1 Traffic Engineering

PROBLEM 1

AASHTO's *A Policy on Geometric Design of Highways and Streets* (GDHS) specifies criteria for safe design speed on highway curves. Which of the following criteria normally apply to designing a safe curve?

I. curve radius
II. passenger comfort factor
III. sight distance
IV. shoulder width
V. speed-limit posting
VI. side friction factor
VII. superelevation rate
VIII. weather conditions

(A) I, II, III, and VII only
(B) I, III, VII, and VIII only
(C) I, II, III, VI, VII, and VIII only
(D) I, II, III, IV, V, and VIII only

Hint: Safe speed on a curve balances the forces needed to keep a vehicle on the roadway against the forces tending to push the vehicle off the roadway.

PROBLEM 2

A four-lane divided highway in a suburban area has the following characteristics.

lane width	10 ft
average grade	<2%
left clearance	4 ft
right clearance	2 ft
percentage of heavy vehicles	7%
access spacing	300 ft
design speed	60 mph
posted speed limit	55 mph
directional design hour volume	2540 vph
peak hour factor (PHF)	0.92

What is the level of service (LOS) of the highway?

(A) C
(B) D
(C) E
(D) F

Hint: Roadway configuration restrictions affect the free-flow speed (FFS). Design speed can be considered to be the base free-flow speed (BFFS) if there is no information to the contrary.

PROBLEM 3

A freeway in rolling terrain has the following characteristics.

commuter traffic volume (one way)	1970 vph
number of lanes (in each direction)	4
percentage of trucks	3%
percentage of buses	3%
percentage of RVs	1%
peak hour factor (PHF)	0.85

What is most nearly the peak hour flow rate?

(A) ~~477~~ pcphpl
(B) 580 pcphpl
(C) ~~623~~ pcphpl
(D) ~~661~~ pcphpl

Hint: The peak hour flow rate is the per-lane passenger-car equivalent of the hourly count of total vehicle flow. Use the formulas for equivalent passenger-car flow rates from the *Highway Capacity Manual* (HCM).

PROBLEM 4

A 10 mi section of freeway in rolling terrain within a metropolitan area of 325,000 population has the characteristics listed. Local commuters make up 90% of the traffic, and the remainder is a combination of through traffic and recreational traffic to a nearby national park.

free-flow speed (FFS)	60 mph (measured)
number of lanes (in each direction)	3
lane width	11 ft
right shoulder width	6 ft
percentage of trucks (TT)	3%
percentage of buses	2%
percentage of RVs	1%
number of interchanges/ramps	4
peak hour factor (PHF)	0.94 (measured)
driver population factor	1.00
hourly traffic volume (one way)	5700 vph
maximum grade	5% for 0.125 mi

What is the level of service (LOS) for this section?

(A) C

(B) D

(C) E

(D) F

Hint: Heavy vehicle factors for RVs are ~~not always~~ included with ~~trucks and~~ buses.

PROBLEM 5

The average number of cars passing a point is 2200 pcphpl. The cars travel at an average speed of 42 mph. If the average length of a car is 19 ft, the distance between the cars is most nearly

(A) 43 ft

(B) 82 ft

(C) 100 ft

(D) 130 ft

Hint: Look for a common element among the defining units, which are related to speed, time, distance, and the number of cars.

PROBLEM 6

Which of the following criteria are used to specify traffic density as determined by the *Highway Capacity Manual* (HCM)?

I. passenger-car equivalents

II. number of occupants in a vehicle

III. vehicle count per unit of time

IV. vehicle length

V. vehicle weight

VI. vehicle spacing

(A) I, IV, and VI only

(B) III, IV, and VI only

(C) III, V, and VI only

(D) I, II, IV, and VI only

Hint: Traffic density is expressed as the number of passenger cars per mile, per lane of roadway.

PROBLEM 7

The stopping distance for a car traveling at 50 mph is 461 ft, including the distance traveled during a 2.5 sec perception-reaction time. If a car traveling at 60 mph is to stop in the same distance with the same friction factor, what is most nearly the required perception-reaction time?

(A) 0 sec

(B) 0.1 sec

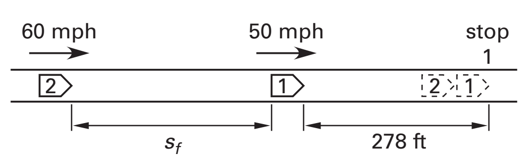
(C) 0.7 sec

(D) 2 sec

Hint: The stopping distance includes braking distance and perception-reaction distance.

PROBLEM 8

A car traveling at 50 mph is followed by a car traveling at 60 mph. The lead car suddenly brakes to a stop within a 278 ft distance. Both cars have the same stopping friction factor, and the driver of the following car has a perception-reaction time of 2.0 sec. The road is level.



In order to avoid a collision, the minimum distance between the two cars must be most nearly

(A) 86 ft

(B) 280 ft

(C) 300 ft

(D) 320 ft

Hint: The driver of the following car is aware that the lead car is stopping when the driver sees the brake lights turn on.

PROBLEM 9

Passenger stations are spaced 1 mi apart from each other on a rapid transit line. Trains accelerate out of a station at 5.5 ft/sec^2 and decelerate into a station at 4.4 ft/sec^2. Trains have a top speed of 80 mph. What is most nearly the average speed of a train between stations?

(A) 12 mph

(B) 40 mph

(C) 52 mph

(D) 54 mph

Hint: The trip stages between stations consist of accelerating, traveling at constant speed, and decelerating.

PROBLEM 10

A rapid transit line has trains scheduled to arrive at a station every 5 min. The trains each have 4 cars and can carry 220 passengers per car. The trains have a 1 min dwell time at each station. What is most nearly the capacity of the transit line?

(A) 880 passengers/hr

(B) 2640 passengers/hr

(C) 10,600 passengers/hr

(D) 13,200 passengers/hr

Hint: The arrival rate and departure rate are the same, regardless of the length of dwell time at the station.

PROBLEM 11

A high-speed train is to reach 150 mph between stations. Acceleration is limited to 0.18 g, and deceleration is limited to 0.12 g. What is most nearly the minimum spacing between stations?

(A) 0.80 mi

(B) 0.90 mi

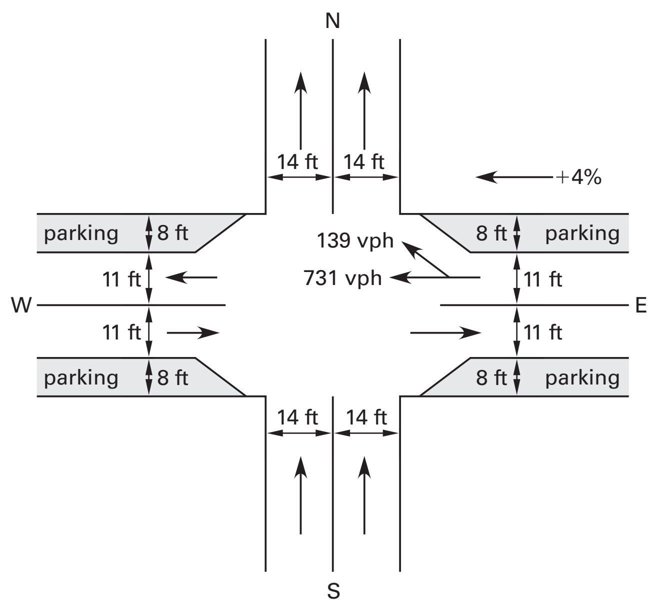
(C) 2.0 mi

(D) 6.5 mi

Hint: The train reaches 150 mph for an instant between stations.

PROBLEM 12

In the layout shown, trucks make up less than 1% of vehicles using approach E, and there are 20 parking maneuvers per hour. There are no buses or pedestrians. The location is near a central business district (CBD). The demand flow rate is 1900 pcphpl.



What is most nearly the saturation flow rate of approach E?

(A) 870 vph

(B) 1080 vph

(C) 1180 vph

(D) 1200 vph

Hint: The saturation flow rate modifies an ideal flow rate by approach condition adjustment factors.

PROBLEM 22

In the *Highway Capacity Manual* (HCM), the peak hour factor is determined by the peak

(A) 15 minutes of flow divided by the peak one-hour flow

(B) hour flow divided by the maximum rate of flow that occurs in the peak interval (usually five minutes)

(C) interval of flow (usually five minutes) divided by the peak one-hour flow

(D) hour flow divided by the peak 15-minute flow rate, expressed as an hourly rate

Hint: The traffic flow rate varies throughout a 1 hr period and usually requires more than 5 min to adjust to a new rate.

PROBLEM 23

When describing highway traffic flow, which of the following statements is NOT true?

(A) A highway with buses in the traffic stream has a higher capacity of persons per hour than does a highway with only automobiles in the traffic stream.

(B) A highway with trucks in the traffic stream has a lower capacity of persons per hour than does a highway with only automobiles in the traffic stream.

