Q1

Test of Independence

H_0: Perceptions of PMDs being allowed on footpath and respondents' personal income are independent of each other

H_1: Otherwise

	Level of Personal Income			
To allow PMDs on Shared Footpath	Low	Middle	High	**Total**
Yes	60	120	420	*600*
No	100	100	200	*400*
Total	*160*	*220*	*620*	*1000*

Expected frequency = $\frac{Column\ Total \times Row\ Total}{Sample\ Size}$, as tabulated below:

	Level of Personal Income		
To allow PMDs on Shared Footpath	Low	Middle	High
Yes	96	132	372
No	64	88	248

$$Degree\ of\ Freedom, v = (2-1)(3-1) = 2$$

χ^2 observed = $Sum\ of\ \left(\frac{(Observed\ Frequency - Expected\ Frequency)^2}{Expected\ Frequency}\right)$

Test of Independence:

$$\chi^2_{obs} = \frac{(60-96)^2}{96} + \frac{(100-64)^2}{64} + \frac{(120-132)^2}{132} + \frac{(100-88)^2}{88} + \frac{(420-372)^2}{372} + \frac{(200-248)^2}{248}$$

$$= 51.96$$

$$\chi^2_{critical} = \chi^2_{0.05,\ v=2} = 5.991$$

Since $\chi^2_{obs} > \chi^2_{critical}$, H_0 is rejected.

Since H_0 is rejected, this means that there is insufficient evidence at 5% level of significance that the perceptions of PMDs being allowed on footpath and respondents' personal income are independent of each other.

Q2

Q2) Unsignalised Intersection

Rank / Priority	Movements
1	2, 5
2	1, 9
3	7

Movement 1 (Rank 2)

$V_{c,1} = V_5 = 480 \text{ veh/h}$

$C_{m,1} = C_{p,1} = 843 \text{ veh/h}$ //

Movement 9 (Rank 2)

$V_{c,9} = \frac{V_5}{N} = \frac{480}{2} = 240 \text{ veh/h}$

$C_{m,9} = C_{p,9} = 793 \text{ veh/h}$ //

Movement 7 (Rank 3)

$V_{c,7} = V_5 + 2V_1 + \frac{V_2}{N}$

$= 480 + 2(220) + \frac{700}{2} = 1270 \text{ veh/h}$

$C_{p,7} = 193 \text{ veh/h}$

$C_{m,7} = 193\underbrace{\left(1 - \frac{220}{843}\right)}_{P_{0,1}} = 142 \text{ veh/h}$ //

C_{SH}

$$= \frac{100 + 110}{\frac{100}{793} + \frac{110}{142}}$$

$= 233$ veh/h //

Q3

<u>Traffic Flow Models</u>

(a)

Four Limitations of Macroscopic Traffic Flow Models:

1. Deficiencies over some portion of density range
2. Tend to be problematic at boundaries
3. Inability to track well the measured field data near capacity conditions
4. Has no bearing on driver behaviour

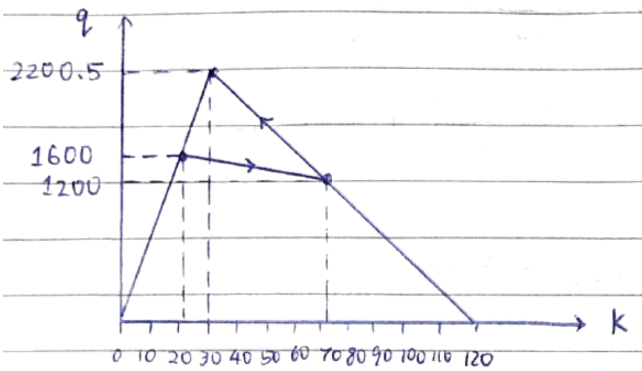
(b)

Limitations of Single-Regime Underwood's Model:

- Speed never reaches zero until jam density goes to infinity
- Model is unsatisfactory for high-density flow
- Like many other single-regime models, it exhibits the following key limitations:
  - Deficiencies over some portion of density range
  - Problematic at boundaries
  - Does not track the measured field data well, especially near capacity conditions.

