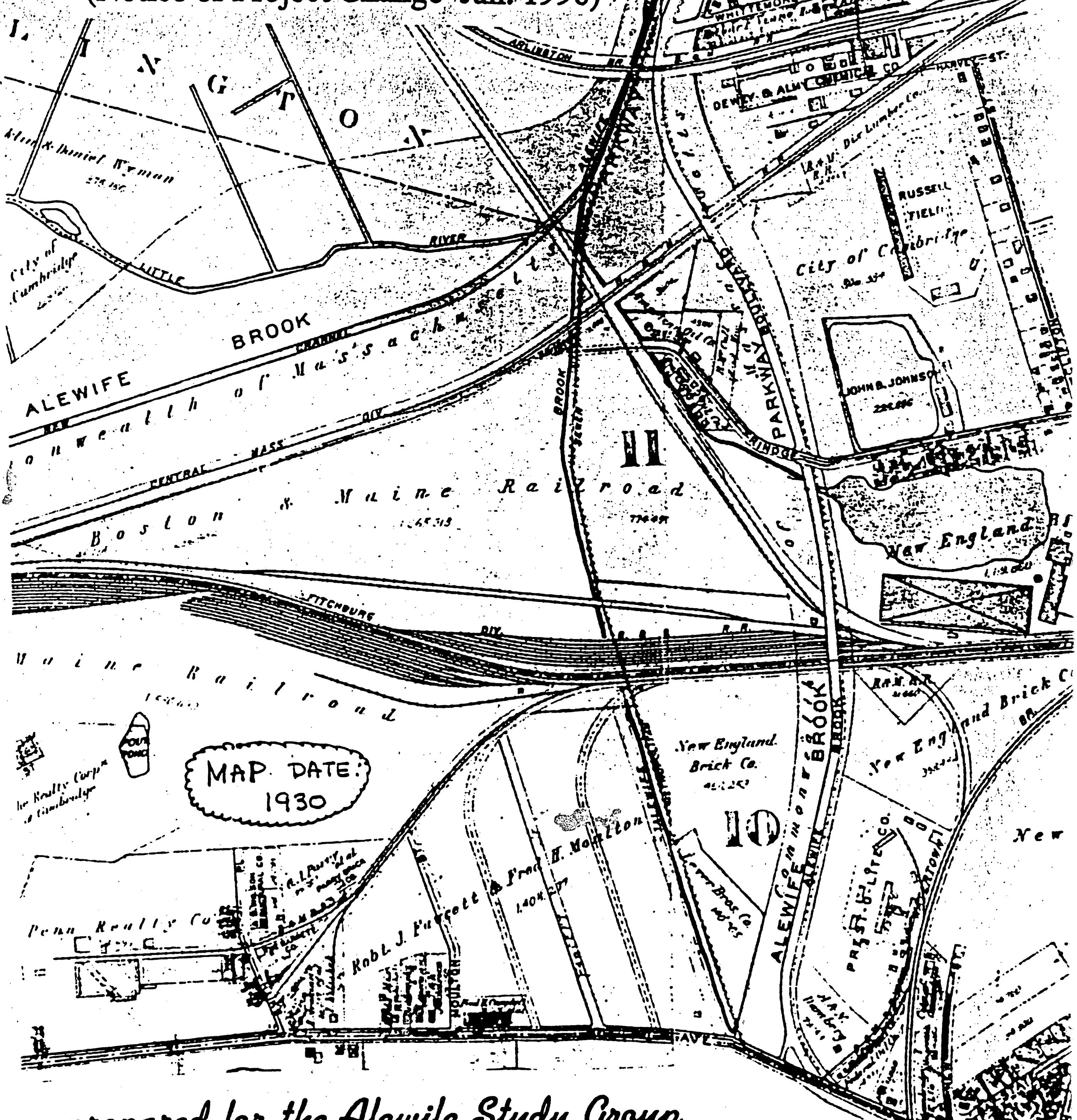
REVIEW OF TRAFFIC ISSUES

(30)

(Notice of Project Change Jan. 1996)



prepared for the Alewife Study Group

by

Stephen H. Kaiser

February 1996

Stephen H. Kaiser
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RM 105 State House
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To : Alewife Study Committee

From : Stephen H. Kaiser, traffic analyst

SUBJECT: **OUTLINE OF REVIEW COMMENTS ON ALEWIFE CENTER**

The attached report is my review of the January 15, 1996 Notice of Project Change for the Alewife Center project for traffic related impacts. I have arranged the analysis into several sections as follows :

In addition, I am preparing and can make available a set of appendix materials of a more technical and detailed nature. These eight appendices are :

A. Trip generation charts

B. Traffic counting program with DOS-FLOW

C. Lane Merges

D. Queue Lengths

E. Short Lanes

F. Technical Report : "Traffic Capacity and Safety Analysis of the Alewife Interim Access" by. S. Kaiser May 1985

G. Alewife History, major highway proposals

H. Comments on Rizzo review of the NPC 2/15/96

I. Alewife Traffic Design Report, March 1976.

Recommendations for what actions to pursue next are on pages 34-37. I urge that the developer not spend any more effort on computer models for air pollution but instead should invest in getting the traffic analysis right, while achieving the highest level of agrement among traffic professionals on the technical issues. I am prepared to work with the developer's traffic consultants to achieve this level of clarity and to report back to the Advisory Committee on those matters we can and cannot agree on relative to traffic matters.

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To : Alewife Study Group

From : Steve Kaiser

SUBJECT: **REVIEW OF TRAFFIC ANALYSIS (Notice of Project Change) FOR THE ALEWIFE CENTER PROJECT**

The January 16, 1996 Notice of Project Change for Alewife Center includes a detailed traffic analysis, comparable to the level normally associated with a state Environmental Impact Report (EIR). The document evaluates the same 14 intersections which were assessed in the 1988 Final EIR and also includes an air pollution analysis. Discussion of other issues -- the site plan, zoning, visual impacts, hazardous wastes, pedestrian access, wetlands and hydrology -- is, by contrast, surprisingly brief.

A summary comparison of the current 1996 proposal with the original plan of 1988 illustrates the similarities and differences is shown below :

	Original 1988 Plan	1996 NPC Plan
Site size	20 acres	23 acres
Parking spaces	2300 spaces	824 spaces
Number of phases	5 phases	2 phases
Total office space	850,000 s.f.	87,500 s.f.
Hotel	250 rooms	250 rooms
Retail space	50,000 s.f.	150,000 s.f.
-- Food Market	0	-- 60,000 s.f.
-- General Retail	0	-- 90,000 s.f.
Total Site development	1,050,000 s.f.	387,000 s.f.
Allowed by zoning	1,750,000 s.f.	2,000,000 s.f.
Percent of Allowed Growth	60%	19%
Route 2 Changes	Various Proposals	None

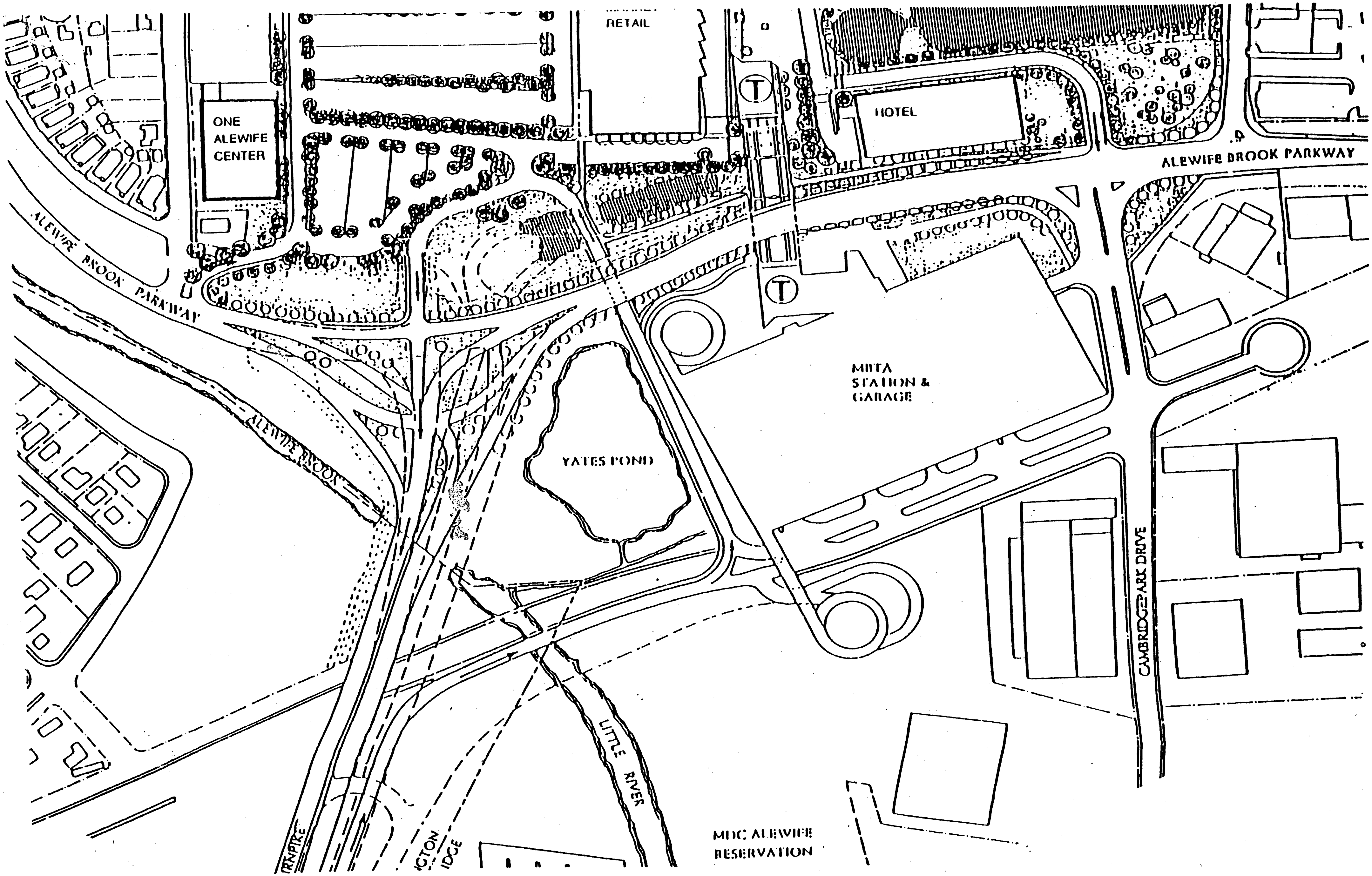
The original plan contained office and hotel uses, with only 50,000 s.f. of commercial space. The buildings would have been multi-story structured built over several levels of underground parking. Some first floor space was reserved for small retail and restaurant activities. In addition to the existing office building (87,000 s.f.), the plan called for 5 additional phases, with the hotel as part of Phase 3.

As changed, the new plan includes two restaurants, a bank, bookstore, and a health club, in addition to the primary retail tenants : a supermarket of 60,000 s.f. with Star or Stop & Shop as a likely tenant, and a 90,000 s.f. general retail store which would be similar to a small shopping center. There are only two development phases, with the hotel as phase two.

The Notice of Project Change contains a good summary of the MEPA history, including the initial 1984 waiver, the rejected 1987 Final EIR and the 1992 Section 61 finding by the state Highway Department. Table 1 below provides a more expansive historical summary of the area development around and including the Grace site. The primary historical focus of concern is whether or not there has been a significant project change between 1988 and 1996 -- enough of a change to increase environmental impacts significantly or require additional mitigation for the Alewife Center site.

Table 2 summarizes the key aspects of Alewife development and operations which have ***not*** changed in the past 10 years. Table 3 lists those elements which ***have changed*** over the past decade.

In my analysis, I have used both the 1996 NPC documents and the original EIRs of 1987-88, as well as the draft EIS for the Route 2 project (1987). In general, peak hour traffic in the past decade has not increased significantly -- expect for activity around the MBTA station and development within the triangle area.


RETAIL
ONE ALEWIFE CENTER
HOTEL
ALEWIFE BROOK PARKWAY
ALEWIFE BROOK PARKWAY
ALEWIFE BROOK
MBTA STATION & GARAGE
YATES POND
CAMBRIDGEPARK DRIVE
LITTLE RIVER
MDC ALEWIFE RESERVATION

TABLES 1 2 and 3

Table 1 Area Chronology 1929-96

1929 Alewife Brook Parkway & Bridges built
1929 Dewey & Almy Chemical Co. builds in North Cambridge
1933-34 Route 2 and rotary built as a new state highway
1948 and 1962 Highway Master Plans for Route 2 Extension
1970-72 Protests against Inner Belt & Route 2 Extension BTPR study began. Route 2 plan was shelved. MBTA announced plans for Red Line extension to Alewife
1978 Cambridge Community Development issued its master plan for Alewife (under CDD director Vickery) Plan was called the "Fish Book" – stressed new development
1980-85 MBTA Alewife station was built. Interim Access ramps and intersections built in 1985.
1984 MEPA granted a waiver for 87,000 s.f. office building at at Alewife Center, with 320 parking spaces
1986 ENF for Alewife Center, Scoping meeting, Draft EIR
1987 FEIR found inadequate by MEPA
1988 Supplemental FEIR for office & hotel was approved
1990 Final controversy over Route 2 plans. Lively protest against "Salvucci's Wall" and the plans for protests
1992 Mass Highway Department issued its Section 61 finding for Alewife Center
1993 Cambridge issues its new master plan for Alewife
1996 Notice of Project Change filed for Alewife Center

Table 2 What has *not* Changed ....

* Overall Peak hour traffic flows between Route 2 and Mass Avenue have retained fairly constant.
* 250 room hotel remains in the plan, as does existing 87,000 sf office building
* Three access points are still proposed on the Parkway side.
* There has been no updating of accident data or consideration of safety issues.
* Proposals for Alewife Boulevard and other development proposals at Alewife remain in limbo, as does the MBTA garage expansion of 1,000 spaces 2 added floors on the garage)

Table 3. What *has* Changed in the Alewife area .....

* The office development market crashed .... Banks much more reluctant to loan money on speculation.
* 1990 recession and office glut. 1988-1994 controversy over Stop & Shop and subsequent two store closures. Increased concerns over loss of community food stores.
* 1993 Alewife Master Plan issued which ignored the Alewife Center site and included no updated traffic analysis).
* Community Development determines that traffic updating is needed since
* City's last major traffic analysis effort was effort in 1985.
* Barry Pell & Rizzo Assoc hired to provide limited role, primarily review.
* Original development plans for Alewife Center abandoned. New plan developed with retail and hotel. Site area increased from 20 acres to 23 acres.
* 813,000 S.F. of Alewife Center office space will not be built
* A 60,000 s.f. food market is proposed next to the MBTA headhouse
* A 90,000 s.f. retail building is proposed for middle of the site
* A clear reduction in morning peak traffic from the site.
* Spaulding & Slye claims a reduction in afternoon peak traffic Seasonal peak traffic becomes important because of the new retail component.
* Any access difficulties (into site from the North, or heading Southbound from the site) becomes more confusing for shoppers, in contrast to regular commuters to office space.
* Access point from Whittemore has been shifted to the Parkway
* 2,300 parking spaces -- almost all in underground parking garages is replaced by 824 at-grade parking spaces (Hotel?)
* Interim Access ramp and new signals at Route 2 were completed, and Alewife garage has filled up to capacity.
* Spaulding & Slye has built several office buildings and open lots in the Triangle area.
* Outbound PM traffic flows have dropped in the Concord Avenue area as the Spaulding & Slye and MBTA garage have filled up in the triangle area.
* Major Route 2 plans have been abandoned ("Salvucci's Wall")
* Rebuilding of the railroad truss bridge by the MDC, with a median extended down to Concord Avenue
* In the afternoon peak, the traffic bottleneck has shifted from Rindge Avenue to Route 2 & Alewife.
* Minor changes to traffic signal operations at Mass. Avenue
* Mass Highway Department specified detailed mitigation in its 1992 Section 61 finding
* New 1995 traffic counts made and a new computer analysis was used to calculate traffic capacity, congestion, and queues.

SUMMARY INTRODUCTION

The focus of Spaulding & Slye's position is that no supplemental EIR should be required and that no significant changes are needed in the Section 61 finding. Their claim is that .....