(C) ~~For a highway in mountainous terrain with long grades, the same passenger-car equivalent is used for trucks on upgrades as is used for trucks on downgrades.~~

(D) For a highway with ~~a large number of~~ trucks in the traffic stream (as compared to RVs), the RVs ~~can be~~ combined with the trucks when determining passenger-car equivalents of heavy vehicles.

Hint: Buses and passenger vans increase the person capacity of a highway, while trucks and RVs decrease the person capacity.

PROBLEM 24

Transit planners are attempting to determine expected transit usage from a community that produces 12,000 trips/day. The population density is 10,000 persons/mi^2, and there is an average of 0.80 autos/household. The following model for the urban travel factor (UTF) has been established for the community.

$$\text{UTF} = \left(\frac{1}{1000}\right)\left(\frac{\text{households}}{\text{auto}}\right)\left(\frac{\text{persons}}{\text{mi}^2}\right)$$

$$\%\ \text{transit usage} = \frac{\text{UTF}}{0.6}$$

Approximately how many residents can be expected to use transit?

(A) 1500 persons

(B) 1600 persons

(C) 2100 persons

(D) 2500 persons

Hint: Households per auto is the inverse of autos per household.

PROBLEM 25

7000 persons commute daily from a bedroom community to an employment center, with an average commute distance of 8 mi in one direction. The average speed of the commute is 20 mph. 25% of the commuters carpool, and the average carpool has 2.10 persons/veh. Fuel consumption is measured by

$$F = 0.0362\ \frac{\text{gal}}{\text{veh-mi}} + \frac{0.746\ \frac{\text{gal}}{\text{veh-hr}}}{\text{v}}$$

With a fuel heating value of 125,000 Btu/gal, what is the approximate daily round trip Btu consumption for this commute pattern?

(A) 7.2×10^3 Btu

(B) 4.5×10^8 Btu

(C) 8.9×10^8 Btu

(D) 10×10^8 Btu

Hint: A daily commute includes two trips.

PROBLEM 26

A transit terminal is being developed in an urban neighborhood. One of the goals of site impact analysis is to improve public safety. Which of the following would have the SMALLEST impact on this goal?

(A) improving personal security of urban travelers

(B) improving reliability of transit service

(C) providing adequate lighting levels throughout walkway and platform areas

(D) reducing noise and vibration impacts

Hint: Public safety involves minimizing anxiety about personal safety.

PROBLEM 27

A study zone contains 600 households, each averaging 3.5 persons and 2.2 autos. The modal split is 0.94/0.05 auto to transit, with 0.01 assigned to other modes. The following model has been determined to show the relationship for the number of trips per household.

$$T = 0.78 + 1.3P + 2.3A$$

T is the number of daily trips per household, P is the number of persons per household, and A is the number of autos per household. Approximately how many auto trips per day are generated by the study zone?

(A) 10 trips/day

(B) 5900 trips/day

(C) 6200 trips/day

(D) 98×10^3 trips/day

Hint: The number of auto trips is a portion of the total number of household trips.

PROBLEM 28

On a highway facility, how does the observed hourly vehicle volume differ from the design peak-period flow rate?

(A) The highest 15 min vehicle volume is divided by the highest 1 hr flow rate to obtain the peak-period flow rate.

(B) The observed hourly vehicle volume includes a mix of heavy vehicles, while the design peak-period flow rate has been adjusted for passenger-car equivalents of heavy vehicles, the peak hour factor, ~~the driver population,~~ and the number of lanes.

(C) The observed hourly vehicle volume is divided by the number of observation hours to obtain the peak-period flow rate.

(D) The observed vehicle volume is divided by the number of observation hours and the number of lanes over which the observation took place.

Hint: Various vehicle sizes must be converted to a common unit of vehicle measure.

PROBLEM 29

Each lane of a four-lane freeway has a directional capacity of 2100 vph. The normal directional flow is 3100 vph. An incident blocks one lane for 20 min and then is cleared to allow the full traffic capacity flow. Approximately how long does it take to dissipate the queue after the blockage has been cleared?

(A) 15 min

(B) 30 min

(C) 50 min

(D) 57 min

Hint: Departure from the blockage is at two rates, while arrival continues at the same rate as before the blockage.

PROBLEM 30

A car brakes suddenly and skids to a stop from 60 mph. The car initially skids 150 ft on pavement with a friction factor of 0.30. The skid continues onto wet grass on hard soil with a friction factor of 0.10. Both parts of the skid are on a 3% upgrade. Approximately how long is the skid on the grassy surface?

(A) 540 ft

(B) 750 ft

(C) 1300 ft

(D) 1600 ft

Hint: An upgrade decreases the length of skid from a given speed.

PROBLEM 31

A car traveling at 70 mph on a 5% downgrade skids 350 ft before striking a retaining wall head-on. The coefficient of friction between the tires and the road is 0.30. What was the approximate speed of the car at impact?

(A) 35 mph

(B) 42 mph

(C) 48 mph

(D) 51 mph

Hint: A downgrade increases the required stopping distance for a given speed.

PROBLEM 32

A car traveling on a 3% upgrade at 60 mph in a construction area skids into a stack of concrete barriers. Skid marks leading to the crash measure 150 ft long. The pavement has a friction factor of 0.30.

SOLUTION 1

Curve radius and speed together determine the radial force needed to hold a vehicle on the curve.

Passenger comfort is important so that the driver can maintain control of the vehicle and so that passengers are not subject to unnecessary disorientation or the feeling of danger.

Sight distance must be adequate so that the driver can avoid hazards.

Side friction must be considered to ensure that the driver can steer the vehicle around the curve.

The superelevation rate makes use of the force of gravity to help keep the vehicle from sliding off the outside of the curve.

Weather conditions such as rain or snow reduce the tire friction available for holding the vehicle on the curve.

The answer is (C).

Why Other Options Are Wrong

(A) This answer omits weather conditions, such as rain or snow, which reduce the available tire friction needed to hold the vehicle on a curve. In regions that are often wet or icy, the design speed is set lower than in regions that have normally dry conditions.

(B) This answer omits criterion II, passenger comfort, which is necessary so that the driver can maintain control of the vehicle and so that passengers are not subject to disorientation or a feeling of danger.

(D) The posted speed limit, criterion V, is subject to local regulatory conditions and does not determine design speed of an existing highway. Posted speed can, however, be used to set the minimum design speed of a new or reconstructed highway.

SOLUTION 2

A criterion of LOS is the maximum service flow rate per hour per lane of highway. The service flow rate is the passenger-car equivalent flow at a free-flow speed. The flow rate must be determined in passenger-car equivalents of the total traffic vehicle mix using *Highway Capacity Manual* (HCM) Eq. 12-9.

$$v_p = \frac{V}{PHF \times N \times f_{\text{HV}}}$$

Determine the heavy vehicle factor using HCM Eq. 12-10. HCM Exh. 12-25 shows passenger car equivalents for vehicles in general terrain segments. Heavy vehicles include all single-unit trucks (SUTs), trailer trucks (TTs), and recreational vehicles (RVs). P_T is the decimal proportion of the total of these three groups.

$$\begin{aligned} f_{\text{HV}} &= \frac{1}{1 + P_T(E_T - 1)} \\ &= \frac{1}{1 + (0.07)(2.0 - 1)} \\ &= 0.935 \end{aligned}$$

Determine the service flow rate. f_p is assumed to be 1.0 and is not included in the volume calculation.

$$\begin{aligned} v_p &= \frac{2540 \ \frac{\text{veh}}{\text{hr}}}{(0.92)(2 \text{ lanes})(0.935)} \\ &= 1476 \text{ pcphpl} \end{aligned}$$

Determine the adjusted free-flow speed using HCM Eq. 12-3, based on the characteristics shown.

$$\text{FFS} = \text{BFFS} - f_{\text{LW}} - f_{\text{LC}} - f_M - f_A$$

The design speed is 60 mph, and the posted speed limit is 55 mph. Therefore, the base free-flow speed (BFFS) is 60 mph (HCM Ch. 12).

The lane width is 10 ft. Therefore, from HCM Exh. 12-20, $f_{\text{LW}} = 6.6$ mph.

The total lateral clearance (TLC) is 6 ft. Therefore, from HCM Exh. 12-22, $f_{\text{LC}} = 1.3$ mph.

The median is divided. Therefore, from HCM Exh. 12-23, $f_M = 0.00$ mph.

There are access points every 300 ft, which equates to 17.6 access points per mile. Considering a speed reduction of 0.25 mph, or interpolating from HCM Exh. 12-24, $f_A = 4.4$ mph.

$$\begin{aligned} \text{FFS} &= 60 \ \frac{\text{mi}}{\text{hr}} - 6.6 \ \frac{\text{mi}}{\text{hr}} - 1.3 \ \frac{\text{mi}}{\text{hr}} \\ &\quad - 0 \ \frac{\text{mi}}{\text{hr}} - 4.4 \ \frac{\text{mi}}{\text{hr}} \\ &= 47.7 \text{ mph} \end{aligned}$$

The density of flow is found using HCM Eq. 12-11.

$$D = \frac{v_p}{S} = \frac{1476 \ \frac{\text{pc}}{\text{hr-ln}}}{47.7 \ \frac{\text{mi}}{\text{hr}}} = 30.9 \text{ pcpmpl}$$

~~The~~ LOS is D.

