M)+ADDT
      DVEH=D
      TV=T
   52 NU=1
      GO TO 41
   15 TT(L,M)=TT(L,M)+ADDT
      DVEH=D
      TV=T
   72 NU=2
      JUT=2
      JLT=1
      JW=6
      CALL EQCK (NU,DVEH,TV,JUT,JLT,JW,JLT,ITEST,T,D,ADDT)
      IF(ITEST) 41,18,17
   17 TT(L,M)=TT(L,M)+ADDT
      DVEH=D
      TV=T
      GO TO 48
   18 JUT=4
      JLT=2
      JW=2
      CALL EQCK (NU,DVEH,TV,JUT,JLT,JW,JLT,ITEST,T,D,ADDT)
      IF(ITEST) 44,41,19
   19 TT(L,M)=TT(L,M)+ADDT
      DVEH=D
      TV=T
      GO TO 44
   70 IF(TV-PT(2)) 72,72,71
   71 IF(TV-PT(5)) 73,40,40
   73 NU=2
      GO TO 18
   60 IF(TV-PT(1)) 41,41,61
   61 IF(TV-PT(7)) 62,62,63
   62 NU=3
      GO TO 50
   63 IF(TV-PT(2)) 48,48,64
   64 IF(TV-PT(8)) 65,65,66
   65 NU=4
      GO TO 46
   66 IF(TV-TX) 44,44,67
   67 IF(TV-PT(6))  42,42,40
   12 CONTINUE
   10 CONTINUE
      DO 82 L=1,10
      DO 83 M=1,10
C
C
C***** TT(L,M)= TRAVEL TIMES
C
   83 TT(L,M)=TT(L,M)*100
   82 CONTINUE
C
      DO 80 L=1,10
   80 WRITE(3,81) SIGNL(L),(TT(L,M),M=1,10)
   81 FORMAT(/,' STA.',F7.2,8X,10(F10.0))
      RETURN
      END
C
      SUBROUTINE EQCK (NU,DVEH,TV,JUT,JLT,JW,JB,ITEST,T,D,ADDT)
C
C***** THIS SUBROUTINE CHECKS BOUNDARY CONDITIONS FO OUTPUT INFOR
C
      COMMON UW(6),PT(9),PD(9),SIGNL(10),UT(4),TT(10,10)
      SUM=-DVEH+PD(JB)+UT(NU)*TV-UW(JW)*PT(JB)
      DIF=UT(NU)-UW(JW)
      IF(DIF) 5,6,5
    6 DIF=1.
    5 T=SUM/DIF
      ADDT=T-TV
      IF(ADDT) 7,7,8
    7 ITEST=-1
      GO TO 9
    8 D=DVEH+UT(NU)*ADDT
      IF(T-PT(JLT)) 2,1,1
    1 IF(PT(JUT)-T) 2,3,3
    2 ITEST=0
      GO TO 9
    3 ITEST=1
    9 CONTINUE
      RETURN
      END
```

SPEED BEFORE INCIDENT = 36.0 MPH (53.0 FPS)
LOCATION OF INCIDENT IS STA. 412+00
DURATION OF INCIDENT IS 5.9 MIN. (360. SEC.)

FREE SPEED = 55.7 MPH (82.0 FPS)

OPERATING SPEEDS AND WAVE SPEEDS

53.0	13.8	68.1	41.0		
-15.1	-27.1	12.0	39.1	27.1	0.0

SHOCKWAVE INTERCEPTS

0.0	3.6	8.1	8.1	8.1	25.8	2.2	6.8	32000.0
412.0	412.0	288.0	288.0	288.0	500.0	500.0	500.0	500.0

T R A V E L T I M E S

FROM TIME********	0	2	4	6	8	10	12	14	16	18
STA. 280.00	535.	583.	566.	549.	532.	515.	497.	480.	463.	446.
STA. 300.00	474.	549.	531.	514.	487.	480.	463.	446.	429.	412.
STA. 305.98	455.	538.	521.	504.	473.	470.	453.	436.	418.	401.
STA. 331.66	376.	494.	477.	434.	410.	410.	408.	391.	374.	357.
STA. 353.52	309.	435.	434.	357.	357.	357.	357.	353.	336.	319.
STA. 373.88	246.	373.	351.	307.	307.	307.	307.	307.	301.	234.
STA. 393.04	201.	266.	266.	260.	260.	260.	260.	260.	260.	251.
STA. 412.63	164.	164.	157.	213.	213.	213.	213.	213.	213.	213.
STA. 441.47	110.	110.	85.	142.	142.	142.	142.	142.	142.	142.
STA. 470.00	56.	56.	43.	64.	73.	73.	73.	73.	73.	73.

// END 03 DEC 73 13.622 HRS