"The traffic impacts of the project change are lower than to (sic) the original project during the weekday evening peak hour and substantially lower during the weekday morning peak hour. On Saturdays when the proposed project generates its highest volumes, background traffic volumes are much lower so that the Levels of Service are better than the weekday evening peak which has been used as the basis for the traffic analysis. Therefore, no changes are proposed to be made in the Section 61 Finding"

except for the description of the project.

In effect, the developer is claiming that the traffic impacts (and peak hour trip generation) are less than the original EIR -- so there is no need for MEPA to reconsider their approval of the 1988 EIR. The only numerical traffic comparison between the two documents is for total trip generation (pages 8-10 to 8-11 and Figure 8-6) :

	Original 1988 Plan (EIR)	Current 1996 Plan (NPC)	Difference
Morning Peak Hour Weekday Car Trips	1302	327	- 965
Afternoon Peak Hour Weekday Car Trips	1209	861	- 348
Average Daily Traffic	8,000	8,933	+ 933

The entire basis for claiming that the traffic impacts of the new plan are less than the old are based on simple trip generation during the weekday afternoon peak hour and the claim that on Saturdays the congestion will be less than on a weekday.

The following review of the Notice of Project Change (NPC) is composed of the following sections :

(1) TRAFFIC COUNTS
(2) YEAR 2000 NO BUILD CASE
(3) ACCESS POINTS
(4) TRIP GENERATION
(5) YEAR 2000 BUILD VOLUMES
(6) INTERSECTION CAPACITY ANALYSIS and MITIGATION
(7) QUEUE LENGTHS

(8) TRIP DIVERSIONS TO OTHER ROUTES
(9) PEDESTRIAN AND SAFETY ISSUES
(10) MITIGATION PROPOSALS
(11) PARKING REQUIREMENTS
(12) ZONING AND GROWTH POTENTIAL
(13) SECTION 61 FINDING
(14) AIR POLLUTION MODELING
(15) CONSTRUCTION STAGING
(16) ISSUES IN THE MEPA FINDINGS

SUMMARY CONCLUSIONS OF MY REVIEW OF THE NPC

(1) ... that the existing 1996 traffic conditions have not changed significantly since 1988.

(2) ... that -- with the proposed retail development -- the morning peak hour traffic impacts are less than with office development.

(3) ... that the afternoon peak hour traffic impacts for retail and for office are about the same : 1226 for the new plan, 1209 for the old.

(4) ... that the Saturday afternoon traffic impacts appear to be less than a weekday, but more analysis and review time is necessary.

(5) ... that the capacity analysis for several intersections (Route 2/Alewife Brook Parkway, the Concord Rotaries) should be substantially revised to reflect actual traffic conditions.

(6) ... that the driveway on the outbound ramp requires much more thorough analysis for capacity, queuing and safety considerations, as well as issues of signalization and officer control.

(7) ... that the proposed mitigation is either ineffective (Mass. Ave & Alewife), non-existent (Route 2/Alewife or contrary to neighborhood interests (Huron Avenue and Sherman Streets).

(8) ... that parking may be insufficient and may cause spillover problems for busy seasonal days. Traffic circulation within the parking lots also warrants more design and analysis. About 824 spaces are proposed, when about 1,000 spaces may be needed for all land uses.

(9) ... that considerable growth potential remains on the site, so the current proposal should be considered as only part of an ultimate site development which could generate higher levels of traffic.

(10) ... that the absence of any proposals for Route 2 capacity increases and the failure to propose effective local traffic mitigation is a significant change from 1988, when many variations on traffic changes were being proposed.

INTRODUCTION

The basic claim of Spaulding & Slye is that the project develops fewer vehicle trips in the AM and PM peak hours than the original office plan of 1988. Moreover, they claim that for Saturdays, site traffic will increase but will not be worse than a typical weekday because Parkway traffic is less on a weekend. The two claims are not consistent because the weekday case is based on a comparison with the 1988 plan, while the Saturday condition is compared to a 1995 weekday -- not to the original plan.

If the developer's position is not consistent, it is reasonable to ask how we would compare the new and the old plans for overall congestion. Increased traffic and congestion on a Saturday could be balanced off against lesser congestion on weekday mornings. However, the prospect for mitigation is also important, so if an identical project were proposal ten years apart, yet the prospects for mitigation are much less today, then the likely impacts of the project would be worse, and everyone should recognize this change.

The Notice of Project Change asserts that :

"The traffic impacts of the project change are lower than to (sic) the original project during the weekday evening peak hour and substantially lower during the weekday morning peak hour. On Saturdays when the proposed project generates its highest volumes, background traffic volumes are much lower so that the Levels of Service are better than the weekday evening peak which has been used as the basis for the traffic analysis. Therefore, no changes are proposed to be made in the Section 61 Finding"

except for the description of the project. The developer proposes to draft a revised Section 61 finding for submission to the Mass Highway Department.

While the developer is claiming that the traffic impacts (and peak hour trip generation) are less than the original EIR, the Notice of Project Change makes no reference to congestion levels or Levels of Service for the 1988 EIR and there is no comparison of street traffic conditions.

Any analysis which seeks to establish a claim to the significance of traffic results must be subject to the full and accurate requirements for analysis outlined in Table 4. In their analysis, Abend Associates have sought to address 10 of the 16 items below – with no discussion of (7) queue lengths, (8) potential for trip diversion to other routes, (9) pedestrians and safety, and (10) parking requirements, (12) zoning and growth, and (15) construction staging. The 10 remaining items must also be evaluated for accuracy and completeness.

Table 4 ANALYSIS REQUIREMENTS

1. Make traffic counts -- weekday peak hour and Saturdays
   Assemble the counts and balance the network for existing 1995 counts

2. Describe the future No-Build conditions
   Assemble the year 2000 No-Build volumes

3. Present Site plan and show all access points, describe intersection operations in future case

4. Calculate Trip Generation from the site, including pass-by volumes

5. Assemble the traffic volumes for the year 2000 BUILD case

6. Run all traffic volumes through the CINCH traffic model -- for signalized intersection, rotaries and unsignalized locations
   Determine the levels of service

7. Describe the queue lengths and resulting congestion

8. Discuss potential for trip diversion to other routes.

9. Assess pedestrian circulation and safety implications of project

10. Assess adequacy of proposed parking

11. Offer practical and effective mitigation for capacity and safety

12. Discuss Zoning and expansion possibilities

13. Section 61 findings and mitigation commitments

14. Air Pollution modeling

15. Construction staging issues

16. Implications for the MEPA Process

TRAFFIC COUNTS

The traffic counts have been included in the Appendix Volume and cover the 4-6 PM weekday and mid-Saturday for all 14 locations. Counts were made for only one day at each location during mid-October 1995. It would have been preferable to have had at least two days of counts at each location. In some cases, the final 15 minutes of the 5-6 PM peak hour was not counted. Although the text does not specify it, the weekday PM peak hour appears to be 5 to 6 PM.

Good feature : the pedestrian flows were measured at both Huron and Mass. Avenue, as 3 % and 1 % respectively of the total auto flow through the intersection.

.... GROWTH AND DECLINE IN TRAFFIC VOLUMES

Generally, traffic volumes have not changed significantly in the past 10 years. Exceptions are a drop in cars on Lake Street since the traffic signals were installed in the last 1980s, and a decline in outbound parkway volumes on the Parkway and Concord Avenue, while activity in the triangle area, especially the MBTA station has increased.

A comparison of 1995 counts with 1985 (the last year of the rotary) show that the new signalized intersection actually carries fewer peak hour vehicles than did the original Route 2 rotary. Indeed, the rotary could have handled more, since before the MBTA station it was not the bottleneck on the Parkway and therefore was not fully loaded to capacity.

.... ACCURACY OF COUNT TRANSCRIPTION AND CHARTS

The only processing error in the traffic volume diagrams I could find was the left turn from the MBTA station, which should have been 400 for the existing weekday PM, not 100. The correct volume was used in the capacity analysis, however.

All relevant counts appear to be included in the technical appendix – except for the important Route 2 loop ramp and the existing site driveway on the ramp.

The counting charts also show stray counts appearing, such as 10 right turns shown in page A2-4 and 14 other extraneous turns on page A2-6. Were these errors made in the field during the counts? Or were they generated by recording equipment?

.... CALCULATION OF PEAK HOUR FACTORS

The data show wide variation in sequential 15 minute counts. For example, on page A2-6 at Alewife and Route 2, the left turn from the south has an 1723 hourly volume but achieves 522 cars in the peak 15-minute period -- which yields a peak hour factor of 0.83. However, the CINCH capacity analysis uses a peak hour factor of 0.95 everywhere, regardless of variations suggested by the actual counts. Normally, traffic count data would be used to generate the peak hour factors for analysis, but the readily available traffic count data in the NPC does not appear to have been used and instead a simple 0.95 was assumed everywhere. What was the explanation for this decision?

.... BALANCED TRAFFIC FLOWS IN THE NETWORK

The individual counts must first be assembled into a network diagram such as Figures 8-3 and 8-4. Next the different counts must be "balanced" so that there are not unexplained increases or decreases in flows between intersections. This process appears to have been applied effectively from Mass Avenue to Rindge Avenue, but along Concord Avenue some of the flows show inconsistencies.

To be fair, the complex nature and change in traffic flows at the two Concord Avenue rotaries makes traffic counting very difficult. It does appear that both rotaries were undercounted -- at least in the northbound direction. Between Huron Avenue and Concord Avenue, northbound flows on weekdays show an unexplained drop of 300-400 cars (from 1500 to 1170) when one would have expected an increase due to the traffic entering from Lake View Avenue/Vassal Lane. Southbound traffic in the opposite direction drops from 1800 to 1635 at Huron Avenue, an unexplained loss of 165 vehicles. (Today very few left turns off the parkway).

Between Concord Avenue and Rindge Avenue, the northbound volumes rise from 1350 to 1750, and some of this growth may be accounted for by vehicles entering from the shopping center driveway. However, the net difference of 400 vehicles does appear on the high side.

YEAR 2000 AFTERNOON PEAK -- NO BUILD

The year 2000 No-Build case appears to have been well constructed -- and includes the Aku-Aku site at 143 Alewife Brook Parkway for 50,000 s.f of retail use. This project is now before the Cambridge Conservation Commission for various approvals.

The magnitude of background growth assumed in the NPC appears sufficient to cover any seasonal or Friday peaking in background traffic.

ACCESS POINTS

The site plan (Figure 3.1) shows four locations for vehicle access.

1. On the outbound Route 2 ramp (2-way, right-turn-in, right-turn-out) Possible need for officer control *[Changed from original plan]*

2. To the MBTA station access road, at the old Lehigh Metals site. (2-way, all turns in and out allowed except the left turn in from the parkway would be banned) *[Same as 1988 plan]*

3. Near Whittemore Avenue (one-way only into the site, accessible only from the Northbound Parkway and Route 2) *[Changed from 1988 plan]*

4. Harvey Street (shown on the site plan but assigned no traffic. Presumed to be emergency-access only, with gate restrictions. *[Changed from 1988 plan]*

.... ROUTE 2 RAMP DRIVEWAY

This driveway would be a single curbcut in the new plan, whereas the 1988 plan had two driveways spaced about 50 feet apart. The new single driveway is shifted about 50 feet to the north away from its present location. The new driveway would be located around the curve of the ramp and would be difficult for drivers to see as they approached through the tunnel underpass. The Section 61 finding of 1992 appears to presume that a police officer will be controlling this driveway, and the poor sightlines could present a visibility problem and potential safety hazard. (As noted later, a traffic signal should be considered for this driveway location).

.... MBTA STATION ACCESS

As currently constructed, the southbound Parkway at the MBTA access road may have enough room for a vehicle to stop in the median area and attempt an illegal left turn into the hotel access road. Mitigation should be considered to construct a raised median to prevent such turns -- as is now in place at Rindge Avenue. The parkway has inadequate width to permit a formal left turn storage lane at this location. The prohibition of this left turn and clear alternate routing of traffic should be an explicit conditions of any Section 61 finding.

Access to the site from the North could be quite confusing for arriving drivers, since they cannot use the south driveway. Instead, they must drive in a clover-leaf loop around the MBTA garage to enter the mall site. This loop will require clear signage for new retail and hotel patrons.

The south access must serve all vehicles leaving the site to go South towards Concord Avenue. Internal signing will be important to prevent any vehicles (such as southbound pass-by traffic) which would enter the site from the ramp and then be required to use another exit point to continue south. If they sought to re-enter the Parkway at the driveway ramp for Route 2 they would face a no-left turn situation and the possibility of illegal left turns within the awkward merging area of the Route 2/Alewife intersection.

.... WHITTEMORE AVENUE ACCESS

Access to Whittemore Avenue has been a controversial issue for many years. The current proposal is new and provides a direct connection from the parkway, with no connection to Whittemore Avenue. The result is two access roads placed side by-side and the potential for weaving motion between drivers seeking an alternate route into the site from Route 2 -- by weaving across northbound parkway traffic.

Under this new plan, traffic on Whittemore Avenue would be cut in half -- from 215 to about 100. Today, almost all the exiting vehicles from the parking lot use the Whittemore exit, with its convenient officer assistance, thereby avoiding the conflicts on the ramp and in the merge. If Grace were to further reduce their activities at the site, it is possible for less than 50 cars an hour to be using Whittemore, and the traffic officer now assigned here could be relocated to the more active area of the ramp driveway. With no officer at Whittemore, i violations of the "DO NOT ENTER" 4-6 PM regulation on Whittemore Avenue could increase, as traffic seeks to bypass the long queues from Mass Avenue (as they once did in the past).

The proposed access road into Alewife Center could be subject to illegal use (such as going the wrong way to exit from the parking lot onto the Parkway) at those times when an officer is not on paid detail at Whittemore Avenue. Moreover, with the access road in place and congestion at Alewife, it would appear a simple matter for the developers to petition the City and MDC to reverse the road and make it one-way out or widen it to two-way -- with access from both Route 2 and the Parkway. Signalization of this new Whittemore Avenue intersection with turning lanes would be possible additions at some future time. None of these features are consistent with the goal of creating a parkway/neighborhood environment in the area.

The projected traffic use of the connector road as an entrance is very small. The NPC shows estimates of only 18 cars an hour on the connector on a weekday and 24 on Saturdays. These low volumes do not demonstrate a private or a public need for the access road and do not justify the risk to the parkway created by making this ramp connection. The Whittemore ramp should be entirely deleted from the proposal.

.... HARVEY STREET CONNECTION

The connection to Harvey Street is shown on the plan but no traffic is assigned to it. I presume it is controlled by a gate and would be used only by emergency vehicles and pedestrians. Any regular opening of the Harvey Street connector into the community would result in considerable cut-through traffic from Route 2 into the North Cambridge community.

Discussion of circulation within the parking lot will be covered in the Section 11 on parking.

TRIP GENERATION

The number of trips in the peak hour is a very significant factor in overall traffic impact. The method of calculating the number of trips is to refer to sources such as the Institute of Transportation Engineer's "Trip Generation," or known counts of similar land uses in the Boston area.

The Notice of Project Change chooses to represent the traffic impacts of the site by considering an *average* day during the year. Such an assumption is a reasonable approach for most land uses such as office, residential, restaurants and small retail establishments. However, there are obvious and well-known increases in traffic activity at larger retail sites during seasonal peaks, such as the Christmas shopping period, so that it is inappropriate to use the average rate for shopping centers. Similarly, variations occur within the 5 weekdays, with Fridays generating more traffic than other days. Supermarkets and hotels do not have documented seasonal peaks.

.... SUPERMARKET

The ITE rate for supermarkets is 12.4 cars per 1,000 s.f. in the average weekday afternoon peak and the NPC uses a slightly higher figure of 12.95. This slightly higher rate could account for any daily or seasonal variations, but whether or not it can include other proposed land uses -- bank, florist, pharmacy and 2 restaurants -- is questionable. I agree with the base figure

of 777 trips generated in the peak hour (supermarket, unadjusted).
Why was the large restaurant not specifically included in the trip generation? Individual land uses such as banks and restaurants are listed included in the ITE Land Use categories and should have been applied.

.... GENERAL RETAIL (SHOPPING MALL)

The NPC uses average ITE data for Shopping Malls (Land Use 820), as presented on page 1237 of the ITE Trip Generation manual. (The NPC appendix omitted this page, as well as pages 1232 and 1233, but see attached Appendix A)

The NPC documents include a strange method of calculating the traffic for a 90,000 s.f. mall by subtracting the difference between a 150,000 sf and a 60,000 s.f. mall. This calculation produces an afternoon peak hour trip generation of only 375 trips, whereas a direct calculation for a 90,000 free-standing mall would be 615 trips, a difference of an additional 240 trips. The NPC should have used a direct calculation for a 90,000 s.f. mall and should have used a figure of 615 vehicle trips an hour as the ***average weekday unadjusted*** trip generation level.