The answer is (B).

Why Other Options Are Wrong

(A) This answer is incorrect because a service flow rate of ~~1429~~ pcphpl would equate to a density of ~~23.8~~ pcpmpl, resulting in LOS C at a free-flow speed (FFS) of 60 mph. This answer could only be selected if FFS were not reduced by the flow friction factors.

(C) This answer is incorrect because it assumes a PHF of 0.80, typical of traffic flows near peak demand locations. The resulting density would appear to be LOS E.

(D) This answer is incorrect because not dividing the adjusted flow volume by two lanes results in a density that appears to be LOS F, or jam density.

SOLUTION 3

The total volume, which consists of a mix of vehicle types, must be converted to equivalent passenger-car volume by assigning passenger-car equivalents to the trucks, buses, and RVs. The heavy vehicle factor, f_{HV}, is determined by *Highway Capacity Manual* (HCM) Eq. 12-10. (See Exh. 12-25 for passenger-car equivalents.)

Heavy vehicles include all single-unit trucks (SUTs), trailer trucks (TTs), and recreational vehicles (RVs). P_T is the decimal proportion of the total of these three groups.

$$\begin{aligned} f_{\text{HV}} &= \frac{1}{1+P_T(E_T-1)} \\ &= \frac{1}{1+(0.07)(3.0-1)} \\ &= 0.877 \end{aligned}$$

The equivalent passenger-car flow rate is determined by HCM Eq. 12-9. ~~f_p is assumed to be 1.0 and is not included in the volume calculation.~~

$$\begin{aligned} v_p &= \frac{V}{PHF \times N \times f_{\text{HV}}} = \frac{1970\ \frac{\text{veh}}{\text{hr}}}{(0.85)(4\text{ lanes})(0.877)} \\ &= 661\text{ pcphpl} \end{aligned}$$

(660 pcphpl)

The answer is (D).

Why Other Options Are Wrong

(A) This incorrect answer results from misplacing the PHF in the numerator.

(B) This erroneous answer occurs if the correction for passenger-car equivalents was not included in the volume adjustment.

(C) This erroneous answer occurs if the passenger-car equivalents were selected for level terrain instead of for rolling terrain.

SOLUTION 4

The free-flow speed, FFS, is given as

$$\text{FFS} = 60\text{ mi/hr}$$

RVs (1%) and buses (2%) are counted together as single-unit trucks (SUTs), for a total of 3%. With trailer trucks (TTs) at 3% and SUTs at 3%, the mix is considered 50% SUT and 50% TT. Therefore, HCM Exh. 12-27 applies. With 5% grade for 0.125 mi,

$$E_T = 2.25$$

Using HCM Eq. 12-10, determine the heavy vehicle factor, f_{HV}.

$$\begin{aligned} f_{\text{HV}} &= \frac{1}{1+P_T(E_T-1)} \\ &= \frac{1}{1+(0.06)(2.25-1)} \\ &= 0.925 \end{aligned}$$

Since the driver population is primarily local commuters familiar with the freeway segment, the driver population factor, f_p, is assumed to be 1.0 and is not included in the volume calculation. Determine the peak traffic volume, v_p, using HCM Eq. 12-9.

$$\begin{aligned} v_p &= \frac{V}{PHF \times N \times f_{\text{HV}}} = \frac{5700\ \frac{\text{veh}}{\text{hr}}}{(0.94)(3)(0.925)} \\ &= 2185\text{ pcphpl} \end{aligned}$$

Using HCM Eq. 12-11, determine the density, D, in passenger cars per mile per lane.

$$D = \frac{v_p}{S} = \frac{2185\text{ pcphpl}}{60\ \frac{\text{mi}}{\text{hr}}} = 36.4\text{ pcpmpl}$$

From HCM Exh. 12-15, based on the density range of 35–45 pcpmpl, the LOS is E.

The answer is (C).

Why Other Options Are Wrong

(A) This incorrect answer could result from improper use of PHF and passenger car equivalents. According to HCM Exh. 12-15, in order to operate at LOS C, there would have to be between 18 pcpmpl and 26 pcpmpl.

(B) In order to operate at LOS D, the maximum density would have to have been found to be between 26 pcpmpl and 35 pcpmpl. This density can be incorrectly found by not including the heavy vehicle ~~and driver population factors~~.

Rearrange to find f, setting v_2 at 0 mph, G at 0 ft/ft, and g at 32.2 ft/sec^2.

$$\begin{aligned} f &= \frac{v_1^2}{2gs_b} \\ &= \frac{\left(\left(50\ \frac{\text{mi}}{\text{hr}}\right)\left(5280\ \frac{\text{ft}}{\text{mi}}\right)\right)^2}{(2)\left(32.2\ \frac{\text{ft}}{\text{sec}^2}\right)(277.7\ \text{ft})\left(3600\ \frac{\text{sec}}{\text{hr}}\right)^2} \\ &= 0.30 \end{aligned}$$

Determine the braking distance from 60 mph using the same friction factor.

$$\begin{aligned} s_{b,60} &= \frac{v_1^2 - v_2^2}{2g(f+G)} \\ &= \left(\frac{\left(60\ \frac{\text{mi}}{\text{hr}}\right)^2 - \left(0\ \frac{\text{mi}}{\text{hr}}\right)}{(2)\left(32.2\ \frac{\text{ft}}{\text{sec}^2}\right)(0.30)}\right)\left(\frac{5280\ \frac{\text{ft}}{\text{mi}}}{3600\ \frac{\text{sec}}{\text{hr}}}\right)^2 \\ &= 400.3\ \text{ft} \end{aligned}$$

Determine the perception-reaction distance for 60 mph.

$$\begin{aligned} s_{r,60} &= s_{s,60} - s_{b,60} \\ &= 461\ \text{ft} - 400.3\ \text{ft} \\ &= 60.7\ \text{ft} \end{aligned}$$

Determine the perception-reaction time required at 60 mph.

$$\begin{aligned} t_{r,60} &= \frac{s_{r,60}}{v} \\ &= \left(\frac{60.7\ \text{ft}}{60\ \frac{\text{mi}}{\text{hr}}}\right)\left(\frac{3600\ \frac{\text{sec}}{\text{hr}}}{5280\ \frac{\text{ft}}{\text{mi}}}\right) \\ &= 0.690\ \text{sec} \quad (0.7\ \text{sec}) \end{aligned}$$

The answer is (C).

Why Other Options Are Wrong

(A) This incorrect answer results from an improper conversion from miles per hour to feet per second.

(B) This incorrect answer results from an improper conversion from miles per hour to feet per second, ignoring the negative value and assuming the answer must be a positive value.

(D) This incorrect answer results from neglecting the perception-reaction distance at 50 mph to establish the braking friction factor, thereby assuming an average friction factor for the entire stopping distance. The answer assumes a positive value.

SOLUTION 8

The lead car stops in 278 ft, which is the braking distance, $s_{b,\text{lead}}$. The braking distance determines the minimum friction factor, f.

$$s_{b,\text{lead}} = \frac{v_1^2 - v_2^2}{2g(f+G)}$$

Set v_2 at 0 mph, and solve for f.

$$\begin{aligned} f &= \frac{v_1^2}{2gs_{b,50}} \\ &= \frac{\left(\left(50\ \frac{\text{mi}}{\text{hr}}\right)\left(5280\ \frac{\text{ft}}{\text{mi}}\right)\right)^2}{(2)\left(32.2\ \frac{\text{ft}}{\text{sec}^2}\right)(278\ \text{ft})\left(3600\ \frac{\text{sec}}{\text{hr}}\right)^2} \\ &= 0.30 \end{aligned}$$

The stopping distance, s_s, for the following car includes braking distance, s_b, and perception-reaction distance, s_r.

$$s_s = s_r + s_b$$

Find the braking distance from 60 mph using the friction factor for the lead car.

$$\begin{aligned} s_{b,60} &= \frac{v_1^2 - v_2^2}{2g(f+G)} \\ &= \left(\frac{\left(60\ \frac{\text{mi}}{\text{hr}}\right)^2 - \left(0\ \frac{\text{mi}}{\text{hr}}\right)}{(2)\left(32.2\ \frac{\text{ft}}{\text{sec}^2}\right)(0.30)}\right)\left(\frac{5280\ \frac{\text{ft}}{\text{mi}}}{3600\ \frac{\text{sec}}{\text{hr}}}\right)^2 \\ &= 400.3\ \text{ft} \end{aligned}$$

Determine the perception-reaction distance for the following car.

$$\begin{aligned} s_{r,60} &= vt_r \\ &= \frac{\left(60\ \frac{\text{mi}}{\text{hr}}\right)\left(5280\ \frac{\text{ft}}{\text{mi}}\right)(2.0\ \text{sec})}{3600\ \frac{\text{sec}}{\text{hr}}} \\ &= 176\ \text{ft} \end{aligned}$$

Determine the stopping distance from 60 mph, $s_{s,60}$.

$$\begin{aligned} s_{s,60} &= s_{r,60} + s_{b,60} \\ &= 176 \text{ ft} + 400.3 \text{ ft} \\ &= 576.3 \text{ ft} \end{aligned}$$

Determine the minimum following distance.

$$\begin{aligned} s_f &= s_{s,60} - s_{b,50} \\ &= 576.3 \text{ ft} - 278 \text{ ft} \\ &= 298.3 \text{ ft} \quad (300 \text{ ft}) \end{aligned}$$

The answer is (C).