(c)

Flow-Density Relationship



When $q = 1600$ veh/h

$k = 1600/73.35$

$= 21.8$ veh/km

When $q = 1200$ veh/h

$k = (1200-2934)/(-24.45)$

$= 70.9$ veh/km

Speed of forming wave $= (1600-1200)/(21.8-70.9)$

$= -8.15$ km/h

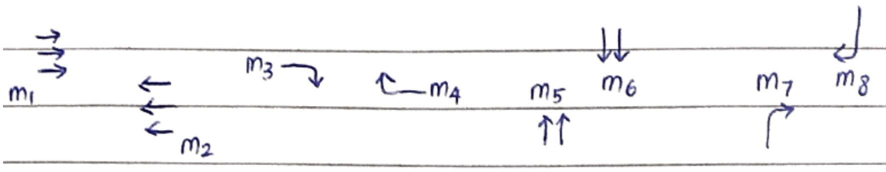
Speed of clearing wave $= (2200.5-1200)/(30-70.9)$

$= -24.5$ km/h

Q4

Signalised Intersection

Proposed Phases :



Phase A　　Phase B　　Phase C　　Phase D

Traffic volume :	Saturation Flow :	Flow Ratio (v/s)
$V_1 = 1300$	$S_1 = 525(3\times3.5) = 5512.5$	$\underline{0.236}$
$V_2 = 1245$	$S_2 = 5512.5$	0.226
$V_3 = 396$	$S_3 = 1800/1 + \frac{1.52}{16} = 1644$	$\underline{0.241}$
$V_4 = 359$	$S_4 = 1644$	0.218
$V_5 = 792$	$S_5 = 525(2\times3.5) = 3675$	0.216
$V_6 = 832$	$S_6 = 3675$	$\underline{0.226}$
$V_7 = 264$	$S_7 = 1644$	$\underline{0.161}$
$V_8 = 228$	$S_8 = 1644$	0.139
		$\Sigma y_c = 0.864$

$$L = 4(2) + 4(1) = 12s$$

$$C_o = \frac{1.5(12)+5}{1-0.864} = 169s \approx 170s$$

$$G = 170 - 4(4) = 154s$$

Allocate Green Time :

$$G_A = \frac{0.236}{0.864} \times 154 = 42.1s \quad > 25s$$

$$G_B = \frac{0.241}{0.864} \times 154 = 43.0s$$

$$G_C = \frac{0.226}{0.864} \times 154 = 40.3s \quad > 32s$$

$$G_D = \frac{0.161}{0.864} \times 154 = 28.7s$$

Discussion: The cycle time obtained is typical of a 4-phase signal control and the green time allocated is sufficient for pedestrians to cross for both minor and major road crossing.

Q5

Two-Way Segment Analysis for Two Lane Highways

Q5) Avg. Travel Speed (ATS):

$f_G = 0.99$, $f_{HV} = \frac{1}{1 + 0.18(1.5-1) + 0.2(1.1-1)} = 0.9009$

$V_p = \frac{2300}{0.99 \times 0.9009 \times 0.91} = 2834 < 3200$

Highest Directional Flow Rate = 1417 pc/h < 1700

FFS = 100 km/h

ATS = $100 - 0.0125(1417) - 0.8 = 81.5$ km/h

PTSF:

$f_G = 1.00$, $f_{HV} = 1/1 + 0.18(1-1) + 0.2(1-1) = 1$

$V_p = 2300/0.91 = 2527 < 3200$

Highest Directional Flow Rate = 1264 pc/h < 1700

BPTSF = $100(1 - e^{-0.000879(2527)}) = 89.15\%$

PTSF = $89.15 + \left(-\frac{527}{600} \times 0.7\right) + 1.8 = 89.15 + 1.185$

$= 90.3\%$

$\Rightarrow$ LOS E.