...........SEASONAL AND WEEKDAY PEAKS AT ALEWIFE RETAIL

Highway designers have commonly attempted to design for the 10th to 50th highest traffic hour of the year, and the Highway Capacity Manual recommends the 10 to 20th highest hour for urban highways, and the 30th highest hour for rural areas (HCM, p. 2-10). The 30th highest hour represents about the highest 3 % of all commuting hours, which appears to be a good compromise for the Alewife area.

Traditionally, shopping mall industry procedures have considered traffic flows for the average day, a busy "design day" and the "peak day." Typically, a design day would have weekday traffic 33 % higher than average, while the peak day would be 66 % higher than average. The design day is intermediate between the average and peak conditions. In recent years, EIRs and the MEPA Guidelines have specified use of the ITE Trip Generation tables, and through general lack of specification the average day figures have sometimes been used by mall developers. In the case of the MEPA scope for the Kingston Mall (EOEA #6600) and at least a dozen others, MEPA required seasonal adjustments for retail developments.

Selection of the design day is critical to a retail project. For parking and internal operations, the ultimate design day is the day after Thanksgiving, which is the busiest shopping day of the year. However, external traffic on this Friday is quite lower because it is a holiday.

Overall, a good functional design day is represented by a busy day in the Christmas season. December is the busiest month of the year for retail and Friday is the busiest day of the week. The combination of a busy Thursday and Friday in December would constitute the 3% possible event.

The ITE Trip Manual contains conflicting data regarding Christmas season trip generation. The fitted curve equation for the average day (Land Use 820, p. 1237) and Shopping center, Christmas season (p. 1244) actually predict 57 fewer peak hour trips during the Christmas peak, than on an average day for a 90,000 s.f. mall. However, the base data includes no shopping centers with less than 100,000 s.f., and only 5 samples around 100,000 s.f. The figures predicting a drop in traffic during the Christmas season contradict all practical experience. The application of simple straight line rates to the ITE data has an even more extreme effect : only 338 vehicle trips in the Christmas peak and 615 trips on an average day. A severe sampling error has occurred in the ITE data.

These results are contradicted by data in ITE Table 5, which indicates that December shopping mall trips are 41.8% higher than average -- based on a sample of only two studies, with the malls having an average size of 938,000 s.f. Earlier editions of the Trip Generation manual (1976, 1979, 1982) had shown December traffic to be 50% higher than the average, with dollar sales double the average month.

Table 7 MONTHLY TRAFFIC VARIATIONS at SHOPPING CENTERS

From ITE *Trip Generation"*	(1976,79,82)	(ITE 1991)
JANUARY	70%	85.3%
FEBRUARY	60%	78.1%
MARCH	100%	92.0%
APRIL	90%	93.2%
MAY	110%	105.4%
JUNE	110%	106.0%
JULY	103%	100.8%
AUGUST	100%	102.1%
SEPTEMBER	95%	94.8%
OCTOBER	115%	98.9%
NOVEMBER	105%	101.5%
DECEMBER	150%	141.8%

The 90,000 s.f. mall proposed for Alewife is too small to appear on the Christmas season graph (p. 1244) and is also only 1/10 the size of the malls sampled for Table 5. The first method produces the illogical result of lower mall traffic during December, while the other suggests a 42% increase. Some evidence suggests the seasonal peaking is somewhat less in smaller malls, so the reasonable compromise is to assume a 20% Christmas season peaking factor for December at the 90,000 s.f. Alewife shopping center.

Weekday peaking on Thursdays and Fridays should be based on Table 4 of *Trip Generation* (5th Edition) for shopping centers under 100,000 s.f. -- all as a percentage of the average weekday ITE rate :

SUNDAY	45.2 %
MONDAY	97.3 %
TUESDAY	92.9 %
WEDNESDAY	92.7 %
THURSDAY	98.2 %
FRIDAY	118.9 %
SATURDAY	125.5 %

The combination of Thursday and Friday as a peak results in a weekday peak adjustment of 110.2 %. This factor would be combined with a Christmas seasonal adjustment of 120 %.

..... PASS-BY TRIPS FOR SHOPPING CENTERS

The matter of "pass-by" trips has generated much controversy. The basic concept is that the total number of vehicles passing through the driveways stays the same, but when a driver already on the road uses the mall, this is not a *new* trip on the highway system. This driver was "passing by" and therefore does not add to the traffic on the area roadways. The amount of new traffic added to the area intersections is the amount of traffic using the driveways ..... subtracting off the amount of pass-by traffic, which the NPC assumes to be 25 % of the total.

The pass-by adjustment is significant because it means that only 75 % of the mall traffic is add-on traffic to other roads. The 1991 edition of the Trip Generation manual includes a lengthy discussion of pass-by trips, including numerous surveys with varying results ranging from 8 % pass-by to 82 % pass-by. Unfortunately, these results cannot be measured directly but are instead the result of conducting *surveys*. Almost all of the surveys were conducted by traffic consultants for shopping mall developers, who have a special interest in showing reduced traffic growth in their traffic studies.

The only surveys conducted by a public agency show 4 sites with pass-by percentages from 8 % to 22 % (Connecticut Department of Transportation). Another from Calgary officials in Canada shows a 25 % pass-by.
The Connecticut data suggests only a 15 % pass-by rate, but there is evidence that the percentage increases for smaller stores, so a figure of 20 % appears reasonable to the Alewife retail center. Supermarkets tend to attract more planned pass-by trips, so the pass-by percentage for this review is 25 %, the same as used in the NPC.

.... HOTEL

The trip generation for the 250 room hotel is fairly straightforward, although the ITE sheet for this use was omitted from the NPC appendix. In the peak hour the NPC estimates 183 trips in the peak hour and I agree. However, the NPC applied the same 25% pass-by figure of the hotel as it did to retail, which is not appropriate. I have assumed a 0% pass-by percentage for the hotel, as was done in the original 1988 EIR for Alewife Center.

.... MODAL DISTRIBUTION OF TRIPS

The NPC assumes generally that transit, walking and bikes will provide 25% of the trips to the site. The selected percentage of pedestrians and transit users appears somewhat arbitrary and is not explained adequately in the NPC. On page 8.14 the report does admit that walking trip percentages will be lower in the PM peak, yet the same 10% figure for peds and bikes was used for daily and peak hour. Therefore, this review has used a 20% transit/ped/bike percentage in the peak hour, rather than 25%.

.... TRIP GENERATION SUMMARY

The total trips generated after allowance for seasonal and daily adjustments plus pass-by traffic in the weekday afternoon peak is 1226, compared to 1209 for the original 1988 plan and 861 as claimed in the NPC. Total vehicles through all driveways would remain 1209 vehicles an hour under the 1988 plan, but would rise to 1103 for the NPC case and 1506 with seasonal and other adjustments.

Table 6 below summarizes the adequacy of the NPC documentation in calculating the peak hour trips for the three primary land uses, including seasonal adjustments.

TABLE 6 ADEQUACY OF NPC FOR TRIP GENERATION ;

	SUPERMKT	RETAIL	HOTEL	TOTAL
1. Estimate trips for an average day	YES	NO	NO	NO
2. Make a seasonal adjustment for Xmas	YES	NO	YES	NO
3. Make a weekday adjustment for Fridays	YES	NO	YES	NO
4. Pass-by Reduction	YES	NO	NO	NO
5. Allow for Trucks & Employees	YES	YES	YES	YES
6. Allow for proper non-auto trips (transit, pedestrian, bike)	NO	NO	YES	NO

TABLE 6A

TRAFFIC GROWTH : ALEWIFE AREA 1985-95 sk 2/3/96 ALEWVOLS.W62

AFTERNOON PEAK HOUR		NORTH	EAST	SOUTH	WEST	TOTAL	CHANGE
1 ALEWIFE AND MASS	-1985	1001	954	1682	678	4315	
	-1995	995	1130	1555	630	4310	-0.1%
2 ALEWIFE & WHITTEMORE	-1985	1107	134	1718	0	2959	
	-1995	1200	215	1549	0	2964	0.2%
3 ALEWIFE & ROUTE 2	-1985	1171	0	2929	2529	6629	
	-1995	1390	935	2210	2380	6915	4.1%
						6330	
4 ALEWIFE & MBTA STATION	-1985	1971	0	2589	643	5203	
	-1995	1545	0	1910	410	3865	-34.6%
5 ROUTE 2 ARL/CAMBR	-1985	0	2960	0	2529	5489	
	-1995	0	3245	0	2380	5625	2.4%
6 ALEWIFE & RINDGE	-1985	2045	569	2436	0	5050	
	-1995	1810	545	1750	0	4105	-23.0%
7 ALEWIFE & CONCORD	-1985	2200	2626	0	1075	5901	
	-1995	2040	1410	0	1045	4495	-31.3%
8 FRESH PD & CONCORD	-1985	0	671	2360	2708	5739	
	-1995	65	380	1170	2478	4093	-40.2%
9 FRESH POND & HURON	-1985	1643	335	1443	549	3970	
	-1995	1635	475	1415	615	4140	4.1%
10 SHERMAN & RINDGE	-1985	0	492	487	319	1298	
	-1995	0	440	670	310	1420	8.6%
				27.3%			
11 CONCORD & BLANCHARD	-1985	425	1164	494	404	2487	
	-1995	495	1110	540	330	2475	-0.5%
12 BRIGHTON & CROSS	-1985	305	785	391	309	1790	
	-1995	385	330	1000	345	2060	13.1%
13 LAKE & RT.2 INBOUND	-1985	317	820	935	0	2072	
	-1995	375	635	540	0	1550	-33.7%
14 LAKE & RT.2 OUTBOUND	-1985	987	136	1523	0	2646	
	-1995	510	190	1005	0	1705	-55.2%
15 LAKE & MASS AVE.	-1985	0	1074	669	638	2381	
	-1995	0	860	640	705	2205	-8.0%

SOURCES:
ALEWIFE CENTER FEIR #5869 TECH APPENDIX November 1987
ALEWIFE CENTER Notice of Project Change Vol 1 Fig. 8-2 Jan 1996

TABLE 7 Alewife Center Trip Generation Calculations

```
EWIFE CENTER TRIP GENERATION ANALYSIS   SK ACTR-GEN.W62 2-5-96
Weekday Afternoon Peak Hour Trips to & from Supermarket
              (Trip Generation,ITE, 1991 Edition  p. 1237)
SupMkt. Auto Trips during Peak Hour   Method 1 = ITE rate
 Size   Method 1 Method 2             Method 2 = ITE Ln Formula
 KSF     (ITE)    (ITE)   (NPC)   (SK)          Auto Trips at
Driveway
 X         T        T                           = IN = OUT =
 60 **    718      743     777     777            48%  52%
Average Day Totals         777     777
=====================================================
Christmas Season -- (for Supermarkets not mentioned in ITE)
=====================================================
Friday Adjustment -- (for Supermarkets not mentioned in ITE)
Christmas Friday Totals    777     777
=====================================================
Person Trips (Peak Hour)   971     971  SUPERMARKET
Employee Trips              14      14
Employees by car             4       4
Employee Car Trips           4       3
Customer Trips             958     958
Customer by Car %          75%     80%    Auto Trips at Driveway
Customer Trips by Car      718     766       = IN = OUT =
Customer Car Trips (NPC)   575                 276  299
   At Driveways    (SK)            613         294  319
Customer & Employee Car    579                 278  301
   Trips at Driveways              616         296  320
Pass-By Percentage         25%     25%
New Customer Car Trips +   431     460
New Customer & ... (NPC)   435                 209  226
 Employee Cars .... (SK)           463         222  241
     Final Total at Driveways = 38 more trips for Supermarket
   Final Total, less Pass-bys = 28 more trips for Supermarket
```

```
Average Weekday Afternoon Peak Hour Trips to & from Retail Mall
              (Trip Generation,ITE, 1991 Edition  p. 1237)
Retail  Auto Trips during Peak Hour, In or Out
 KSF     (ITE)            (NPC)   (SK)     Auto Trips at Driveway
  X        T              trips   trips       = IN = OUT =
-----    -----            -----   -----       ----- -----
 150  **  850
  60  **  474
150-60    376              375                 188  188
  90  **  614                     614          307  307
Average Day Totals         375    614  239 more trips for retail
Christmas Season (p. 1244,1233, ITE)   Method 1= ITE Ln Formula
   Auto Trips during Peak Hr. In/Out Method 2= 41.8% Increase
   ------- ------- ------- ------ SK Method = 20% Increase
Retail  Method 1 Method 2
 KSF     (ITE)    (ITE)   (NPC)   (SK)
  X        T        T
 150  **  791     1205            -59 fewer trips in Xmas Season
  60  **  421      672            -53 fewer trips in Xmas Season
150-60    370      533     375     -6  fewer    188  188 in/out
  90  **  557      870            736  -57 fewer 368  368 in/out
Assumed Christmas Effects :
        -9.3%   +41.8%   + 0%   + 20%
Christmas Season Totals    375    736 +361 more trips for retail
Friday Adjustment   (p. 1233 ITE, Table 4)
 150  **  941     1432            150 more trips in Xmas Season
  60  **  501      799             80
150-60    440      633     375         70       188  188
  90  **  662     1035            811  105      406  406
Assumed Friday Effects :                      = IN = OUT =
        +18.9%   +18.9%   + 0% +10.2%
Christmas Friday Totals    375    811 +436 more trips for retail
                          (NPC)   (SK)
Person Trips               469    1014  RETAIL
Employee Trips              20      20
Employees by car             6       6
Employee Car Trips           6       5
Customer Trips             449     994     Auto Trips at Driveway
Customer by Car %          75%     80%        = IN = OUT =
Customer Trips by Car      336     795
Customer Car Trips (NPC)   269                 135  135
   At Driveways    (SK)            636         318  318
Customer & Employee Car    275                 138  138
   Trips at Driveways              641         321  321
Pass-By Percentage         25%     20%
New Customer Car Trips +   202     509
Customer & ....... (NPC)   208                 104  104
 Employee Cars .... (SK)           514         257  257
     Final Total at Driveways = 366 more trips for retail
   Final Total, less Pass-bys = 306 more trips for retail
```

```
Weekday Afternoon Peak Hour Trips to & from 250-room Hotel
              (p. 523 ITE)
 Hotel  Auto Trips during Peak Hour, In or Out
 Size   Method 1 Method 2
#Rooms   (ITE)    (ITE)   (NPC)   (SK)            Auto Trips at
Driveway
 X         T        T                            = IN = OUT =
 250 **   184      190     183     184             54%  46%
Average Day Totals         183     184
=====================================================
Christmas Season -- (for Hotels not mentioned in ITE)
=====================================================
Friday Adjustment -- (for Hotels not mentioned in ITE)
Christmas Friday Totals    183     184
=====================================================
Person Trips (Peak Hour)   229     230  HOTEL
Employee Trips              20      20
Employees by car             6       6
Employee Car Trips           6       5
Customer Trips             209     210
Customer by Car %          75%     75%
Customer Trips by Car      156     157
Customer Car Trips (NPC)   125                       68   58
   At Driveways    (SK)            126               68   58
Customer & Employee Car    131                       71   60
   Trips at Driveways              131               71   60
Pass-By Percentage         25%      0%
New Customer Car Trips +    94     126
New Customer & ... (NPC)   100                       54   46
 Employee Cars .... (SK)           131               71   60
     Final Total at Driveways =   0 more trips for hotel
   Final Total, less Pass-bys =  31 more trips for hotel
```

TOTALS FOR ALL LAND USES

```
                              (NPC)  (SK)  Increase
Total Person Trips to Site     1669  2216   547
Total Customer Trips           1615  2162   547
Total Customer Trips by Car    1211  1719   508
Total Customer Veh Trips        969  1375   406
Total Employee Veh Trips         16    13              = IN = OUT =
Total Vehicle Trips to Site     985  1388   403          687  701
Total New Veh on Roads          743  1107   365          550  558
Total Pass-By Traffic           242   280    38          137  143
Existing Office Space 87 ksf    118   118     0
Total Project New Trips with All Phases
                                861  1226   365
Total Trip Generation from Site
                               1103  1506   403
Total Trips Generated by Original Proposal
                               1209  1209     0
Change in Number of New Trips
                               -348    17  more new trips .....
```

TRIP DISTRIBUTION

```
                                                 Pass-By
                     (NPC)  (SK)  (NPC) (SK)     Both
PM TRIPS INTO SITE   100%   100%   485  687      137
PM TRIPS OUT OF SITE 100%   100%   501  701       -
---- IN SITE
 ... Whittemore Avenue
PERCENT FROM NORTH     0      0      0    0        0
PERCENT FROM WEST      0      0      0    0        0
PERCENT FROM SOUTH     5      5     24   34       -7
 ... Route 2 Ramp
PERCENT FROM NORTH    25     25    121  172      -34
PERCENT FROM WEST     35     35    170  240      -48
PERCENT FROM SOUTH     0      0      0    0        0
 ... MBTA Station
PERCENT FROM NORTH     0      0      0    0        0
PERCENT FROM WEST      0      0      0    0        0
PERCENT FROM SOUTH    35     35    170  240      -48

--- LEAVING SITE
 ... Whittemore Avenue
PERCENT TO NORTH       0      0      0    0
PERCENT TO WEST        0      0      0    0
PERCENT TO SOUTH       0      0      0    0
 ... Route 2 Ramp
PERCENT TO NORTH      35     35    175  245
PERCENT TO WEST       35     35    175  245
PERCENT TO SOUTH       0      0      0    0
 ... MBTA Station
PERCENT TO NORTH       0      0      0    0
PERCENT TO WEST        0      0      0    0
PERCENT TO SOUTH      30     30    150  210
```

CONCLUSIONS and DISCUSSION OF TOTAL TRIP FIGURES

Trip generation from the new retail/hotel land uses -- after allowance for seasonal and daily adjustments plus pass-by traffic -- in the weekday afternoon peak is virtually the same as the original 1988 plan : 1226 , compared to 1209 for the original 1988 plan and 871 as claimed in the NPC.