Why Other Options Are Wrong

(A) This incorrect answer results from assuming that the braking distance for the lead car includes perception-reaction time (2.0 sec).

(B) This answer results from incorrectly subtracting the length of the lead car (20 ft standard from *A Policy on Geometric Design of Highways and Streets* (GDHS)) from the available stopping distance for the following car.

(D) This incorrect answer results from adding the length of the lead car (20 ft standard from GDHS) to the stopping distance available for the following car.

SOLUTION 9

The total distance between stations has three components of travel.

$$s_{\text{total}} = s_{\text{accel}} + s_{\text{running}} + s_{\text{decel}} = 5280 \text{ ft}$$

Determine the distance necessary to accelerate to 80 mph and the distance necessary to decelerate from 80 mph to a stop.

$$\begin{aligned} s_{\text{accel}} &= \frac{\text{v}_2^2 - \text{v}_1^2}{2a} \\ &= \left(\frac{\left(80 \ \frac{\text{mi}}{\text{hr}}\right)^2 - \left(0 \ \frac{\text{mi}}{\text{hr}}\right)^2}{(2)\left(5.5 \ \frac{\text{ft}}{\text{sec}^2}\right)\left(3600 \ \frac{\text{sec}}{\text{hr}}\right)^2} \right)\left(5280 \ \frac{\text{ft}}{\text{mi}}\right)^2 \\ &= 1252 \text{ ft} \end{aligned}$$

$$\begin{aligned} s_{\text{decel}} &= \frac{\text{v}_2^2 - \text{v}_1^2}{2d} \\ &= \left(\frac{\left(0 \ \frac{\text{mi}}{\text{hr}}\right)^2 - \left(80 \ \frac{\text{mi}}{\text{hr}}\right)^2}{(2)\left(-4.4 \ \frac{\text{ft}}{\text{sec}^2}\right)\left(3600 \ \frac{\text{sec}}{\text{hr}}\right)^2} \right)\left(5280 \ \frac{\text{ft}}{\text{mi}}\right)^2 \\ &= 1564 \text{ ft} \end{aligned}$$

Subtract the acceleration and deceleration distances from 1 mi to find the distance of constant running at 80 mph.

$$\begin{aligned} s_{\text{running}} &= 5280 \text{ ft} - s_{\text{accel}} - s_{\text{decel}} \\ &= 5280 \text{ ft} - 1252 \text{ ft} - 1564 \text{ ft} \\ &= 2464 \text{ ft} \end{aligned}$$

Determine the constant running speed time.

$$\begin{aligned} t_{\text{running}} &= \frac{s_{\text{running}}}{\text{v}} \\ &= \left(\frac{2464 \text{ ft}}{\left(80 \ \frac{\text{mi}}{\text{hr}}\right)\left(5280 \ \frac{\text{ft}}{\text{mi}}\right)} \right)\left(3600 \ \frac{\text{sec}}{\text{hr}}\right) \\ &= 21.0 \text{ sec} \end{aligned}$$

Determine the acceleration time.

$$\begin{aligned} t_{\text{accel}} &= \frac{\text{v}_2 - \text{v}_1}{a} \\ &= \frac{\left(80 \ \frac{\text{mi}}{\text{hr}}\right)\left(5280 \ \frac{\text{ft}}{\text{mi}}\right) - 0 \ \frac{\text{mi}}{\text{hr}}}{\left(5.5 \ \frac{\text{ft}}{\text{sec}^2}\right)\left(3600 \ \frac{\text{sec}}{\text{hr}}\right)} \\ &= 21.3 \text{ sec} \end{aligned}$$

Determine the deceleration time.

$$\begin{aligned} t_{\text{decel}} &= \frac{\text{v}_2 - \text{v}_1}{d} \\ &= \frac{0 \ \frac{\text{mi}}{\text{hr}} - \left(80 \ \frac{\text{mi}}{\text{hr}}\right)\left(5280 \ \frac{\text{ft}}{\text{mi}}\right)}{\left(-4.4 \ \frac{\text{ft}}{\text{sec}^2}\right)\left(3600 \ \frac{\text{sec}}{\text{hr}}\right)} \\ &= 26.7 \text{ sec} \end{aligned}$$

Determine the total distance to accelerate to 150 mph then decelerate to a stop.

$$\begin{aligned} s_{\text{total}} &= \frac{4175 \text{ ft} + 6263 \text{ ft}}{5280 \ \frac{\text{ft}}{\text{mi}}} \\ &= 1.98 \text{ mi} \quad (2.0 \text{ mi}) \end{aligned}$$

The minimum station spacing is approximately 2.0 mi.

The answer is (C).

Why Other Options Are Wrong

(A) This incorrect distance only covers acceleration to 150 mph.

(B) This answer results from an incorrect conversion from miles per hour to feet per second.

(D) This incorrect answer results from using the metric value of gravity acceleration without converting to feet per second.

SOLUTION 12

Saturation flow rate is determined from *Highway Capacity Manual* (HCM) Eq. 19-8.

$$\begin{aligned} s &= s_o f_w f_{\text{HV}g} f_p f_{\text{bb}} f_a f_{\text{LU}} f_{\text{LT}} f_{\text{RT}} f_{L\text{pb}} f_{R\text{pb}} f_{\text{wz}} f_{\text{ms}} f_{\text{sp}} \\ s_o &= 1900 \text{ pcphpl} \\ f_w &= 1.00 \quad \text{[HCM Exh. 19-20]} \end{aligned}$$

Solve for the following.

$$\begin{aligned} f_{\text{HV}g} &= \frac{100 - 0.3P_{\text{HV}} - 2.07P_g}{100} \\ &= \frac{100 - 0 - (2.07)(4)}{100} \\ &= 0.92 \end{aligned}$$

$$\begin{aligned} f_p &= \frac{N - 0.1 - \frac{18N_m}{3600}}{N} \\ &= \frac{1 - 0.1 - \frac{(18)(20)}{3600}}{1} \\ &= 0.80 \end{aligned}$$

$$\begin{aligned} f_{\text{bb}} &= \frac{N - \frac{14.4N_B}{3600}}{N} \\ &= \frac{1 - \frac{(14.4)(0)}{3600}}{1} \\ &= 1.00 \end{aligned}$$

$$f_a = 0.90 \quad \text{[for CBD]}$$

$$f_{\text{LU}} = 1.0 \quad \text{[HCM p. 19-47]}$$

$$\begin{aligned} f_{\text{LT}} &= \frac{1}{1.0 + 0.05P_{\text{LT}}} \\ &= \frac{1}{1.0 + (0.05)(0)} \\ &= 1.00 \end{aligned}$$

Use HCM Eq. 19-13 to adjust for the right-turn shared lane turning path. Apply $E_R = 1.18$ only to the proportion of right-turning vehicles.

$$\begin{aligned} f_{\text{RT}} &= 1 - \left(\frac{1}{E_R}\right)\left(\frac{V_{\text{RT}}}{V}\right) \\ &= 1 - \left(\frac{1}{1.18}\right)\left(\frac{139 \ \frac{\text{veh}}{\text{hr}}}{139 \ \frac{\text{veh}}{\text{hr}} + 731 \ \frac{\text{veh}}{\text{hr}}}\right) \\ &= 0.86 \end{aligned}$$

$$f_{L\text{pb}} = 1.00$$

$$f_{R\text{pb}} = 1.00$$

Determine the saturation flow rate.

$$\begin{aligned} s &= \left(1900 \ \frac{\text{pc}}{\text{hr-ln}}\right)(1.00)(1.00)(0.92)(0.80)(1.00)(0.90) \\ &\quad \times (1.00)(1.00)(0.86)(1.00)(1.00)(1.00) \\ &= 1082 \text{ vphpl} \end{aligned}$$

There is only one lane, so the total saturation flow rate is 1082 vph (1080 vph).

The answer is (B).

Why Other Options Are Wrong

(A) This incorrect answer is the sum of the intersection movements for approach E, or the approach volume.

(C) This incorrect answer is the result of neglecting to consider the approach grade.

(D) This incorrect answer is the result of considering the area type factor as 1.0 for CBD instead of 0.90.

by dividing a peak one-hour flow rate by the peak 15-minute flow rate within that hour.

The answer is (D).

Why Other Options Are Wrong

(A) The peak 15-minute flow rate divided by the peak one-hour flow rate yields a simple flow fraction, but not a comparison of flow rates.

(B) While other analysis methods, such as peak within a peak, may use a five-minute flow rate, the HCM uses a 15-minute flow rate for the peak-hour factor.

(C) The peak five-minute flow rate divided by the peak one-hour flow rate yields a simple flow fraction, but not a comparison of flow rates.