V/C ratio = 2834/3200 = 0.886

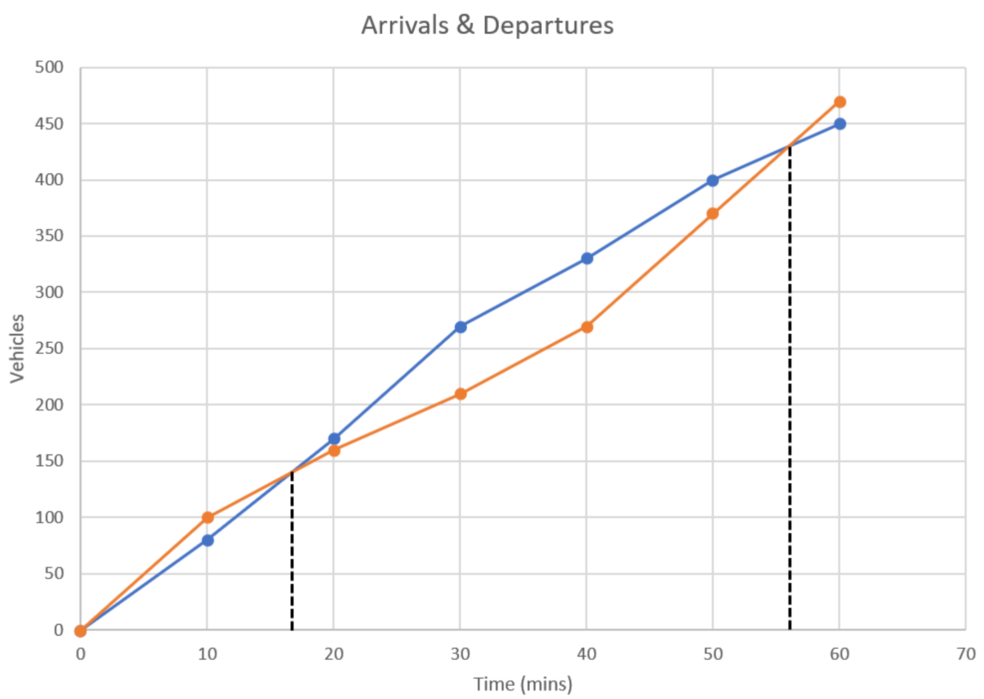
Comment: The level of service as obtained using two-way segment for the two-lane rural highway segment is E which is near the capacity conditions, as computed from the V/C ratio above.

CV4112 Traffic Engineering
Semester 1 Examinations 2022-2023
Suggested Solutions by Lim Kai Jian

Q6(a)

Time Period		10-min volume (veh/10min)	Meter Cycle (s)	Departure Rate (veh/10min)
6.30	6.40	80	6	100
6.40	6.50	90	10	60
6.50	7.00	100	12	50
7.00	7.10	60	10	60
7.10	7.20	70	6	100
7.20	7.30	50	6	100

Time	Arrival (Cumulative)	Departure (Cumulative)	Queue Length
0	0	0	0
10	80	100	-20
20	170	160	10
30	270	210	60
40	330	270	60
50	400	370	30
60	450	470	-20



$100 + 6x = 80 + 9x$

$x = 6.67$

Time the queue begins = 0640 + 6.67mins = <u>0647</u>

$370 + 10x = 400 + 5x$

$x = 6$

Time the queue ends = 0720 + 6mins = <u>0726</u>

Maximum queue length occurs at 0700 and 0710 = <u>60 vehicles</u>

Q6(b)

Inelastic Demand – Change in price has little effect on demand

Elastic Demand – Change in price has large effect on demand

Q6(c)

Congestion → Public Pressure to Increase Capacity → New Capacity → Movements become easier → Urban sprawl becomes favoured → Average length of movements increase → New Demand generated → Number of movements increase further → **More Congestion**

- END -