Table 7 contains the entire set of calculations for trip generation, allowing for each land use, pass-bys and transit ridership. Column (A) replicates exactly the calculations of the NPC, whereby 861 new trips are generated from the site. It also shows in Column (B) that with a different set of assumptions, the new trips would total 1226. The primary difference in assumptions is :

	NPC	SK
90,000 as 150k - 60k	subtracts the two rates	calculates 90K rate
Design Day	average over the year	30th highest hour (3%)
Retail Trips in Christmas Season	average	+ 20%
Retail Trips on a Thursday/Friday	average	+ 10%
Retail access by auto	75%	80%
Retail pass-by in peak hour	25%	20%
Hotel pass-by	25%	0%

In summary, the NPC calculations are numerically and procedurally accurate, for the assumptions used. the disagreements are in the assumptions and methods used to achieve the calculated results, and these have been spelled out above. Due to the simultaneous effect of opposing factors, the total number of pass-by trips is the same for the two conditions (the increase in pass-by trips due to seasonal volume is balanced by the decrease allowed for hotel pass-by trips).

YEAR 2000 BUILD VOLUMES (Including SEASONAL VARIATIONS)

The primary difference in peak hour Build Volumes occurs at the Route 2 intersection (180 more cars per hour than claimed by the NPC) and at Mass. Avenue (110 additional vehicles). As noted earlier, the basic No-Build highway volumes appear reasonable.

CAPACITY ANALYSIS

.... MODEL USED

Because this project had an initial filing in 1986 – predating the MEPA guidelines – the project is "grandfathered" and the developer has three choices : Critical Lane methods, the CINCH model based on the 1985 Highway Capacity Manual, or version 2.6 of the HCS Model based on the 1994 Highway Manual. The traffic consultant chose to use the CINCH model, but has not compared the new results with those obtained in earlier EIRs. This omission is unfortunate, because we are trying to compare two traffic studies and their results.

While the CINCH model is familiar and readily available, it is based on the *1985* HCM. Without explanation, all NPC intersections are assessed with a saturated lane capacity of 1900 vph, which is an adjustment allowed in the *1994* Manual. Thus the NPC is based on a hybrid or mixed analysis. If the 1900 vph is actually a measured or empirical value applicable to the Alewife area, this presumption should have been explained explicitly in the NPC document.

.... ASSUMPTIONS

The NPC assumes that all signalized intersections are actuated. In Cambridge, all signals are actually pre-timed. This designation is significant because the use of actuated signals results in an automatic 15% reduction in total intersection delay, according to the Highway Capacity Manual -- regardless of the degree of congestion. Whether or not the analysis is seeking to take advantage of this "loophole" in the HCM can be debated – but it is clear that a 15% delay credit is significant and inappropriate.

Pedestrians -- although measured in two of the traffic counts -- have been totally ignored in the subsequent traffic analysis, including the existence of the pedestrian push-buttons at Huron and Massachusetts Avenue and their actuation by pedestrians in the peak hour.

..... MASS AVENUE and ALEWIFE BROOK PARKWAY

In the afternoon peak, the queues from northbound traffic today will extend as far as Whittemore Avenue, as they have since the mid-1980s. Long queues also occur on the Southbound parkway at Mass Avenue and on outbound Mass. Avenue. Signal phasing was changed in recent years along Mass Avenue but levels of service have not been significantly affected.

Without mitigation, the NPC indicates that critical traffic volumes would increase by 10% over the No-Build and that queues would be 2/3 longer on the parkway -- extending towards Whittemore Avenue. Adjusting these results for seasonal flows would add about another 5% to the volumes and the queues would probably be about double the No-Build. The queues on the Northbound parkway are primarily sensitive to two factors -- inbound traffic on Route 2 turning towards Mass Avenue and northbound parkway traffic making its way through the Rindge Avenue and Route 2 areas. In effect, congestion at Route 2 prevents Mass. Avenue from becoming more overloaded than it is now.

.... ROUTE 2 AND ALEWIFE BROOK PARKWAY

The intersection of Route 2 and Alewife Brook Parkway was first constructed as a rotary in 1933 and was rebuilt in its present signalized configuration in 1985. Based on a comparison of 1985 and 1995 counts, the new intersection carries fewer vehicles than the old rotary and is now clearly the Alewife bottleneck in the afternoon peak hour.

The NPC analysis contends that the critical capacity movements at the intersection are only two -- the outbound parkway flow onto Route 2 and the southbound parkway right turn onto Route 2. The CINCH analysis shows these two moves as critical, in contrast to previous studies which had found the center crossing of three moves plus the ramp as being critical.

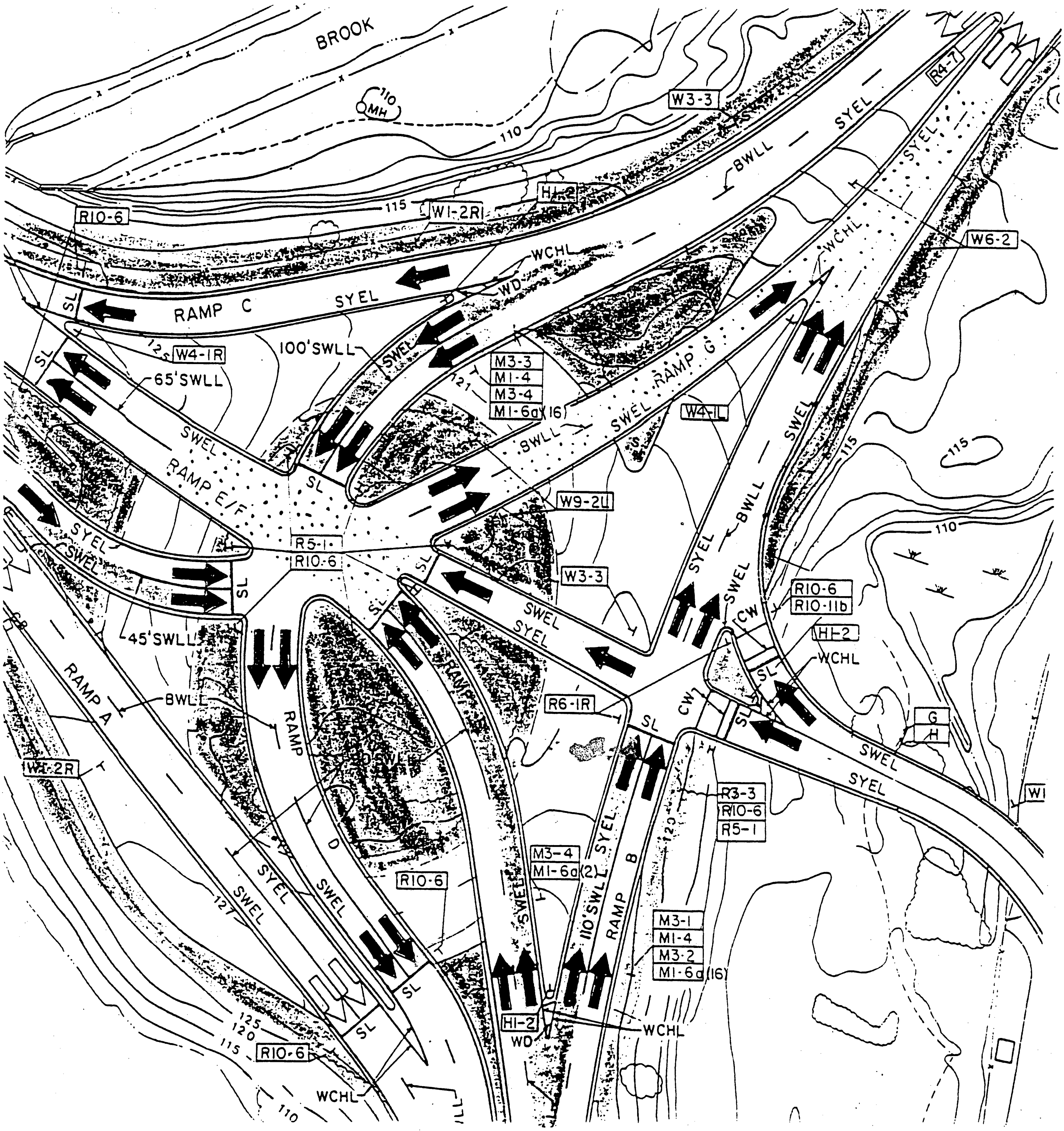
The reason why the HCM/CINCH analysis does not show the center three phases as critical is due to several factors :

1. Underestimation of the lane capacity of the right turn lane to Route 2. Measured flows in the lane (including startup losses) are 2300 vph during the green, which the CINCH estimate for the lane capacity is only 1661. This lane is physically 16 feet wide (not 13 feet as in the NPC) and some vehicles do move slowly in two lanes on this ramp near the beginning of the cycle.

2. The existing intersection contains four different examples of ***short lane*** conditions. A short lane means that queuing can occur in a lane for only a short distance before it blocks off another lane. Two short lane conditions apply to the northbound parkway approach and one each to the eastbound and southbound approaches. Any observer watching traffic in the peak hour can see that one ***never*** gets a good two lanes of flow across the stop line in any of these four situations. In three cases, it is the right lane which is short and often goes underutilized. Only on the northbound parkway movement does the left lane qualify as short and underutilized.

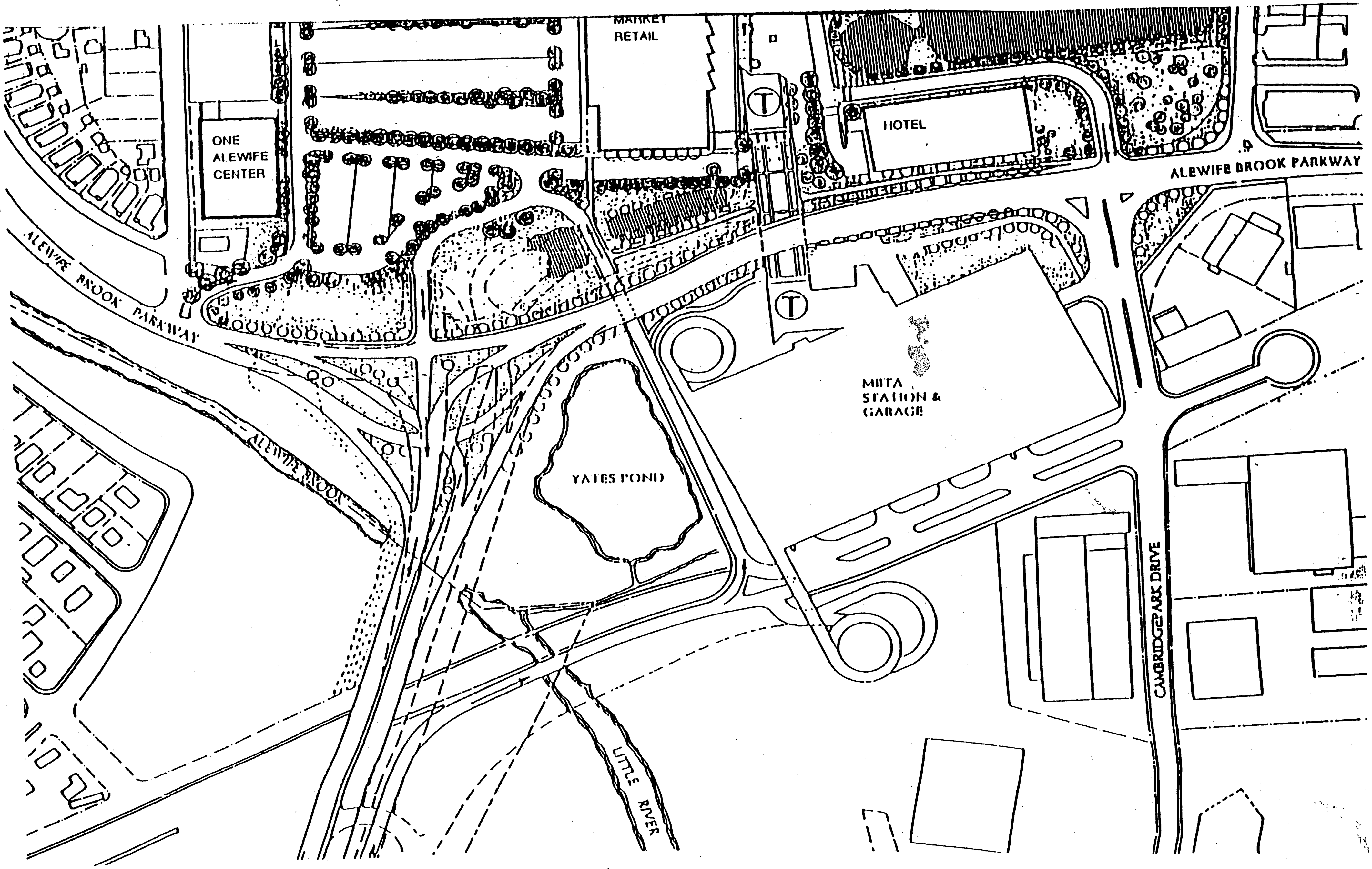
INTERIM ACCESS ROADWAY

ROUTE 2 / ALEWIFE



MERGING AREAS SHOWN IN YELLOW

SHORT LANES SHOWN IN ORANGE



MARKET
RETAIL
ONE
ALEWIFE
CENTER
HOTEL
ALEWIFE BROOK PARKWAY
ALEWIFE BROOK PARKWAY
ALEWIFE BROOK
MBTA
STATION &
GARAGE
YATES POND
CAMBRIDGEPARK DRIVE
LITTLE RIVER

The fact that the *Highway Capacity Manual* has failed to include short lane effects as capacity variables -- despite the demonstrated techniques of the Swedes, British and Australians -- is no excuse for not including these effects at Alewife. A simple calibration count of the existing approach flows could have been made, and these measurements could be used as at least an empirical adjustment of the saturated lane capacities to recognize the significance of short lanes.

Instead, the NPC fails to recognize the existence of short lane effects, let alone discuss their impacts.

3. The NPC analysis ignored the existence of the very significant *merging* movement within the intersection, whereby 2 lanes from the parkway plus one lane from the ramp merge almost immediately into two lanes. The NPC further combines the outbound parkway traffic with the ramp traffic into three lanes at the stop line. The result is a designation of zero westbound vehicles assigned to the loop ramp, which significantly reduces the queues which CINCH calculates for the ramp.