SOLUTION 23

Trucks descending long, steep downgrades generally travel more slowly than they do on long upgrades, in order to avoid loss of braking power and loss of directional control. Slower speeds on severe downgrades equate to larger passenger-car equivalents than on upgrades.

The answer is (C).

Why Other Options Are Wrong

(A) Buses in a traffic stream increase the passenger capacity far beyond the decrease in vehicle capacity based on auto equivalents.

(B) Trucks require more auto-equivalent space and usually carry no more, if not fewer, people than autos. Therefore, the passenger capacity of the roadway is lower.

(D) According to the *Highway Capacity Manual* (HCM), RVs ~~in small proportion to trucks can be included with trucks.~~

SOLUTION 24

Determine the UTF.

$$\text{UTF} = \left(\frac{1}{1000}\right)\left(\frac{\text{households}}{\text{auto}}\right)\left(\frac{\text{persons}}{\text{mi}^2}\right)$$
$$= \left(\frac{1}{1000}\right)\left(\frac{1\ \text{household}}{0.80\ \text{auto}}\right)\left(10{,}000\ \frac{\text{persons}}{\text{mi}^2}\right)$$
$$= 12.50$$

Determine the percentage of trips on transit.

$$\%\ \text{transit usage} = \frac{\text{UTF}}{0.6} = \frac{12.50}{0.6} = 20.8\%$$

Determine the number of residents expected to use transit.

$$\begin{aligned}\text{no. of residents using transit} &= (\text{transit usage})(\text{no. of trips}) \\ &= (0.208)(12{,}000) \\ &= 2500\ \text{persons}\end{aligned}$$

The answer is (D).

Why Other Options Are Wrong

(A) This incorrect answer results from applying the UTF directly to the population to obtain transit users.

(B) This incorrect answer results from reversing the auto density to 0.80 households per auto instead of 0.80 autos per household.

(C) This incorrect answer results from applying the transit percentage to the population density instead of to the total trips.

SOLUTION 25

Determine the total round trip vehicle-miles traveled.

$$\text{VM}_{\text{total}} = s\left(P_c P_{\text{total}}\left(\frac{1}{O_c}\right) + P_{\text{sov}} P_{\text{total}}\left(\frac{1}{O_{\text{sov}}}\right)\right)$$
$$= (2)(8\ \text{mi})\left(\begin{array}{l}(0.25)(7000\ \text{pers})\left(\dfrac{1}{2.10\ \dfrac{\text{pers}}{\text{veh}}}\right) \\ +(0.75)(7000\ \text{pers})\left(\dfrac{1}{1\ \dfrac{\text{pers}}{\text{veh}}}\right)\end{array}\right)$$
$$= 97{,}333\ \text{veh-mi}$$

Determine the fuel consumption rate per vehicle.

$$F = 0.0362\ \frac{\text{gal}}{\text{veh-mi}} + \left(\frac{0.746\ \dfrac{\text{gal}}{\text{veh-hr}}}{20\ \dfrac{\text{mi}}{\text{hr}}}\right) = 0.0735\ \text{gal/veh-mi}$$

Determine the gallons of fuel consumed.

$$\begin{aligned}\text{total fuel consumption} &= (\text{total vehicle-miles})F \\ &= (97{,}333\ \text{veh-mi})\times\left(0.0735\ \frac{\text{gal}}{\text{veh-mi}}\right) \\ &= 7154\ \text{gal}\end{aligned}$$

Determine the Btu use.

$$\begin{aligned}\text{Btu use} &= (\text{total fuel consumption})\left(\frac{\text{Btu content}}{\text{gal}}\right)\\ &= (7154\ \text{gal})\left(125{,}000\ \frac{\text{Btu}}{\text{gal}}\right)\\ &= 8.94\times10^{8}\ \text{Btu}\quad(8.9\times10^{8}\ \text{Btu})\end{aligned}$$

The answer is (C).

Why Other Options Are Wrong

(A) This is the number of gallons consumed per day.

(B) This is the total one-way Btu consumption.

(D) This is the Btu consumption if all 7000 commuters arrived in a single-occupant vehicle.

SOLUTION 26

Improvement to public safety in urban areas involves providing an adequate physical environment so that individuals do not feel insecure or uncertain about what is happening nearby. All of the listed items can improve the feeling of personal comfort or reduce the level of discomfort with surroundings. The traveling public is surrounded by noise and vibration in an urban setting at a fairly constant level. Conceivably, the noise level in a transit terminal, option D, would not be much different from that on the street. The other three options could be perceived as making a greater difference within the transit terminal than in the surrounding neighborhood, thereby improving the safety of the terminal itself.

The answer is (D).

Why Other Options Are Wrong

(A) Improving personal security is an inherent goal of improving public safety.

(B) Reliability in transit service reduces anxiety about unknown arrival times. This reduces the need for longer waiting times in the terminal, especially during off-peak travel times.

(C) Adequate lighting levels significantly improve the travelers' confidence in their surroundings and lead to an improved feeling of personal safety.

SOLUTION 27

Determine the number of person-trips per household.

$$\begin{aligned}T &= 0.78 + 1.3P + 2.3A\\ &= 0.78 + (1.3)\left(3.5\ \frac{\text{persons}}{\text{household}}\right)\\ &\quad + (2.3)\left(2.2\ \frac{\text{autos}}{\text{household}}\right)\\ &= 10.39\ \text{trips/household-day}\end{aligned}$$

Determine the number of person-trips in the entire zone.

$$\begin{aligned}N &= T(\text{no. of households})\\ &= \left(10.39\ \frac{\text{trips}}{\text{household-day}}\right)(600\ \text{households})\\ &= 6234\ \text{trips/day}\end{aligned}$$

Determine the number of auto trips per day.

$$\begin{aligned}T_A &= (\text{fraction of auto trips})\left(\frac{\text{total trips}}{\text{day}}\right)\\ &= (0.94)\left(6234\ \frac{\text{trips}}{\text{day}}\right)\\ &= 5860\ \text{trips/day}\quad(5900\ \text{trips/day})\end{aligned}$$

The answer is (B).

Why Other Options Are Wrong

(A) This is the number of trips per household.

(C) This is the total number of trips per day for the zone, including transit and other modes.

(D) This answer results from misuse of the modal split ratio.

SOLUTION 28

The hourly vehicle volume is a count of the general mix of vehicle types, usually reported in units of vehicles per hour. The design hourly flow rate must reflect the influence of heavy vehicles~~, the~~ hourly variation of traffic~~,~~ ~~and the characteristics of the driver population.~~ The design equivalent passenger-car flow rate is calculated using the heavy-vehicle and peak-hour adjustment factors ~~and includes the driver population adjustment.~~

The answer is (B).

Why Other Options Are Wrong

(A) This incorrect answer is an inaccurate definition of peak hour factor.

(C) This incorrect answer does not convert to passenger-car equivalents.

(D) This incorrect answer does not correct for passenger-car equivalents.

SOLUTION 29

The directional capacity is the capacity per lane times the number of lanes in each direction.

$$\begin{aligned}\text{directional capacity} &= \left(\frac{\text{capacity}}{\text{hr-lane}}\right)(\text{no. of lanes})\\ &= \left(2100\ \frac{\text{veh}}{\text{hr-lane}}\right)(2\ \text{lanes})\\ &= 4200\ \text{vph}\end{aligned}$$

The arrival rate is more than the one-lane capacity. Therefore, when one lane is blocked, there will be a queue of vehicles behind the blockage. Once the blocked lane is opened, the capacity will be greater than the arrival rate, so the queue will begin to dissipate. The total queue to be dissipated is as follows.

$$\begin{aligned}\text{total queue length} &= \sum\text{arrivals} - \sum\text{departures}\\ &= \text{arrivals during blockage}\\ &\quad + \text{arrivals during queue dissipation}\\ &\quad - \text{departures during lane blockage}\\ &\quad - \text{departures during queue dissipation}\end{aligned}$$

Arrivals for 20 min plus arrivals for time t equals departures at the one-lane rate for 20 min plus departures at the two-lane rate for time t.

Determine the arrival rate.

$$\begin{aligned}\text{arrival rate} &= \frac{3100\ \frac{\text{veh}}{\text{hr}}}{60\ \frac{\text{min}}{\text{hr}}}\\ &= 51.7\ \text{vpm}\end{aligned}$$

Determine the two-lane departure rate, which is the full directional capacity.

$$\begin{aligned}\text{two-lane departure rate} &= \frac{4200\ \frac{\text{veh}}{\text{hr}}}{60\ \frac{\text{min}}{\text{hr}}}\\ &= 70\ \text{vpm}\end{aligned}$$

Determine the one-lane departure rate, which is the departure rate during the incident blockage (see *Highway Capacity Manual* (HCM) Exh. 11-23).

$$\begin{aligned}\text{incident blockage departure rate} &= \frac{\left(70\ \frac{\text{veh}}{\text{min}}\right)(0.70)}{2}\\ &= 24.5\ \text{vpm}\end{aligned}$$

Departure time, t, is the time necessary to dissipate the queue. The time is 0 sec at the instant of clearing the blockage. The total queue length is

$$\begin{aligned}\text{total queue length} &= \left(51.7\ \frac{\text{veh}}{\text{min}}\right)(20\ \text{min}) + \left(51.7\ \frac{\text{veh}}{\text{min}}\right)t\\ &= \left(24.5\ \frac{\text{veh}}{\text{min}}\right)(20\ \text{min}) + \left(70\ \frac{\text{veh}}{\text{min}}\right)t\\ t &= 29.7\ \text{min}\quad (30\ \text{min})\end{aligned}$$

The entire queue will be dissipated in approximately 30 min.

The answer is (B).

Why Other Options Are Wrong

(A) This incorrect answer results from including in the calculation only the vehicle(s) that arrive during the blockage.

(C) This incorrect answer results from adding the 20 min blockage time to the queue dissipation time after the blockage.

(D) This incorrect answer results from not subtracting the single-lane departures during the 20 min lane blockage.

Determine the ratio of the segment crash rate with respect to the statewide critical rate.

$$\frac{R_{\text{seg}}}{R_{\text{crit}}} = \frac{35.2 \ \frac{\text{crashes}}{\text{HMVM}}}{190 \ \frac{\text{crashes}}{\text{HMVM}}} = 0.185 \quad (0.19)$$

The answer is (B).

Why Other Options Are Wrong

(A) This incorrect answer results from applying the test factor directly to the statewide crash rate.

(C) This answer results from incorrectly determining the critical rate.

(D) This incorrect answer results from comparing the statewide critical rate to the segment crash rate.

SOLUTION 42

Find the base total number of roadway segment crashes per year using *Highway Safety Manual* (HSM) Eq. 11-9 and Table 11-5.

$$\begin{aligned} N_{\text{spf rd}} &= e^{(a+b(\ln \text{AADT})+\ln L)} \\ &= e^{(-9.025+(1.049)(\ln 18{,}000)+\ln 2)} \\ &= 7.003 \text{ crashes/yr} \end{aligned}$$

Check against the graph of SPF shown in HSM Fig. 11-4.

For an AADT of 18,000 veh/day, the approximate average crash frequency is ± 3.5 crash/mi. For a 2 mi segment, $2 \pm 3.5 = \pm 7.0$ crash/mi. Therefore, the calculated base total number of crashes for this segment is correct.

Adjust the base total to reflect the site-specific geometric conditions and determine the crash modification factors, $\text{CMF}_{1\text{rd}}$ through $\text{CMF}_{5\text{rd}}$, for divided roadway segments. From HSM Table 11-16, the CMF for ~~a 12 ft lane width is 1.00, the~~ default value for the proportion of total crashes constituted by related crashes, p_{ra}, is 0.50~~, and CMF_{ra} is 1.03.~~

$$\begin{aligned} \text{CMF}_{1\text{rd}} &= p_{\text{ra}}(\text{CMF}_{\text{ra}} - 1.0) + 1.0 \\ &= (0.50)(1.03 - 1.0) + 1.0 \\ &= 1.015 \end{aligned}$$

The CMF for a 6 ft wide paved right shoulder is determined from HSM Table 11-17.

$$\text{CMF}_{2\text{rd}} = 1.04$$

The CMF for an 8 ft wide median without a barrier is approximated from HSM Table 11-18.

$$\text{CMF}_{3\text{rd}} = 1.04$$

The CMF for lighting is

$$\text{CMF}_{4\text{rd}} = 1.00 \quad \text{[base condition]}$$

The CMF for speed enforcement is

$$\text{CMF}_{5\text{rd}} = 1.00 \quad \text{[base condition]}$$

Using HSM Eq. 11-3, the predicted average crash frequency for the road segment is

$$\begin{aligned} N_{\text{predicted rs}} &= N_{\text{spf rd}} C_r \text{CMF}_{1\text{rd}} \text{CMF}_{2\text{rd}} \\ &\quad \times \text{CMF}_{3\text{rd}} \text{CMF}_{4\text{rd}} \text{CMF}_{5\text{rd}} \\ &= \left(7.003 \ \frac{\text{crashes}}{\text{yr}}\right)(0.94)(1.015)(1.04) \\ &\quad \times (1.04)(1.00)(1.00) \\ &= 7.227 \text{ crashes/yr} \quad (7.23 \text{ crashes/yr}) \end{aligned}$$

The answer is (C).

Why Other Answers Are Wrong

(A) This incorrect answer results from using a 1 mi long segment, instead of 2 mi.

(B) This incorrect answer results from using a 1 mi long segment, instead of 2 mi, and not applying the local calibration factor.

(D) This incorrect answer results from not applying the local calibration factor.

SOLUTION 43

Using HSM Eq. 10-8, the predicted average crash frequency of a three-leg, stop-controlled intersection is

$$\begin{aligned} N_{\text{spf 3ST}} &= \exp\left(\begin{array}{l} -9.86 + 0.79 \ln(\text{AADT}_{\text{maj}}) \\ +0.49 \ln(\text{AADT}_{\text{min}}) \end{array}\right) \\ &= \exp\left(\begin{array}{l} -9.86 + 0.79 \ln\left(7000 \ \frac{\text{veh}}{\text{day}}\right) \\ +0.49 \ln\left(1000 \ \frac{\text{veh}}{\text{day}}\right) \end{array}\right) \\ &= 1.681 \text{ crashes/yr} \end{aligned}$$

Why Other Options Are Wrong

(B) This incorrect answer results from using the formula for the offset from tangent to a point on a curve, then subtracting the result from the offset to the pier foundation corner.

(C) This incorrect answer results from considering triangle AOB. The hypotenuse, BO, is 206.16 ft. Subtracting the offset of 33.33 ft from the hypotenuse length yields 34.36 ft, which is the incorrect clearance distance to the closest corner of the pier foundation.

(D) This incorrect answer results from subtracting the squares of the side lengths in the Pythagorean theorem instead of adding the squares of the sides.

SOLUTION 9

Stopping distance around an obstruction is the distance along the centerline of the innermost lane of the curve, which is on a 250 ft radius. The sight line is along the chord of the curve. The clearance distance, $Z_{\text{clearance}}$, plus one-half of the inside lane width, is the horizontal sight offset (HSO). The HSO is the mid-ordinate, M, of the circular curve centered at the point of closest clearance to the edge of the road.

$$\begin{aligned}\text{HSO} = M &= Z_{\text{clearance}} + \frac{\text{lane width}}{2} \\ &= 14 \text{ ft} + \frac{12 \text{ ft}}{2} \\ &= 20 \text{ ft}\end{aligned}$$

Determine the curve length using the mid-ordinate and the radius. First, find the deflection, I, of the curve arc. The deflection is the angle subtended by the ends of the sight-line chord.

$$M = R\left(1 - \cos\frac{I}{2}\right)$$

Rearrange to solve for I.

$$\begin{aligned}I &= 2\arccos\left(1 - \frac{M}{R}\right) \\ &= 2\arccos\left(1 - \frac{20 \text{ ft}}{250 \text{ ft}}\right) \\ I &= 46.148°\end{aligned}$$

Determine the length of curve, L, using the radius and the degree of curve, D.

$$\begin{aligned}L &= \frac{100I}{D} = \frac{\pi IR}{180°} = \frac{\pi(46.148°)(250 \text{ ft})}{180°} \\ &= 201.4 \text{ ft}\end{aligned}$$

Determine the chord length for the sight distance.

$$\begin{aligned}C &= 2R\sin\frac{I}{2} = (2)(250 \text{ ft})(\sin 46.148°) \\ &= ~~195.56~~ \text{ ft}\end{aligned}$$

~~A sight distance of 195.96 ft is good for a 30 mph design speed.~~

The answer is (C).

Alternate Solution

Using AASHTO's *A Policy on Geometric Design of Highways and Streets* ~~(GDHS) Fig. 3-26~~, from a point at 20 ft mid-ordinate on the bottom horizontal scale, intersect a line from a point at a 250 ft radius on the left vertical scale. The intersection lies on the v = 30 mph curve.

Why Other Options Are Wrong

(A) After determining $I/2$ from the mid-ordinate equation, this incorrect answer results from not doubling the answer for the length-of-curve equation.

(B) This answer results from incorrectly using the radius of the inside edge of the lane as the sight line.

(D) This answer results from incorrectly using the radius of the outside edge of the traveled lane as the sight line.