From field measurements, the flow rates through the merge area can achieve a peak of about 2400 vph per lane, before short lane effects reduce the flow rates to 2200 vph overall. This flow implies the ability to process a 4400 vph rate through the merge on green, while the NPC calculations show a capacity of 5483 vph.

The 1995 count for the vehicles through this merge area is 2265 vehicles in an hour, and since this flow gets a minute or green time or about half the cycle time, the resulting V/C ratio is 1.03 -- an accurate representation of the existing congested condition.

The NPC changes the existing signal timing for its calculation of existing conditions and provides only 48 seconds of green to an adjusted flow rate of 2623 and obtains a V/C ratio of 1.18. Thus, even with an unduly high flow capacity due to ignoring the 2-lane merge, the NPC produces an error in the opposite direction of 18 %.

There is no discussion in the NPC as to why a study of *existing* traffic should show that capacity is exceeded by 18 %. This result reflects the result of mechanically applying the HCM/CINCH model. It is possible that using older, more traditional analysis methods would have produced a more accurate result.

The existing merging operation is very significant because it is so uneven. The ramp traffic does almost all of the yielding -- often coming to a complete stop, despite the green signal indications. In typical operation, ramp traffic will stop and wait for gaps in the right lane of northbound flow. Thus, any increase in ramp traffic will tend to build up queues on the *ramp*, and not spread them onto the parkway.

This merge requires good visibility to work. The effects of rain, darkness and the setting sun in the west (which can be quite blinding) should have been considered as capacity and safety factors.

4. The analysis fails to comply with the MEPA Guidelines of July 1989, which require that in capacity studies, *"Delay, v/c ratio and queue length ... merge, ...ramp and road segment analysis analyses should be included where applicable. Also departure lane merge capacity should be addressed as required and saturated flow rates adjusted accordingly."*

5. The ramp volumes in the NPC have been adjusted to include traffic which today uses Whittemore Avenue, but the allowance for seasonal and other changes noted above in the trip generation analysis would add another 130 cars to the ramp and 65 cars into the merging area.

RAMP CONSEQUENCES OF IGNORING THE MERGE.....

The NPC method of combining westbound into northbound parkway volumes having three lanes and no merging consequences produces an assignment of zero through cars on the T-station loop ramp. According to the CINCH analysis, this two-lane ramp is shown to carry only the right turns from the ramp onto the Northbound parkway. According to the average day NPC figures, this future volume is 517 cars an hour, while the 966 cars heading to Route 2 are neatly transferred onto the parkway and off the ramp through computer magic. If allowance is made for a seasonal day, the right turn is 565 cars and the movement to Route 2 is 1030 vph.

The most direct consequence of ignoring the effects of the Route 2 merge and relocating the ramp traffic onto the Parkway is to dramatically reduce the queues on the ramp. The CINCH results are seriously misleading in that regard because of the removal and relocation of almost 2/3 of the traffic off of the ramp.

THE DRIVEWAY ON THE RAMP ....

The NPC is strangely silent on traffic conditions at the driveway itself. While it analyzes unsignalized Whittemore Avenue (225 cars an hour) in detail, it does not assess the unsignalized mall driveway (640-900 cars an hour). Given the concern of the MHD in their Section 61 finding of 1992, it is most peculiar that the developer would not analyze an issue of such concern to the permitting agency and reviewer of this Project Change Notice.

The driveway intersection can be considered with or without a consideration of queues from Route 2. It can also be considered as police officer controlled or signalized. Let us consider first the problem of officer control.

Under police officer control, It is not clear how an officer could safely control the ramp traffic, since he would be operating on a rather blind curve, and by standing on the outside of the curve at the driveway -- as he must -- he would be required to stop the outside lane first, and then the more congested and often stopped inside lane. This free-flowing right lane traffic would be able to see the officer only over the roofs of the cars in the queued up left lane. At best they might see his head and shoulders because of the curve. This arrangement places the officer at undue risk and is especially problematic under conditions of inclement weather, darkness or other periods of poor visibility. The risk posed for any detail officer strongly supports the safety argument for a traffic signal at the driveway on the ramp, if primary access is proposed at this location.

The MEPA guidelines specify that sightlines be measured and discussed.

"Appropriate intersection sight distance should also be plotted. Sight distances should be in conformance with Figure IX-27 of the AASHTO manual : A Policy on Geometric Design of Highways and Streets, 1984 edition."

While normally I do not take AASHTO guidelines as gospel, the stated recommendations do suggest a need for sight lines of 200-300 feet -- but it appears that 50-100 feet at best is available for drivers in the right lane to see past the left lane queue.

The text does not make clear how the ramp driveway would work. The existing driveway is one lane in and one lane out, and any more lanes would be unwise. One turning lane onto the ramp should be the maximum, because access to Route 2 is limited to one lane only -- the left-most lane on the ramp. If two lanes turned on the driveway, the right lane on the ramp could become blocked by vehicles waiting to merge into the left lane.

It should be noted that any time traffic on the ramp is stopped to allow vehicles to exit from the mall parking lot this stoppage of traffic will also hold up any cars seeking to enter the mall site. The exception is cars at the head of the queue and thus able to make an unblocked right turn on red. Estimates of queues on the ramp must include both through traffic on the ramp and mall traffic which is blocked from the right turn.

The original 1988 site plan showed a design for 2 driveways connecting to the internal access road system. Could complications caused by queues and turns at a single driveway bring about a return to the two-driveway plan?

As I understand the history of the Interim Access ramp near the Grace site, former state transportation officials were in such haste to build the road project that they accepted a deal whereby the land for part of the ramp was donated by Alewife Center, in exchange for a driveway on the ramp. As a result, there was no separate Mass Highway/DPW, MEPA or independent traffic engineering review of this driveway on a ramp. Today, this ramp has virtually no traffic consequence in the afternoon because most of the parking lot traffic leaves by way of Whittemore Avenue.

In the future, an active driveway on the ramp will create major circulation problems both on the public roads and ramps at Alewife and within the parking lot access road system. The 1992 Section 61 finding by MHD concentrated on the driveway operations as a major point of concern. There remains a valid public argument that any driveway located on a ramp is unwise and contrary to good road design policy. At Alewife, the queuing and congestion at the driveway argue persuasively against access at this point.

The modern solution to the excessive number and location of driveways is the concept of controlled access, such as high speed expressways. The Alewife area is only a mile from the Route 2 traffic approaching at 70 mph down Belmont Hill. The safety solution is to reduce these speeds to 35 mph or less at Alewife, to deal with traffic signals, turning vehicles and pedestrians. Adding driveways to this complex area does not help the safe transition from a high speed road to parkway operations.

Even on roads without access controls, driveways should be located in such a manner as to assure excellent sight lines for visibility and safe stopping. The location of driveways on curved ramps works against such goals and is generally undesirable design practice. In the Notice of Project Change, the developer and his traffic engineer make no effort to discuss this crucial issue and its consequences for safety and traffic flow.

It is worth noting that the traffic problems associated with the driveway are almost entirely related to the traffic entering the ramp, not traffic exiting from the ramp to the parking lot. Would a driveway be acceptable on the ramp if it were one-way, exit only?

The MHD specified in its 1992 Section 61 finding that a study be commissioned by the developer at 1/3 of the maximum development level. The clear goal for congested conditions is to require that the developer pay for a traffic control officer at the driveway and that the officer would hold back driveway traffic and give preference to regional traffic using the outbound ramp. As stated by MHD ....

"If, in the opinion of the MHD, the queue of exiting traffic from the Alewife Center site displaces regional traffic from queuing on the Outbound Ramp approach to the Dewey-Almy intersection, the proponent will hire a traffic officer to control access from the project site to the Outbound Interim Access ramp.

"The project proponent will enter into a formal agreement with the police department providing the police officer control stating that the objective of the police control is to ensure that exiting site traffic does not displace regional traffic from queuing on the Outbound Ramp."

While this mitigation concept is imaginative, it appears to contradict simple human dynamics of the relationship between traffic offers on detail and the paying client – in this case, Alewife Center. How could this officer be expected to favor the general public, at the expense of the customers of the client who is paying for his services? The concept of working contrary to the self-interests of the client goes against the very concept of hiring a detail officer.

Indeed, such an incentive would be in favor of a developer not hiring the officer or artificially reducing the number of hours he was available for duty. The practical consequences of such a policy would produce extraordinary tensions when actually applied in the field.

Unfortunately, the more likely result is that there would be a formal agreement on paper stating the "objective" of police control to be a favoring of regional traffic, but the likely daily practice would be quite different, and the officer would likely favor the party who is paying him. The mitigation would be ineffective at best and a sham at worst.

If the MHD wishes to accept the concept of a long-term, very active, two-way driveway on the ramp, the best alternative is not police officer control but *signalization*. Through signal timing -- which state highway engineers could control – the intersection could be controlled and backups of traffic on the ramp into the MBTA station area can be controlled and avoided. The intent of the 1992 Section 61 finding would best be achieved through a signal on the ramp.

.... CIRCULATION WITHIN THE PARKING LOT ....

It is not clear from the site plan how traffic would circulate within the parking lot and around the driveway area. The present design provides two adjacent intersections – the driveway connector at the ramp and the internal access road at the driveway connector. These two intersections are

less than 50 feet apart, which is exceptionally close and could contribute to localized conflicts and gridlock conditions. It is not clear what is the directionality of the access road and the allowed turns.

If there is any traffic on the access road coming from the direction of the hotel /loading dock area, the potential for blockage of this secondary intersection could be a significant factor in overall circulation. The NPC makes no analysis of circulation and queuing within the parking lot area. This analysis should be correlated with operation of the ramp driveway (including signalized operation).

If the MHD requirement of its Section 61 stipulation for no worsening of ramp congestion is respected, the resulting queue spillover would be required to occur within the mall parking lot and its access roads. The magnitude of this queuing backup should have been estimated as part of the Notice of Project Change.

.... SINGLE LANE RAMP FLOW TO ROUTE 2 .....

Due to capacity limitations in the CINCH analysis, the afternoon peak right turn flow from the southbound Parkway onto Route 2 was identified as one of the critical capacity movements. However, this treatment is very pessimistic and results in CINCH identifying the wrong critical movements.

This turn movement is made on a 16-foot-wide ramp (not 13 feet as shown in the NPC). This wide lane is normally driven as a single lane, although at the beginning of the cycle it is possible for vehicles to travel slowly side-by-side. Measured flows are in the order of 2300 vphg, whereas the NPC ascribes a capacity of only 1661 to this move. In other words, the actual capacity of this lane is 40% higher than the NPC calculations show.

Today, queues for this movement extend well past Whittemore Avenue in the afternoon peak, but this move gets almost a minute of green time -- which is sufficient to process 35-40 vehicles every cycle.

.... OTHER CRITICAL MOVES AT THE RT. 2 INTERSECTION .....

In addition to the outbound merge, the other two critical movements at Route 2/Alewife are the inbound Route 2 left turn onto the parkway and the southbound through parkway movement. The NPC analysis assumes two free flowing and unconstrained lanes for both of these moves. In practical operation today, both of these approaches are limited to a short left lane length of only 3-5 vehicles.

As noted above, the short lane effects of these critical movements could have been measured by making traffic flow measurements in the field, but instead all saturated lane capacities everywhere were set at 1900 vph, without explanation. Such measurements are essential to provide a solid empirical basis for any further capacity analysis of the Route 2/Alewife intersection and its three critical phases.

..... ALEWIFE BROOK PARKWAY AND THE MBTA STATION

The NPC treats the MBTA station access as an isolated actuated intersection, operating on a 90-second cycle. Actual afternoon operations are on a 120-second cycle, pre-timed, with interconnection to the Route 2 signal. During peak hours, any interconnection is ineffective because the queues from Route 2 extend into the Rindge Avenue area and contribute to the queuing along Alewife Brook Parkway to Concord Avenue and back to Fresh Pond Parkway.

The traffic volumes in the NPC do include the proposed 50,000 s.f. retail development at 143 Alewife Brook Parkway on the current Aku-Aku site.

The backup of queues from Route 2 produces reduced flows both on the parkway and on the left turns out of the MBTA access road. Norm Abend claims an afternoon peak level of service "B" at the MBTA station, a result which is contradicted by the evident congestion at this location today. By classical definition, an intersection which is congested because of traffic backups from another intersection should be designated as LOS F. The 1965 Highway Capazcity Manual (p. 130) provides the definitions : "Level of serviuce F represents jammed conditions. Back-ups from locations downstream or on the cross street may restrict or prevent movement of vehicles ..." The NPC fails to include these back-ups from Route 2 and treats the intersection as isolated. Since the traffic flow volumes have been reduced by congestion, it is no surprise that the CINCH model can produce results showing the following optimistic conclusion : that the existing T-station intersection with the Parkway today operates at Level of Service B in the afternoon peak. This conclusion is at odds with anyone's observation of congestion at this location.

CONCORD AVENUE AND ALEWIFE BROOK PARKWAY ......

This rotary retains its classic 1930's shape and has handled large volumes of traffic for many decades. Moreover, the accident record has been quite impressive, especially for the low number of injury accidents. However, any rotary can intimidate the sensitive and timid elements of society, and provides few opportunities for convenient pedestrian crossings.

Over the years proposals have been made to signalize this location -- with ideas for a signalized rotary or a channelized T-intersection. Any rotary design has the added advantage of handling U-turns from any direction, which is a significant issue given the median treatment which will extend from Fresh Pond Parkway to Rindge Avenue with the completion of the new MDC railroad bridge. All abutting land uses will need the ability to have vehicles able to make the necessary U-turns to enter or leave the various sites.

The traffic volumes at the Concord rotary for the three weaving sections show a startling drop between 1985 and 1995 :

	1985	1995
1. Southbound Parkway and westbound Concord (by the Ground Round)	2760	2330
2. Southbound Parkway and eastbound Concord (by Fresh Pond)	3025	2250
3. Eastbound Concord and westbound Concord (by the office building and the hotel)	2940	1870

The Notice of Project change makes no comparison of these volumes -- taken from old and new studies for Alewife Center. Has the MBTA station reduced the traffic volumes? Are either the 1985 or 1995 counts flawed? It would appear that the 1995 rotaries were undercounted.

Rotaries in the Alewife area have proven to be difficult to analyze by conventional methods -- from the Mugar EIR of 1982 until today. The NPC method is to treat the rotary as not a weaving conflict but instead an unsignalized intersection whereby right-turning vehicles wait for gaps in traffic traveling from the left. This model may be satisfactory for rural locations or the famous dual rotaries in Augusta, Maine, but it is for Alewife, because even with the markedly deflated traffic counts from 1995, the NPC shows the rotary is over capacity, when we know that 10 years ago the rotary handled much higher volumes.

Past experience has shown that treating the rotaries as fast cycle signals is an effective way of determining capacity of operations. The unsignalized analysis methods of the NPC should be shelved, and the rotary should be assessed in a way which is consistent with observed capacity at this location.

QUEUE LENGTHS AND IMPLICATIONS

The CINCH model has the beneficial aspects of being able to calculate automatically the average and 95%-ile queue lengths, based on Webster's equation for queues and delay. The NPC analysis did not include the optional 95%-ile queue lengths, which would provide the longest queue length in a 45-second period during the 15-minute peak.

Unfortunately, the CINCH model does not provide a queue length for the full peak hour, but only for a 15-minute peak period, with the queue starting at zero. The lengthy queues we observe today at Alewife over a 2-hour peak period should indicate that a queuing model based on a 15-minute queue is unduly optimistic.

The unanswered question at this time is how far will queues extend on the Outbound Ramp to Route 2 and how far will they extend into the site itself. On the one hand, there is a danger of ramp queues extending past the site driveway and into the MBTA station area. In the other direction, queues can be so long as to gridlock circulation within the parking lot. It is quite clear that bus circulation around the MBTA station would be severely hampered by these additional queues.

MITIGATION

At Route 2/Alewife, the design for the current intersection was always intended to be just "Interim" – a short-term change in circulation at Alewife. The original Interim Access studies assumed a development at Alewife Center of only 200,000 sf of office (not 1 million), with 150 hotel rooms (not 250). In this sense, the Interim Access plan was design to handle only 1/4 of the proposed traffic from the Alewife Center site. We should not be surprised if the existing intersection cannot handle the traffic.

The NPC proposes no mitigation at this location.

The mitigation idea for Alewife and Mass Avenue consists of changing the left turns from either direction on Alewife Brook Parkway so that they are in conflict with oncoming parkway traffic. (Technically, this is called making these turns "permissive" rather than "exclusive.") The CINCH model claims increased capacity, apparently because more left turn cars can now sneak through on the yellow light. This design is no improvement.