SOLUTION 10

Change the design speed from miles per hour to feet per second.

$$\begin{aligned}v_{\text{ft/sec}} &= \frac{\left(70\ \frac{\text{mi}}{\text{hr}}\right)\left(5280\ \frac{\text{ft}}{\text{mi}}\right)}{3600\ \frac{\text{sec}}{\text{hr}}} \\ &= 102.7 \text{ ft/sec}\end{aligned}$$

The roadway is already sloped down to the right at 0.01 ft/ft, and the superelevation will increase the slope of the roadway down to the right since this is a right-hand curve. Therefore, there will be no reverse cross slope to run out on the right side. Determine the difference between normal slope and full superelevation.

$$\begin{aligned}\text{required slope transition} &= 0.08\ \frac{\text{ft}}{\text{ft}} - 0.01\ \frac{\text{ft}}{\text{ft}} \\ &= 0.07 \text{ ft/ft}\end{aligned}$$

The rate of change can be related to the unknown length of the vertical curve.

$$R = \frac{G_2 - G_1}{L} = \frac{0.038\ \frac{\text{ft}}{\text{ft}} - \left(-0.036\ \frac{\text{ft}}{\text{ft}}\right)}{L}$$
$$= \frac{0.074\ \frac{\text{ft}}{\text{ft}}}{L}$$
$$\frac{R}{2} = \frac{0.037\ \frac{\text{ft}}{\text{ft}}}{L}$$

Insert values into the complete elevation formula.

$$E_\text{P} = E_\text{PVC} + G_1 x + \left(\frac{R}{2}\right)x^2$$
$$638.42\ \text{ft} = 646.12\ \text{ft} + \left(-0.036\ \frac{\text{ft}}{\text{ft}}\right)(550\ \text{ft}) + \left(\frac{0.037\ \frac{\text{ft}}{\text{ft}}}{L}\right)(550\ \text{ft})^2$$

Rearrange.

$$L = \frac{\left(0.037\ \frac{\text{ft}}{\text{ft}}\right)(550\ \text{ft})^2}{638.42\ \text{ft} - 642.12\ \text{ft} + \left(0.036\ \frac{\text{ft}}{\text{ft}}\right)(550\ \text{ft})}$$
$$= 925\ \text{ft}$$

The answer is (D).

Why Other Options Are Wrong

(A) Incorrectly determining the difference in absolute grades instead of the total change in grade results in a very short grade length.

(B) Neglecting to square the distance in the third term and assuming the answer is in stations because it is so small results in a vertical curve that is much too short.

(C) Omitting the minus sign for G_1 in the second term of the elevation equation and then ignoring the minus sign of the answer results in a curve that is too short.

SOLUTION 4

Convert travel speed to feet per second.

$$\text{v}_\text{horiz} = \frac{\left(150\ \frac{\text{mi}}{\text{hr}}\right)\left(5280\ \frac{\text{ft}}{\text{mi}}\right)}{3600\ \frac{\text{sec}}{\text{hr}}}$$
$$= 220\ \text{ft/sec}$$

Using 1 g = 32.2 ft/sec^2, calculate the vertical acceleration limit.

$$a_\text{vert} = (0.03)\left(32.2\ \frac{\text{ft}}{\text{sec}^2}\right)$$
$$= 0.966\ \text{ft/sec}^2$$

When the train is traveling on a downgrade, it has a negative vertical velocity. On an upgrade, the train has a positive vertical velocity.

Determine the vertical velocity on the downgrade.

$$\text{v}_\text{vert} = G_1 \text{v}_\text{horiz} = \left(\frac{-0.75\%}{100\%}\right)\left(220\ \frac{\text{ft}}{\text{sec}}\right)$$
$$= -1.650\ \text{ft/sec}$$

Determine the vertical velocity on the upgrade.

$$\text{v}_\text{vert} = G_2 \text{v}_\text{horiz} = \left(\frac{0.45\%}{100\%}\right)\left(220\ \frac{\text{ft}}{\text{sec}}\right)$$
$$= 0.990\ \text{ft/sec}$$

Determine the total change in vertical velocity.

$$\Delta\text{v} = |\text{v}_\text{down} - \text{v}_\text{up}|$$
$$= \left|-1.650\ \frac{\text{ft}}{\text{sec}} - 0.990\ \frac{\text{ft}}{\text{sec}}\right|$$
$$= 2.64\ \text{ft/sec}$$

Determine the number of seconds required to change direction.

$$t = \frac{\Delta\text{v}}{a_\text{vert}} = \frac{2.64\ \frac{\text{ft}}{\text{sec}}}{0.966\ \frac{\text{ft}}{\text{sec}^2}}$$
$$= 2.73\ \text{sec}$$

The distance traveled in 2.73 sec is the minimum length of vertical curve required.

$$L_{\text{min}} = tv_{\text{horiz}} = (2.73\ \text{sec})\left(220\ \frac{\text{ft}}{\text{sec}}\right)$$
$$= 601\ \text{ft} \quad (600\ \text{ft})$$

The answer is (C).

Why Other Options Are Wrong

(A) Using the difference in absolute vertical velocity values results in too short a curve.

(B) Neglecting to convert miles per hour into feet per second results in too short a curve.

(D) Inserting the metric value for gravitational acceleration and not converting to feet per second squared to determine the vertical acceleration limit results in too long a curve.

SOLUTION 5

Check for the approximate required sight distance from AASHTO's *A Policy on Geometric Design of Highways and Streets* (GDHS) Table 3-1. The recommended stopping distance for 70 mph is 730 ft. Therefore, the curve length of 1500 ft is probably greater than the required stopping sight distance.

Check the available sight distance. Use the GDHS formula for crest vertical curves for stopping sight distances less than the curve length.

$$L = \frac{AS^2}{2158} = \frac{(G_2 - G_1)S^2}{2158}$$

Solve for the sight distance available.

$$S = \sqrt{\frac{2158L}{A}} = \sqrt{\frac{(2158\ \text{ft})(1500\ \text{ft})}{3.0\% - (-2.5\%)}}$$
$$= 767\ \text{ft}$$

From GDHS Table 3-3, 767 ft is in the lower range of 50 mph sight distance conditions. However, designing for minimum stopping sight distance on freeways is not entirely safe. A car braking rapidly or coming to a full stop can cause multiple rear-end crashes in heavier traffic found on freeways, especially when rain or fog are present. Therefore, avoidance maneuvers A and B do not apply, and one of the remaining conditions, C, D, or E, applies. Avoidance maneuvers C, D, and E allow for a greater perception-reaction time and allow time for the driver to change vehicle path or speed when a full stop is undesirable. Avoidance maneuver C applies to rural roads; therefore, the speed should be posted at 50 mph.

The answer is (A).

Why Other Options Are Wrong

(B) Posting for 60 mph places the 767 ft stopping sight distance within the range of 610 ft to 1150 ft for avoidance manuevers A and B, which does not leave the additional distance required for lane change decisions and adverse weather conditions.

(C) Misinterpreting GDHS Table 3-2 by using the former criterion of a 0.6 ft object height yields a longer stopping sight distance required than does using the newer criteria. Applying the value to GDHS Table 3-2 yields a speed limit of 60 mph.

(D) Incorrectly using the stopping distance values from GDHS Table 3-1 and Table 3-2 for a design speed of 70 mph ignores the additional sight distance needed for avoidance maneuvers. Such measures may be required on the downgrade at the end of the vertical curve where the ramp merges into the mainline traffic, especially when experience has shown that a higher rate of crashes occur at this location.

SOLUTION 6

Assume $S < L$. From AASHTO's *A Policy on Geometric Design of Highways and Streets* (GDHS) Table 3-34, the stopping sight distance for a 50 mph design speed is 425 ft. Equation 3-43 is used with the eye height as 3.5 ft and the object height as 2.0 ft.

$$L = \frac{AS^2}{2158} = \frac{\big(3\% - (-4\%)\big)(425\ \text{ft})^2}{2158\ \text{ft}}$$
$$= 586\ \text{ft} \quad (590\ \text{ft})$$

Assume $S > L$, and solve for L.

$$L = 2S - \frac{2158}{A} = (2)(425\ \text{ft}) - \frac{2158\ \text{ft}}{\big(3\% - (-4\%)\big)}$$
$$= 542\ \text{ft} \quad (540\ \text{ft})$$

Since 542 ft is greater than the stopping sight distance required, the first assumption—that the stopping sight distance is less than the curve length—is valid. The length can be checked using the minimum design K value from GDHS Table 3-36.

$$L = KA = \left(84\ \frac{\text{ft}}{\%}\right)\big(3\% - (-4\%)\big)$$
$$= 588\ \text{ft} \quad (590\ \text{ft})$$

Vertical Design

By trial and error, the optimal cycle length is 54 sec.

The answer is (C).

Why Other Options Are Wrong

(A) In this incorrect answer, the critical intersection volume/capacity ratio included only the first two of the three approaches.

(B) In this incorrect answer, the lost time for each phase included only the start-up delay of 2 sec, and did not include the clearance delay.

(D) In this incorrect answer, the all-red interval default of 2 sec for each phase was added to the 4 sec delay for each phase. The 4 sec delay already includes the all-red interval.

SOLUTION 2

MUTCD is very clear that simply meeting the minimum warrants for a traffic signal is not sufficient to satisfy the requirements for installation. The *Traffic Engineering Handbook* further states that "…the requirements for a signal should be thoroughly analyzed with a decision to install based on a demonstrated traffic need."

Regardless of the minimum requirements that are met by traffic conditions, an engineering study is still needed to indicate that the installation will improve overall safety of the intersection.

The answer is (B).

Why Other Options Are Wrong

(A) A traffic jam in the evening caused by employee discharge from work can be reduced by alternate means, such as varying the quitting times of employees. Alternate means would have to be employed and observed before declaring that a signal is justified.

(C) Recurring crashes are only relevant if a traffic signal could somehow help prevent them. If drivers do not stop at a stop sign, there is no reason to believe they would stop at a traffic signal either. Thus, more information is needed about the nature of the crashes in order for this factor to be taken into consideration.

(D) Minimum pedestrian volume is to be 100 persons for each of any 4 hr period. Therefore, 100 persons for a 2 hr period is insufficient justification on its own.

SOLUTION 3

Solve for the following.

From *Highway Capacity Manual* (HCM) Eq. 19-9,

$$
\begin{aligned}
f_{\text{HV}} &= \frac{100 - 0.3P_{\text{HV}} - 2.07P_g}{100} \\
&= \frac{100 - 0 - 0}{100} \\
&= 1.0
\end{aligned}
$$

$$
\begin{aligned}
f_w &= 0.96 && \text{[HCM Ex. 19-20]} \\
f_{\text{bb}} &= 1.00 && \text{[bus movements not given]} \\
f_a &= 0.90 && \text{[CBD default value, HCM p. 19-12]} \\
f_{\text{LU}} &= 1.0 && \text{[HCM p. 19-12]} \\
f_{\text{LT}} &= 1.0 && \text{[no turning movements in through-only lanes]} \\
f_{\text{RT}} &= 1.0 && \text{[no turning movements in through-only lanes]} \\
f_{Lpb} &= 1.0 && \text{[ped-bike counts not given]} \\
f_{Rpb} &= 1.0 && \text{[ped-bike counts not given]}
\end{aligned}
$$

Determine the saturation flow rate from HCM Eq. 19-8.

$$
\begin{aligned}
s_o &= 1900 \text{ pcphpl} \\
s &= s_o f_w f_{\text{HV}g} f_p f_{\text{bb}} f_a f_{\text{LU}} f_{\text{LT}} f_{\text{RT}} f_{Lpb} f_{Rpb} f_{\text{wz}} f_{\text{ms}} f_{\text{sp}} \\
&= \left(1900 \frac{\text{pc}}{\text{hr-ln}}\right)(0.96)(0.97)(1.0)(1.00)(0.90)(1.0) \\
&\quad \times (1.00)(1.0)(1.0)(1.0) \\
&= 1592 \text{ pcphpl}
\end{aligned}
$$

Determine the lane group volume-to-capacity ratio.

$$
\begin{aligned}
c &= Ns\frac{g}{C} = (2)\left(1641 \frac{\text{pc}}{\text{hr-ln}}\right)\left(\frac{48 \text{ sec}}{110 \text{ sec}}\right) \\
&= 1432 \text{ pcph} \quad (1430 \text{ pcph})
\end{aligned}
$$

The answer is (B).

Why Other Options Are Wrong

(A) This incorrect answer is the capacity for one through-only lane.

(C) This incorrect answer is the total demand volume of the through lanes.

(D) This incorrect answer is the total through volume, including the volume from the shared right turn lane.

Signal Design

9 Drainage

PROBLEM 1

A stormwater detention pond uses a submerged, sharp-edged orifice to control discharge to an open channel. A standpipe is used to prevent the pool elevation from exceeding 100 ft. The centerline of the orifice is at an elevation of 92.6 ft. Most nearly, what diameter of the orifice will limit the discharge to 20 ft^3/sec?

(A) 0.77 ft

(B) 0.85 ft

(C) 1.1 ft

(D) 1.4 ft

Hint: This is similar to discharge from an open tank.

PROBLEM 2

Manometers are installed on the upstream and downstream sides of a straight pipe 20 ft long with an inside diameter 0.75 in. The flow rate through the pipe is 8 gal/min. The upstream manometer reading is 37.1 in, and the downstream manometer reading is 9.2 in. Assuming turbulent flow, what is most nearly the specific roughness of the pipe?

(A) 0.000013 ft

(B) 0.00022 ft

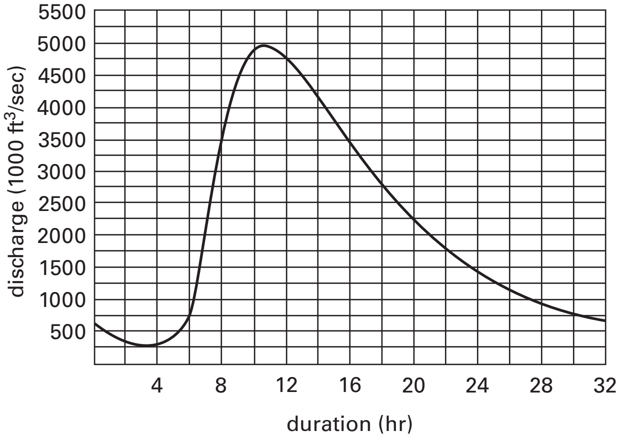
(C) 0.014 ft

(D) 0.019 ft

Hint: Use the Darcy equation.

PROBLEM 3

A storm hydrograph is shown. The basin drainage area is 5480 mi^2.



What is the approximate direct runoff from the storm?

(A) 0.016 in

(B) 2.6 in

(C) 13 in

(D) 16 in

Hint: Separate groundwater from direct runoff ~~and integrate~~.

SOLUTION 1

Assume the orifice to have negligible loss from velocity or fluid contraction. The elevation head above the orifice is

$$\begin{aligned} h &= 100\ \text{ft} - 92.6\ \text{ft} \\ &= 7.4\ \text{ft} \end{aligned}$$

For a sharp-edged orifice, the orifice coefficient is approximately 0.62.

$$\begin{aligned} A &= \frac{Q}{C_d\sqrt{2gh}} = \frac{20\ \frac{\text{ft}^3}{\text{sec}}}{(0.62)\sqrt{(2)\left(32.2\ \frac{\text{ft}}{\text{sec}^2}\right)(7.4\ \text{ft})}} \\ &= 1.48\ \text{ft}^2 \end{aligned}$$

Use the cross-sectional area equation and solve for the diameter.

$$\begin{aligned} A &= \frac{\pi D^2}{4} \\ D &= \sqrt{\frac{4A}{\pi}} \\ &= \sqrt{\frac{(4)(1.48\ \text{ft}^2)}{\pi}} \\ &= 1.4\ \text{ft} \end{aligned}$$

The answer is (D).

Why Other Options Are Wrong

(A) This incorrect answer applies the square root only to the numerator in the final equation to determine diameter. Other assumptions, definitions, and equations are the same as those used in the correct solution.

(B) This incorrect answer multiplies instead of divides the flow rate by the orifice coefficient in the area equation. Other assumptions, definitions, and equations are the same as those used in the correct solution.

(C) This incorrect answer ignores the contraction and velocity losses corrected by the orifice coefficient. Other assumptions, definitions, and equations are the same as those used in the correct solution.

SOLUTION 2

Assume that the only head loss in the pipe is from friction.

$$\begin{aligned} h_f &= z_2 - z_1 = \frac{37.1\ \text{in} - 9.2\ \text{in}}{12\ \frac{\text{in}}{\text{ft}}} \\ &= 2.33\ \text{ft} \end{aligned}$$

The cross-sectional area of the pipe is

$$A = \pi\frac{D^2}{4} = \frac{\pi\frac{(0.75\ \text{in})^2}{4}}{\left(12\ \frac{\text{in}}{\text{ft}}\right)^2} = 0.0031\ \text{ft}^2$$

The flow velocity is

$$\begin{aligned} \text{v} &= \frac{Q}{A} = \frac{8\ \frac{\text{gal}}{\text{min}}}{(0.0031\ \text{ft}^2)\left(60\ \frac{\text{sec}}{\text{min}}\right)\left(7.48\ \frac{\text{gal}}{\text{ft}^3}\right)} \\ &= 5.8\ \text{ft/sec} \end{aligned}$$

The friction factor is

$$\begin{aligned} f &= \frac{2h_f D_g}{L\text{v}^2} \\ &= \frac{(2)(2.33\ \text{ft})(0.75\ \text{in})\left(32.2\ \frac{\text{ft}}{\text{sec}^2}\right)}{(20\ \text{ft})\left(5.8\ \frac{\text{ft}}{\text{sec}}\right)^2\left(12\ \frac{\text{in}}{\text{ft}}\right)} \\ &= 0.014 \end{aligned}$$

Since flow is turbulent, use the von Karman-Nikuradse smooth pipe equation.

$$\begin{aligned} \frac{1}{\sqrt{f}} &= 1.74 - 2\log\left(\frac{2\epsilon}{D}\right) \\ \frac{1}{\sqrt{0.014}} &= 1.74 - 2\log\left(\frac{2\epsilon}{D}\right) \\ \frac{\epsilon}{D} &= 0.00021 \\ \epsilon &= \left(\frac{\epsilon}{D}\right)D \\ &= \frac{(0.00021)(0.75\ \text{in})}{12\ \frac{\text{in}}{\text{ft}}} \\ &= 0.000013\ \text{ft} \end{aligned}$$

The answer is (A).