Today, the short left turn slots are often full on both sides of the intersection -- with an exclusive turning phase. If these turns had to be made against conflicting oncoming traffic, the queues would extend further.

On Saturdays, mitigation at Huron Avenue is achieved by banning any left turns. The NPC does not explain where this traffic goes or why such neighborhood access should be sacrificed as "mitigation" for a large Alewife development.

Although pedestrians were counted at Mass Avenue and Huron Ave, they were ignored during the capacity analysis, including any allowance for the pedestrian push-button phase which exists today. The existence of pedestrians should be recognized, including at Ridge Avenue, and their needs should be explicitly recognized in the intersection capacity analysis.

At Sherman Street and Rindge Avenue, proposed mitigation amounts to adding a leading phase for westbound Rindge Avenue to turn onto Sherman Street. Westbound traffic is shown as moving in only one lane, with an existing V/C ratio of 1.20. Rindge Avenue is narrow but usually at the Sherman signal the parking lane is empty and available for vehicles to travel in a second lane. With this correction, the CINCH analysis should come closer to replicating existing conditions.

Assuming that the "mitigation" is successful at moving more cars through the Sherman/Rindge intersection, it is questionable whether this is beneficial to the community. More signal change could result in more capacity on the local streets, hence more traffic on these same streets as drivers seek to avoid Alewife congestion. The NPC is proposing mitigation for commuters, not for the neighborhood or the local environment.

Sherman Street has a long history of traffic contention, including the safety at the railroad grade crossing. In the mornings and mid-afternoon a school crossing guard is stationed at this location to assist children walking along Rindge Avenue who go to the Fitzgerald or St. John's schools. A pedestrian push-button is shown in the CINCH calculations but no pedestrian phase is used in the capacity analysis.

In summary, the MEPA guidelines are quite explicit as to the requirements for mitigation :

"Any future year performance degradation under the build scenario should be shown to be fully mitigated : delay and v/c ratios should be no worse than under the no-build scenario."

Based on proposed mitigation in the Notice of Project Change, the Alewife Center as currently planned does not meet this requirement for mitigation.

PARKING REQUIREMENTS

Parking requirements at Alewife Center would have the benefit of two factors : the MBTA station and walk-in traffic would reduce the number of auto trips, compared to a shopping mall and hotel located in an area accessible only by car ..... and the office parking could become available on weekends and holidays for busy shopping periods.

The NPC includes no analysis of parking requirements or hourly accumulation of vehicles on site. From typical parking plans in recent years, and including an allowance for transit/pedestrian access, I would estimate the parking needs for a seasonal weekday as :

Parking spaces for Supermarket	200 spaces
Parking spaces for Retail	400 spaces
Parking spaces for Hotel	200 spaces
Parking spaces for Office	200 spaces
TOTAL	1000 spaces

Currently, the plan includes a total of 824 parking spaces, primarily to service the 150,000 s.f. of retail. However, the summary page 2-1 states specifically that "*This retail use, as well as the hotel use, would be served by about 800 parking spaces at-grade .....* " Other sources inform me that the hotel would have underground parking, so there is a question about the amount, location and status of the hotel parking. Two restaurants are proposed, and the site plan appears to show reserved parking for the stand-along restaurant. How are the individual land uses to be provided with adequate, controlled parking? How would possible illegal parking by MBTA patrons be controlled?

Porter Square is the best analogy as a large retail surface lot very close to an MBTA station. Here, private security guards mark and ticket vehicles which are parked too long. The presence of a mixture of Hotel, office and restaurant patrons may make the situation very difficult to enforce parking controls.

Under the original office plan, office and hotel parking was carefully controlled. With the large areas of open parking for several land uses, policing becomes very problematic.

ZONING, GROWTH POTENTIAL AND SEGMENTATION

In 1987, the developer obtained local approval from Cambridge for an Industrial-C PUD (Planned Unit Development) which would allow an FAR (Floor Area Ratio) of 2.0 and a maximum development of 1.8 million s.f. The original million s.f. proposal would have used about 60% of the available FAR. The expansion of the site from 20 to 23 acres allows for a new maximum building limit of over 2 million s.f., of which the current proposal uses only 387,000 or less than 20%.

The site has potential for growth and change. Stores could be connected into a larger mall, and structured parking could be built -- similar to the garages at malls in Newton and Saugus. Existing Grace buildings along Whittemore Avenue could also be added to the mall site. Future office expansion would also be a possibility, using the previously proposed technique of underground parking garages and office space overhead.

The issue of incremental site growth is not academic. In 1983, Spaulding and Slye disclosed to MEPA a master plan for the Triangle site of 2.275 million s.f. with 4882 spaces of structured parking covering virtually all of the site. When told by MEPA that this level of development was excessive in view of local traffic congestion, Spaulding & Slye submitted a new proposal for 670,000 s.f. which was later amended by at least two subsequent filings of a total of over 1,300,000 s.f. The Alewife Center site itself witnessed a jump of 15% in size from the original 1984 discussions for a Phase 1 waiver to the plan actually discussed in the EIR -- from 900,000 to 1,050,000 s.f.

Because the Alewife Center site has 1.6 million s.f. of unutilized FAR, the potential for future expansion is not a mere academic question. Again, the is silent on zoning and potential growth issues, despite the requirements of the MEPA Guidelines for the submission of .....

> *"A zoning map -- indicate the current zoning of the site and the adjacent parcels. Any proposed changes in zoning should be discussed relative to the potential full development of the site."*

The MEPA regulations, 301 CMR 11.16 on Segmentation are also quite explicit :

> *"In ... conducting project review, the entirety of a proposed project, including likely future expansions, shall be considered, and not separate phases or segments thereof. ... It is the intent of this rule that projects not be segmented or phased to evade or defer review."*

SECTION 61 FINDING

In 1992 the MHD specified a process by which development might occur in carefully planned phases, with additions being dependent upon capacity improvements. Originally, at the end of phase 2 or 341,000 s.f. of new development, the MHD would have required a traffic study from the developer -- explicitly for the purpose of determining the degree of congestion on the ramp and the means of reducing it, including the option mentioned earlier for a traffic officer hired to operate the driveway intersection with the objective of not delaying traffic already on the ramp.

The MHD threshold of 341,000 s.f. represents about a third of the total development originally proposed, or a third of the 1200 vph peak hour trips or 400 vph. For a *retail* site generating a similar level of traffic, the 400 vph level would be reached at 100,000 s.f. of retail/hotel development, of which 90 % of the peak hour trips would be generated by the retail and only 10 % by the hotel. Therefore, the previous threshold for MHD study for mitigation would be triggered at only slightly more than one-third of the 150,000 s.f. level of retail development.

The original MEPA comments of 1988 are also very much keyed to capacity limitations and are supportive of the general policy approach of the MHD. Currently, there is nothing in the design of the Interim Access roadway or the impact analyses which indicates that traffic from Alewife Center can satisfactorily be accommodated. Previous state studies of the existing road system have considered only 1/4 to 1/3 of the development potential to be even conceivable as compatible with the capacities of existing roadway elements.

SAFETY AND PEDESTRIAN ACCESS

The NPC provides no discussion of pedestrian circulation in the area nor of accident records in the vicinity *"At least three years of the most recent accident data available should be presented"* as required by the MEPA Guidelines. The Interim access design was the topic of considerable discussion over its safety features and the implications of additional vehicles at this location should be part of the overall assessment by the MHD.

AIR POLLUTION

The air pollution model uses begins with very crude numerical input data and then proceeds to apply other adjustments to present a seemingly precise result. The best example is link #3 in Table 10-4, whereby a road carrying 34,000 vehicles a day at an average travel speed of 30 mph is calculated to produce *43,824.98* grams a day of VOC (Volatile Organic Compounds).

Because the volume of 34,000 ADT is accurate to two decimal places at best, the Vehicle Miles traveled cannot be presented as 27,914 – it is no more accurate than 28,000. The error is in a range of 1.5%. The average speed is accurate to no more than +/- 2.5 mph and the VOC emission factor is shown as 1.57, when in actuality it could range from 1.49 to 1.67. This variation is within a 6% error range.

Combining both variations in Volume measurement and VOC emission factor for a net 7.5%, the proper VOC for link #3 should be shown in the range of 41,000 to 47,000 grams per day -- not the 43,824.98 figure presented in the Notice of Project Change.

CONSTRUCTION STAGING ISSUES

Spaulding and Slye proposes to begin construction either in 1996 or 1997. The Hotel might come at a later time. The MDC railroad bridge project would be complete and the primary outstanding unfinished business would be the replacement of the Route 2 bridge over the Minuteman bikeway. The phasing of this bridge replacement and any roadway changes involved should have been explicitly discussed in the NPC, since this bridge may be replaced before the year 2000.

Stephen H. Kaiser
Traffic and Transportation Engineer

RECOMMENDATIONS FOR NEXT STEPS:

NEED TO RESOLVE AREAS OF TECHNICAL DIFFERENCES

The preceding review has attempted to point out those areas where the NPC analysis was adequate or inadequate. Our next priority should be to seek resolution of the critical differences in traffic numbers which have come out so far in the review.

It is not clear that the Citizen's Advisory Committee has any mandate or authority to resolve these differences. First of all, many aspects are highly technical and warrant traffic engineers informally seeking agreement or correction, rather than engaging in a public pie-fight.

The first question for the developer is whether there are traffic-related issues which can be agreed to right now -- corrections made or commitments made for a timely new analysis. For example, the absence of a traffic accident analysis has been identified as a significant omission. Whose responsibility is it to obtain this data, especially if government agencies fail to cooperate? Overall, we should make a list of all the technical issues and see which ones can be highlighted for easy agreement or finding a solution.

Other issues will be more difficult to resolve, such as assumptions on trip generation, such as pass-bys or seasonal peaks. Some issues are so complex and widespread that they may not be fully resolved at Alewife Center, but an effort should be made.

The list of resolved and unresolved issues should be ready for the MEPA meeting of March 20. We have about a month to give this list as much definition as we can.

NEED TO CLARIFY THE NEXT STEPS IN THE MEPA PROCESS

The developer is planning to treat the current Notice of Project Change as a Draft Environmental Impact Report, with a Final EIR to follow in a month or so. There will have been no public scoping process and a written scoping document has already been prepared by Cambridge Community Development and Rizzo. Is the March 20 MEPA meeting for a review of the informal "draft" report we now have or is it to determine scoping issues for further study? We should find out.

In the Notice of Project Change the developer has attempted to limit the scope of any further environmental studies. In its submission letter of January 12, 1996 to the state Department of Environmental Protection, Beals and Thomas for the developer stated that "It is our hope that submittal of this analysis with the Notice of Project Change will preclude the need for scoping air quality within an EIR. ... Our client wishes to shorten the MEPA timeframe while also satisfying the requirements for environmental impact assessment." Will there be similar attempts to limit traffic review or any other area?

Nevertheless, the developer does appear to have made a public commitment to the preparation of an additional document, which would be the equivalent of a Final Environmental Impact Report. At the February 15 Advisory Committee meeting, Norm Abend made the observation that the current report is a "first cut" and that a follow-up document will be prepared. Currently, the developer's schedule does not allow for either of the documents to be found inadequate -- as occurred with the Final EIR for Alewife Center in 1987. Therefore, we should be prepared to recommend to MEPA whether or not we find the current NPC document adequate as a Draft Report or not. Any additional materials the developer can provide between now and mid-March should be considered as part of that judgment of adequacy.

SCOPING FOR TRAFFIC AND AIR POLLUTION

In terms of scoping elements, I would recommend that no further time be spent on air pollution modeling. The models are crude, misleading and have never, to my knowledge, produced any meaningful consequences or mitigation. The air pollution studies are particularly useless if the traffic analysis is not right. MEPA should identify the traffic analysis needs at Route 2/Alewife, including the ramp and driveway functions with queuing as inadequate and worthy of substantial revision in any document which is to be called a Final Supplemental EIR.

I also believe that the neighborhood and the city should work with the developer to get the traffic analysis as good and as mutually agreeable as we can over the next month -- so that no doubts are left except the specific points for which we cannot come to specific agreement. This approach will maximize agreement and understanding of uncontroversial items, which defining and highlighting those points of difference -- with clarity.

IDENTIFYING EXISTING INFORMATION RESOURCES

The scoping for this report by CDD/Rizzo remains a mystery and the first order of business should be to find out what was agreed to for this scope. The existence of traffic counts last year by Rizzo highlights another significant information source. We should receive copies of these traffic counts and any others made in recent years.

The Rizzo report does not show evidence that they reviewed the Abend report for accuracy in its intersection capacity studies, but this work may have been done and we should request copies of any such capacity review and calculations by Rizzo. Of critical significance is any review Rizzo may have done of the operations of the Route 2/Alewife intersection, of the outbound ramp from the MBTA station and queues on this ramp, and of the operations of the site driveway proposed for location on this ramp.

SPECIFIC ISSUES FOR RESOLUTION

TRIP GENERATION.... At the February 15 Abend presentation and in the Rizzo report, no mention was made of the seasonal peaking effects of retail operations and the need to assess a "design day" for traffic impact purposes. Data I have seen from the industry suggests a working "design day" as representing about 33 % more traffic than an average day. We should seek agreement to define this design goal soon, so that the discrepancies can be resolved for the number of peak hour vehicles coming from the Alewife Center site -- between my estimate of 1126 vehicles per hour and the Abend estimate of 861 cars an hour.

THE ALEWIFE/ROUTE 2 INTERSECTION.... The analysis of the Route 2/Alewife intersection, the outbound ramp from the MBTA station and the driveway on the ramp represents the most unsatisfactory design and analysis elements of the current project. The awkward merging and short lane features of the existing signal will require special measurement methods so that we can calibrate the lane capacities -- in other words, find a way so that our capacity calculations accurately reflect the number of cars getting through the merges and moving on the short lanes. I believe that these counts can be made fairly easily and quickly, in March, and I would suggest that several parties be used to gather the data. Specifically, I am willing to work with qualified field analysts from the developer, the City and the state to perform these measurements.

MAKING DETAILED TRAFFIC COUNTS... I have already made preliminary measurements (on a very cold day) for traffic merges onto Route 2 and on the one-lane connector ramp from the Parkway to Route 2. More extensive and detailed counts will provide a good empirical database to achieve some reasonable degree of technical agreement on traffic flows at Alewife and how cars queue up when they are not moving. A strategy also needs to be developed so that congestion at the driveway does not result in blockage of the ramp. We will need to discuss in detail how the driveway would work -- police officer control, traffic signal or no control whatsoever.

OBSERVING AND CALCULATING TRAFFIC QUEUES OR BACKUPS

... As we become more cognizant of the traffic congestion on the parkway, we should estimate the peak 15-minute and hourly traffic queues or "backup" for existing and future traffic conditions. These results will give us a sense for how many vehicles may try to use other routes to sneak around congestion at Alewife – specifically using North Cambridge streets. This issue has been a North Cambridge concern since the days of the Alewife Task Force in the mid-1970s and no progress has been made on it by any government agency. We should specifically ask the developer what routes traffic might use to avoid congestion at Alewife. Cambridge officials should also be expected to contribute thoughts and information. In additional, neighborhood expertise should be tapped so that maps can be prepared showing all short cut routes around Alewife.

"MITIGATION" AS A PRETEXT FOR REBUILDING THE PARKWAY

.... Rizzo has proposed that the developer make "mitigation" proposals for Alewife Brook Parkway, and on February 15 Norm Abend reviewed the history of ideas from the early 1980s for a "fly-under" underpass which gradually evolved into a major reconstruction plan for Route 2 and finally into Parkway reconstruction with overpasses that became known as "Salvucci's Wall." Apparently we are in for another round of superhighway proposals at Alewife, with Alewife Center as the catalyst for the discussion. The ideas raised by the Alewife Coalition for a Parkway design compatible with neighborhood interests appears to have been lost as the concept of mitigation" is transformed into the widening of much of the parkway. It may well be that Fred Salvucci never really lost the battle of Alewife after all.

THE FUTURE OF THE PARKWAY..... Of larger planning concern is the integration of the entire parkway system as a better looking parkway, now that the ugly old truss bridge is gone. The new bridge has nothing to recommend it aesthetically, but extensive landscaping could restore much of the parkway character. The potential for new development and traffic in this section needs urgent attention, as did not occur in the 1993 Alewife master plan. The possibility that Cambridge could see the area from Whittemore to Fresh Pond Parkway as a "Sacrificial Zone," for highway-related development (as planners in Searsport, Maine recently referred to one outlying highway section), could spell serious problems for the aesthetics and traffic concerns along the parkway.

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NOTE ON APPENDICES

I have also prepared several technical appendices, which are available for detailed technical assessment. Traffic generation forms and text from ITE are included in Appendix A, while Appendix B (still in preparation) will explain how the computer traffic counting program DOS-FLOW can be used to count and store data directly on computer disk and how 3 to 4 engineers with laptop computers could count lane flows at weaves and merges for the purposes of establishing actual existing traffic flows and capacities.

Appendix C utilizes the DOS-FLOW program to make flow measurements at merge locations. This technical paper is still in final preparation, but includes specific measurements on Route 28 in Milton and at the outbound merge at Route 2/Alewife.

Appendix D illustrates an improved method for calculating maximum queue extension, allowing for physical growback of the queue and gradual stopping and starting. It illustrates a way to calculate queues for both a 15 minute and one hour condition.

Appendix E is a 1992 paper on short lane conditions and methods of calculating their effects.

Appendix F is a report I prepared in May 1985, analysing the operating with pros and cons for the Interim Access intersection and ramp design. At the time, I indicated that compared with the old rotary, the new intersection would have less capacity and probably a worse safety record. I have seen no recent accident data, but the new intersection is clearly processing fewer cars. This report also includes a detailed analysis of short lanes and merging problems.

Appendix G includes comments on the review of the NPC by Rizzo Associates, for Cambridge Community Development.

Appendix H is still in preparation but is a summary chronology of Alewife through 1985, with the period 1986-1996 to be forthcoming. The final history will include the conflicts over development in the 1980s and the famous battle over "Salvucci's Wall." The 1980 Fly-under plan and others were precursors to a lengthy study in the mid-1980s to look at Alewife roadway improvements, ending in the extremely divisive conflict over massive road designs and shattered public process --best symbolized in the two words "Salvucci's Wall".

Appendix H is a Traffic and Design report I prepared in March 1996 -- 20 years ago -- and submitted to the original Alewife Task Force. In includes scale design sketches for the MBTA garage, the access roads and the Parkway corridor from Whittemore Avenue to Vassal Lane. There are maps of bus routes and pedestrian/bicycle circulation routes, as well as primary traffic diversion routes around Alewife through North Cambridge, Belmont and Arlington. Traffic signal timing, crosswalks, and signal locations are shown. The Alewife Center traffic analysis and mitigation plan should have at least this level of detail, and the developer's method of assessing neighborhood street impacts and diversion routes should use this earlier report as a starting point for format and analysis.

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INTRODUCTION TO THE DOS-FLOW PROGRAMS TO MEASURE TRAFFIC FLOWS

Traditionally, traffic counts have made with hand-held tally counters, sometimes with several attached to a board. The field engineer could record the number of vehicles moving straight or turning right or left, as well as possible observations on number and size of trucks. In more recent years, counting has become more automated with direct data storage. The computer can, for example, keep track of sampling periods and provide prompts and summaries for the user.

Unfortunately, many of these programs are rigid, specialized, expensive, complicated and proprietary. The necessary concentration of the field engineer on watching moving traffic may be broken by requirements to look at screen instructions, or to look down at a computer keyboard. The potential for erroneous recording of traffic events increases and the overall accuracy of traffic counts is diminished. There is a need for a commonly accessible program which can record traffic events simply, directly, and permanently. The data should be saved reliably in a form which can be imported into a spreadsheet for analysis.

The DOS-FLOW program represents an effort to provide such a traffic measurement ability – but includes the ability to measure the movement of pedestrians, buses and trains. It works if any DOS system in a PC and is very compact, requiring less than 100k of disk space.

Some of the uses for DOS-FLOW are :

1. Measure the stop line flow at a signalized intersection (determine saturated lane flows and lost time on startup)

2. Measure stop line flows through conflict areas (heavy pedestrian flows, parking, many turns, construction)

3. Record the operation of actuated signals cycle-by-cycle for actual arrival volumes (to verify proper timings)

4. Record startup response times of drivers in a queue
5. Measure stop times vs. approach headways at intersections
6. Record the growth in queue length over time in each lane and the time that turn blockage begins where short lanes exist.
7. Record both arrival and departure times to measure delay
8. Measure frequency of Right-Turns-On-Red
9. Measure flows on merging lanes after lane drops
10. Measure gaps and gap acceptance at unsignalized intersections
11. Measure elapsed time between two points (speed)
12. Measure the performance impacts of trucks and buses
13. Measure pedestrian flows along sidewalks or on crosswalks
14. Measure pedestrian walking speeds
15. Record jaywalking events and corridors along a highway
16. Measure pedestrian flows on stairways
17. Measure pedestrian flows on an escalator
18. Measure pedestrian flows through a turnstile area
19. Measure times for waiting, loading and travel on elevators
20. Measure on-off pedestrian flows for elevators (fixed or moving observer)
21. Make multimodal counts on residential streets -- cars, trucks, pedestrians, bikes, parking maneuvers)
22. Measure start and stop times of trains or buses, as well as sampled on-off counts of riders through one door.

==

To date, DOS-FLOW has been used to measure items #1, 2, 4, 5, and 9 above, and plans are to expand the applications to the other 17 possible applications during warm weather periods of 1996.

It should be noted that the effects of large trucks on stopping, starting and flow rates can be recorded with the recognition that visually there may be no difference between a large loaded and an unloaded truck, but the performance difference between loaded and unloaded may have significant implications for traffic flow. However, truck loadings can be inferred by other means, such as acceleration and shift rates, as well as noise and spring rates over road bumps. DOS-FLOW may be used to enter short lines of text commentary, as time permits.

A SAMPLE DOS-FLOW APPLICATION

The most obvious application of DOS-FLOW is for measuring stop-line flows and calibrating for saturated lane flows. An initial menu is presented to allow the user to select the measurement type -- vehicle flow, signal timing, pedestrian flows, etc. After a set of introductory and explanatory screens, the computer asks for general site information such as location, date, weather conditions, etc. This information is prompted by pressing ALT-F1 through ALT-F10 in sequence.

Now DOS-FLOW is ready to occur traffic events. The DOS prompt C> is replaced by the time, in the form 01:23:45.67. Whenever a new prompt line is displayed, it displays the new current time and saves it to computer memory, as the hour (01), minute (23) and the seconds to 1/100 (45.67). Any information can be recorded on the command line, such as

`"GREEN LIGHT ON AT ....."`

and the computer will save the entire line but not treat it as a command if the line ends with Control-C. So we press the F5 key, which is programmed to do `"GREEN LIGHT ON AT ...."` Control-C. A new command line appears, showing the time that we pressed F5.

At this point, the screen would look like :

```
01:23:45.67  GREEN LIGHT ON AT .....
01:23:56.00
```

where 56.00 is the time F5 was pressed. Suppose each car passes over the stop line 2 seconds apart and F7 records each event, the passage of 10 cars produces a screen of :

```
01:23:45.67  GREEN LIGHT ON AT .....
01:23:56.00  CAR MOVES AT ..........
01:23:58.00  CAR MOVES AT ..........
01:24:00.00  CAR MOVES AT ..........
01:24:02.00  CAR MOVES AT ..........
01:24:04.00  CAR MOVES AT ..........
01:24:06.00  CAR MOVES AT ..........
01:24:08.00  CAR MOVES AT ..........
01:24:10.00  CAR MOVES AT ..........
01:24:12.00  CAR MOVES AT ..........
01:24:14.00  CAR MOVES AT ..........
01:23:16.00
```

All of this data does not disappear as it scrolls off the screen, since a simple program already loaded in memory has told the computer to PRINT everything on the screen and then to divert any information headed for the printer and save it to memory. At the end of the measurement run, the memory is automatically saved to disk, with a filename of the user's choosing. The common term for this process is "printing to disk" so that everything shown on the screen will now be saved on a disk. With modern laptops, this disk is usually the hard drive and from 1 to 3 hours of field measurements are possible --depending upon battery capacity.

Any time after the end of the data recording session, the data saved on the disk can be imported into a word processor and printed directly. All files are in simple ASCII format and have text status only. However, the data can be reformatted to make it more visually readable and also prepare it for use in any spreadsheet. These two reformatting steps produce the following :

```
GREEN LIGHT ON AT .....23:45.67
CAR MOVES AT ..........23:56.00
CAR MOVES AT ..........23:58.00
CAR MOVES AT ..........24:00.00
CAR MOVES AT ..........24:02.00
CAR MOVES AT ..........24:04.00
CAR MOVES AT ..........24:06.00
CAR MOVES AT ..........24:08.00
CAR MOVES AT ..........24:10.00
CAR MOVES AT ..........24:12.00
CAR MOVES AT ..........24:14.00
CAR MOVES AT ..........24:16.00
```

and for the "comma-delimited" format suitable for importing into any spreadsheet :

```
GREEN LIGHT ON AT ....,23,45.67
CAR MOVES AT .........,23,56.00
CAR MOVES AT .........,23,58.00
CAR MOVES AT .........,24,00.00
CAR MOVES AT .........,24,02.00
CAR MOVES AT .........,24,04.00
CAR MOVES AT .........,24,06.00
CAR MOVES AT .........,24,08.00
CAR MOVES AT .........,24,10.00
CAR MOVES AT .........,24,12.00
CAR MOVES AT .........,24,14.00
CAR MOVES AT .........,24,16.00
```

With this latter format, the spreadsheet will read the minutes 23 and 24 as numbers, and the seconds will be treated as numbers as well, with other data shown as ASCII labels. The spreadsheet shows the time increments between vehicles, the incremental flow rate and the cumulative flow rate. In this example, the flow rates are all the same -- 1800 vph – but in field measurements, the variations and trends will be calculated, and the flow rates can be graphed directly.

				FLOW RATE vph	
			HEADWAY	INSTANT	CUMULATIVE
GREEN LIGHT ON AT	23 :	45.67			
CAR MOVES AT	23 :	56.00	2.00	1800	1800
CAR MOVES AT	23 :	58.00	2.00	1800	1800
CAR MOVES AT	24 :	0.00	2.00	1800	1800
CAR MOVES AT	24 :	2.00	2.00	1800	1800
CAR MOVES AT	24 :	4.00	2.00	1800	1800
CAR MOVES AT	24 :	6.00	2.00	1800	1800
CAR MOVES AT	24 :	8.00	2.00	1800	1800
CAR MOVES AT	24 :	10.00	2.00	1800	1800
CAR MOVES AT	24 :	12.00	2.00	1800	1800
CAR MOVES AT	24 :	14.00	2.00	1800	1800
CAR MOVES AT	24 :	16.00	2.00	1800	1800

DOS-FLOW does require formatting procedures and cut-and-pasting of blocks of data into a spreadsheet, but it allows the data to be used in all spreadsheets and requires no retyping of any information -- thus avoiding the errors which are introduced in transcribing data from one application to another (such as assembling turning movement volumes in TRANPLAN). Use of the spreadsheet, as always, requires knowledge and caution. Most flow calculations, especially at traffic signals, will involve platoon lengths of vehicles, in 30-second to 1 minute intensive periods. Measurement of intensive period flows requires this data to be separated from the intermediate stagnant periods when flows are lower or zero.

==

APPENDIX C

LANE MERGING

(TEXT IN PREPARATION)

FIELD COUNTS OF TRAFFIC FLOWS THROUGH THE WESTBOUND MERGE ---- 3 LANES INTO TWO LANES -----

...LOCATION - Alewife and Route 2
> APPROACH - outbound merge WB and NB
> DIRECTION - westbound
> LANE - All
> WEATHER - sunny 25 degrees
> PAVEMENT - dry
> Recorded by - SHK
> Date - February 12, 1996
> Time - 4:57 TO 5:05 PM

CYCLE 1

		time.....	headway	rate	#	Cum.Rate
Car starts	57	26.33			0	
Car starts	57	27.04	0.71	2535	1	2535
Car starts	57	27.87	0.83	2169	2	2338
Car starts	57	28.91	1.04	1731	3	2093
Car starts	57	29.52	0.61	2951	4	2257
Car starts	57	30.12	0.60	3000	5	2375
Car starts	57	31.00	0.88	2045	6	2313
Car starts	57	32.81	1.81	994		
Car starts	57	33.53	0.72	2500	1	2500
Car starts	57	34.51	0.98	1837	2	2118
Car starts	57	35.17	0.66	2727	3	2288
Car starts	57	36.38	1.21	1488	4	2017
Car starts	57	37.26	0.88	2045	5	2022
Car starts	57	38.41	1.15	1565	6	1929
Car starts	57	39.02	0.61	2951	7	2029
Car starts	57	43.03	4.01	449		
Car starts	57	47.09	4.06	443		
Car starts	57	48.30	1.21	1488		
Car starts	57	48.74	0.44	4091		
Car starts	57	51.05	2.31	779	0	
Car starts	57	51.98	0.93	1935	1	1935
Car starts	57	53.41	1.43	1259	2	1525
Car starts	57	54.01	0.60	3000	3	1824
Car starts	57	54.95	0.94	1915	4	1846
Car starts	57	55.55	0.60	3000	5	2000
Car starts	57	57.09	1.54	1169	6	1788
Car starts	57	57.91	0.82	2195	7	1837

CYCLE 2

Car starts	58	55.09			0	
Car starts	58	56.52	1.43	1259	1	1259
Car starts	58	58.22	1.70	1059	2	1150
Car starts	58	59.26	1.04	1731	3	1295
Car starts	58	59.92	0.66	2727	4	1491
Car starts	59	1.19	1.27	1417	5	1475
Car starts	59	1.79	0.60	3000	6	1612
Car starts	59	3.05	1.26	1429	7	1583
Car starts	59	3.60	0.55	3273	8	1692
Car starts	59	4.15	0.55	3273	9	1788
Car starts	59	4.87	0.72	2500	10	1840
Car starts	59	5.42	0.55	3273	11	1917
Car starts	59	5.97	0.55	3273	12	1985
Car starts	59	6.57	0.60	3000	13	2038
Car starts	59	7.28	0.71	2535	14	2067
Car starts	59	8.00	0.72	2500	15	2091
Car starts	59	8.88	0.88	2045	16	2088
Car starts	59	9.54	0.66	2727	17	2118
Car starts	59	10.25	0.71	2535	18	2137
Car starts	59	10.85	0.60	3000	19	2170
Car starts	59	11.95	1.10	1636	20	2135
Car starts	59	12.56	0.61	2951	21	2164
Car starts	59	13.22	0.66	2727	22	2184
Car starts	59	13.98	0.76	2368	23	2192
Car starts	59	15.14	1.16	1552	24	2155
Car starts	59	16.57	1.43	1259	25	2095
Car starts	59	18.65	2.08	865	26	1986
Car starts	59	22.50	3.85	468	27	1773
Car starts	59	23.65	1.15	1565	28	1765
Car starts	59	24.64	0.99	1818	29	1766
Car starts	59	27.88	3.24	556	30	1647
Car starts	59	28.48	0.60	3000	31	1671
Car starts	59	29.36	0.88	2045	32	1681
Car starts	59	31.56	2.20	818	33	1629
Car starts	59	33.98	2.42	744	34	1574
Car starts	59	34.42	0.44	4091	35	1602
Car starts	59	34.91	0.49	3673	36	1627
Car starts	59	35.35	0.44	4091	37	1654
Car starts	59	35.90	0.55	3273	38	1676
Car starts	59	36.89	0.99	1818	39	1679
Car starts	59	37.55	0.66	2727	40	1696
Car starts	59	38.70	1.15	1565	41	1692
Car starts	59	41.06	2.36	763	42	1645
Car starts	59	42.66	1.60	1125	43	1627
Car starts	59	42.88	0.22	8182	44	1657
Car starts	59	45.40	2.52	714	45	1610
Car starts	59	45.79	0.39	4615	46	1633
Car starts	59	46.94	1.15	1565	47	1632
Car starts	59	48.15	1.21	1488	48	1628
Car starts	59	48.59	0.44	4091	49	1649
Car starts	59	50.13	1.54	1169	50	1635
Car starts	59	51.11	0.98	1837	51	1639
Car starts	59	51.55	0.44	4091	52	1658
Car starts	59	52.87	1.32	1364	53	1651
Car starts	59	53.31	0.44	4091	54	1670
Car starts	59	54.79	1.48	1216	55	1658
Car starts	59	56.11	1.32	1364	56	1650

gap is break in ramp.... only a partial count

CYCLE 3

Car starts	0	55.32			0	
Car starts	0	56.53	1.21	1488	1	1488
Car starts	0	57.96	1.43	1259	2	1364
Car starts	0	58.67	0.71	2535	3	1612
Car starts	0	59.55	0.88	2045	4	1702
Car starts	1	0.27	0.72	2500	5	1818
Car starts	1	1.25	0.98	1837	6	1821
Car starts	1	1.80	0.55	3273	7	1944
Car starts	1	2.41	0.61	2951	8	2031
Car starts	1	2.79	0.38	4737	9	2169
Car starts	1	3.34	0.55	3273	10	2244
Car starts	1	4.05	0.71	2535	11	2268
Car starts	1	4.82	0.77	2338	12	2274
Car starts	1	5.87	1.05	1714	13	2218
Car starts	1	6.09	0.22	8182	14	2340
Car starts	1	6.25	0.16	11250	15	2470
Car starts	1	6.47	0.22	8182	16	2583
Car starts	1	6.64	0.17	10588	17	2703
Car starts	1	7.13	0.49	3673	18	2743
Car starts	1	7.35	0.22	8182	19	2843
Car starts	1	8.12	0.77	2338	20	2813
Car starts	1	8.94	0.82	2195	21	2775
Car starts	1	9.49	0.55	3273	22	2795
Car starts	1	9.93	0.44	4091	23	2834
Car starts	1	10.37	0.44	4091	24	2870
Car starts	1	11.09	0.72	2500	25	2854
Car starts	1	12.24	1.15	1565	26	2766
Car starts	1	12.73	0.49	3673	27	2791
Car starts	1	13.78	1.05	1714	28	2730
Car starts	1	14.99	1.21	1488	29	2654
Car starts	1	15.53	0.54	3333	30	2672
Car starts	1	16.36	0.83	2169	31	2652
Car starts	1	17.07	0.71	2535	32	2648
Car starts	1	17.51	0.44	4091	33	2677
Car starts	1	18.01	0.50	3600	34	2697
Car starts	1	19.10	1.09	1651	35	2649
Car starts	1	19.60	0.50	3600	36	2669
Car starts	1	20.15	0.55	3273	37	2682
Car starts	1	21.41	1.26	1429	38	2622
Car starts	1	22.73	1.32	1364	39	2561
Car starts	1	23.33	0.60	3000	40	2571
Car starts	1	24.27	0.94	1915	41	2549
Car starts	1	24.93	0.66	2727	42	2553
Car starts	1	25.48	0.55	3273	43	2566
Car starts	1	25.97	0.49	3673	44	2584
Car starts	1	26.79	0.82	2195	45	2574
Car starts	1	27.56	0.77	2338	46	2568
Car starts	1	27.67	0.11	16364	47	2615
Car starts	1	27.89	0.22	8182	48	2653
Car starts	1	29.10	1.21	1488	49	2611
Car starts	1	29.60	0.50	3600	50	2625
Car starts	1	30.80	1.20	1500	51	2587
Car starts	1	31.41	0.61	2951	52	2594
Car starts	1	32.40	0.99	1818	53	2573
Car starts	1	33.44	1.04	1731	54	2550
Car starts	1	34.15	0.71	2535	55	2550
Car starts	1	34.76	0.61	2951	56	2556
Car starts	1	35.25	0.49	3673	57	2569
Car starts	1	36.02	0.77	2338	58	2565
Car starts	1	36.57	0.55	3273	59	2575
Car starts	1	37.12	0.55	3273	60	2584
Car starts	1	38.11	0.99	1818	61	2566
Car starts	1	39.10	0.99	1818	62	2549
Car starts	1	39.98	0.88	2045	63	2539
Car starts	1	40.80	0.82	2195	64	2533
Car starts	1	41.40	0.60	3000	65	2539
Car starts	1	42.45	1.05	1714	66	2521
Car starts	1	43.11	0.66	2727	67	2524
Car starts	1	43.71	0.60	3000	68	2529
Car starts	1	45.80	2.09	861	69	2460
Car starts	1	47.12	1.32	1364	70	2432
Car starts	1	48.98	1.86	968	71	2382
Car starts	1	51.13	2.15	837	72	2322
Car starts	1	53.76	2.63	684	73	2248
Car starts	1	55.30	1.54	1169	74	2221
Car starts	1	56.67	1.37	1314	75	2200

February 13, 1996

CYCLE 4

Car starts	2	53.80			0	
Car starts	2	55.72	1.92	937	1	937
Car starts	2	56.82	1.10	1636	2	1192
Car starts	2	57.04	0.22	8182	3	1667
Car starts	2	58.85	1.81	994	4	1426
Car starts	2	59.40	0.55	3273	5	1607
Car starts	3	0.61	1.21	1488	6	1586
Car starts	3	1.05	0.44	4091	7	1738
Car starts	3	2.20	1.15	1565	8	1714
Car starts	3	2.58	0.38	4737	9	1845
Car starts	3	2.97	0.39	4615	10	1963
Car starts	3	3.79	0.82	2195	11	1982
Car starts	3	4.62	0.83	2169	12	1996
Car starts	3	5.22	0.60	3000	13	2049
Car starts	3	5.61	0.39	4615	14	2134
Car starts	3	6.48	0.87	2069	15	2129
Car starts	3	6.87	0.39	4615	16	2204
Car starts	3	7.42	0.55	3273	17	2247
Car starts	3	8.41	0.99	1818	18	2218
Car starts	3	8.63	0.22	8182	19	2306
Car starts	3	9.07	0.44	4091	20	2358
Car starts	3	9.83	0.76	2368	21	2358
Car starts	3	10.49	0.66	2727	22	2373
Car starts	3	12.03	1.54	1169	23	2271
Car starts	3	12.86	0.83	2169	24	2267
Car starts	3	13.35	0.49	3673	25	2302
Car starts	3	14.17	0.82	2195	26	2297
Car starts	3	15.00	0.83	2169	27	2292
Car starts	3	15.66	0.66	2727	28	2306
Car starts	3	16.70	1.04	1731	29	2279
Car starts	3	18.07	1.37	1314	30	2225
Car starts	3	19.23	1.16	1552	31	2194
Car starts	3	20.27	1.04	1731	32	2176
Car starts	3	21.04	0.77	2338	33	2181
Car starts	3	21.81	0.77	2338	34	2185
Car starts	3	22.52	0.71	2535	35	2194
Car starts	3	23.24	0.72	2500	36	2201
Car starts	3	23.90	0.66	2727	37	2213
Car starts	3	24.94	1.04	1731	38	2197
Car starts	3	25.60	0.66	2727	39	2208
Car starts	3	26.20	0.60	3000	40	2222
Car starts	3	26.81	0.61	2951	41	2236
Car starts	3	27.58	0.77	2338	42	2238
Car starts	3	28.18	0.60	3000	43	2251
Car starts	3	28.95	0.77	2338	44	2253
Car starts	3	29.72	0.77	2338	45	2255
Car starts	3	30.43	0.71	2535	46	2260
Car starts	3	31.20	0.77	2338	47	2262
Car starts	3	31.80	0.60	3000	48	2274
Car starts	3	32.52	0.72	2500	49	2278
Car starts	3	34.33	1.81	994	50	2221
Car starts	3	35.15	0.82	2195	51	2220
Car starts	3	35.92	0.77	2338	52	2222
Car starts	3	36.80	0.88	2045	53	2219
Car starts	3	37.41	0.61	2951	54	2229
Car starts	3	38.07	0.66	2727	55	2236
Car starts	3	38.73	0.66	2727	56	2243
Car starts	3	39.38	0.65	2769	57	2251
Car starts	3	40.37	0.99	1818	58	2242
Car starts	3	40.98	0.61	2951	59	2251
Car starts	3	41.64	0.66	2727	60	2258
Car starts	3	42.62	0.98	1837	61	2249
Car starts	3	43.89	1.27	1417	62	2228
Car starts	3	44.88	0.99	1818	63	2220
Car starts	3	46.41	1.53	1176	64	2190
Car starts	3	47.13	0.72	2500	65	2194
Car starts	3	47.79	0.66	2727	66	2200
Car starts	3	48.56	0.77	2338	67	2202
Car starts	3	49.27	0.71	2535	68	2207
Car starts	3	50.26	0.99	1818	69	2200
Car starts	3	51.25	0.99	1818	70	2193
Car starts	3	52.62	1.37	1314	71	2173
Car starts	3	53.23	0.61	2951	72	2181
Car starts	3	54.10	0.87	2069	73	2179
Car starts	3	54.93	0.83	2169	74	2179
Car starts	3	56.03	1.10	1636	75	2169
Car starts	3	56.91	0.88	2045	76	2168
Car starts	3	57.89	0.98	1837	77	2163

CYCLE 5

Car starts	4	54.52			0	
Car starts	4	55.84	1.32	1364	1	1364
Car starts	4	56.66	0.82	2195	2	1682
Car starts	4	57.60	0.94	1915	3	1753
Car starts	4	59.03	1.43	1259	4	1596
Car starts	4	59.69	0.66	2727	5	1741
Car starts	5	0.51	0.82	2195	6	1803
Car starts	5	1.06	0.55	3273	7	1927
Car starts	5	1.83	0.77	2338	8	1970
Car starts	5	2.54	0.71	2535	9	2020
Car starts	5	3.20	0.66	2727	10	2074
Car starts	5	4.02	0.82	2195	11	2084
Car starts	5	5.23	1.21	1488	12	2017
Car starts	5	5.78	0.55	3273	13	2078
Car starts	5	6.06	0.28	6429	14	2184
Car starts	5	7.32	1.26	1429	15	2109
Car starts	5	9.41	2.09	861	16	1934
Car starts	5	10.12	0.71	2535	17	1962
Car starts	5	10.84	0.72	2500	18	1985
Car starts	5	11.44	0.60	3000	19	2021
Car starts	5	11.99	0.55	3273	20	2061
Car starts	5	12.59	0.60	3000	21	2092
Car starts	5	13.31	0.72	2500	22	2108
Car starts	5	13.91	0.60	3000	23	2135
Car starts	5	14.35	0.44	4091	24	2179
Car starts	5	14.95	0.60	3000	25	2203
Car starts	5	15.39	0.44	4091	26	2242
Car starts	5	15.89	0.50	3600	27	2274
Car starts	5	16.38	0.49	3673	28	2306
Car starts	5	16.93	0.55	3273	29	2329
Car starts	5	18.03	1.10	1636	30	2297
Car starts	5	18.58	0.55	3273	31	2319
Car starts	5	19.18	0.60	3000	32	2336
Car starts	5	19.73	0.55	3273	33	2356
Car starts	5	20.94	1.21	1488	34	2316
Car starts	5	21.49	0.55	3273	35	2336
Car starts	5	22.04	0.55	3273	36	2355
Car starts	5	22.59	0.55	3273	37	2373
Car starts	5	23.30	0.71	2535	38	2377
Car starts	5	23.96	0.66	2727	39	2385
Car starts	5	25.01	1.05	1714	40	2361
Car starts	5	25.67	0.66	2727	41	2369
Car starts	5	26.38	0.71	2535	42	2373
Car starts	5	27.15	0.77	2338	43	2372
Car starts	5	27.81	0.66	2727	44	2379
Car starts	5	28.96	1.15	1565	45	2352
Car starts	5	29.67	0.71	2535	46	2356
Car starts	5	30.55	0.88	2045	47	2348
Car starts	5	31.49	0.94	1915	48	2337
Car starts	5	33.03	1.54	1169	49	2290
Car starts	5	33.63	0.60	3000	50	2301
Car starts	5	34.84	1.21	1488	51	2277
Car starts	5	35.28	0.44	4091	52	2296
Car starts	5	35.99	0.71	2535	53	2300
Car starts	5	38.13	2.14	841	54	2229
Car starts	5	38.74	0.61	2951	55	2239
Car starts	5	39.62	0.88	2045	56	2235
Car starts	5	40.22	0.60	3000	57	2245
Car starts	5	40.88	0.66	2727	58	2252
Car starts	5	42.14	1.26	1429	59	2230
Car starts	5	42.75	0.61	2951	60	2239
Car starts	5	43.35	0.60	3000	61	2249
Car starts	5	45.00	1.65	1091	62	2211
Car starts	5	45.66	0.66	2727	63	2217
Car starts	5	46.54	0.88	2045	64	2215
Car starts	5	48.62	2.08	865	65	2163
Car starts	5	49.17	0.55	3273	66	2174
Car starts	5	50.22	1.05	1714	67	2165
Car starts	5	50.77	0.55	3273	68	2176
Car starts	5	52.03	1.26	1429	69	2160
Car starts	5	52.58	0.55	3273	70	2170
Car starts	5	53.24	0.66	2727	71	2176
Car starts	5	53.95	0.71	2535	72	2181
Car starts	5	55.21	1.26	1429	73	2165
Car starts	5	56.48	1.27	1417	74	2150
Car starts	5	57.52	1.04	1731	75	2143

APPENDIX D

QUEUE LENGTHS

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SUBJECT: THE CALCULATION OF TRAFFIC QUEUES

A Technical Report Prepared for Walk Boston
Version 1.1 November 1995

--------- CONTEXT AND PURPOSE OF THIS REPORT ---------

Traffic queues are significant for both vehicular and pedestrian flows. Long queues can cause gridlocking of traffic at intersections, with stopped or slow-moving cars blocking crosswalks. The goal is to assure that any queuing will occur in those areas outside of the intersection area -- or in the midblock area between the crosswalks of adjacent intersections.

Unfortunately, calculating queues is not a clearly established procedure in traffic analysis. The Highway Capacity Manual is totally silent on queue length. NCHRP has suggested that a future LOS F description "might include some measure of the duration and extent of the oversaturation" -- which is at best only a very indirect reference to the concept of queues. Chapter 9 needs to become a unified, intermodal chapter which can deal with delays and queues for both vehicles and pedestrians.

CINCH -- the officially approved EOTC/CTPS program -- uses an equation which estimates queue length at the beginning of the green phase for isolated intersections. TRANSYT -- the systems model preferred by many CA/T and Boston engineers -- is based on a maximum queue extension during the green phase for a single traffic cycle. TRANSYT cannot deal with the accumulation of queues.

The intent of this report is to establish a technical basis for traffic queuing -- with a balanced consideration of both vehicles and pedestrians. Queues will be discussed from the historical analyses of the past 40 years, with primary reliance on the work of Webster. The goal will be to achieve a reliable predictor of practical queue lengths for the batch processing of traffic signals, in contrast to individual service offered by toll booths. Specifically, these practical considerations will include the physical growback of the queue, as well as the 1-second start up time in the queue.

Chapter 1. OUTLINE OF TECHNICAL ANALYSIS

Version 1.0 of this WalkBoston Technical Report was released in September 1995, covering drafts of Chapter 1 through 4. This updated edition (Version 1.1) includes revisions and corrections to these chapters, in preparation for the addition of new chapters on the following schedule :

Version 1.0 .... QUEUING with UNIFORM Arrivals *September 1995*
Introduction
Chapter 1. OUTLINE OF TECHNICAL ANALYSIS
Chapter 2. THREE QUEUING EQUATION BY WEBSTER
Chapter 3. MODIFICATIONS TO THE WEBSTER #3 EQUATION -- *Unsaturated* Flows
Chapter 4. MODIFICATIONS -- with *Saturated* Flows

Version 2.0 .... QUEUING with RANDOM Arrivals *November 1995*
Chapter 5. QUEUING with RANDOM Arrivals -- Additional modifications

December 1995
Version 3.0 ... QUEUING with INTERCONNECTION and ACTUATION
Chapter 6. QUEUING with Signal INTERCONNECTION
Chapter 7. QUEUING with Signal ACTUATION

Version 4.0 .... PEDESTRIAN QUEUING *January 1996*
Chapter 8. Pedestrian QUEUING with UNIFORM Arrivals
Chapter 9. Pedestrian QUEUING with RANDOM Arrivals
Chapter 10. Pedestrian QUEUING with Platooning and Surge Arrivals

February 1996
Version 5.0 .... VEHICLE DELAY WITH UNIFORM ARRIVALS
Chapter 11. FOUR VERSIONS OF DELAY EQUATIONS
Chapter 12. MODIFICATIONS to DELAY EQUATIONS--Unsaturated
Chapter 13. MODIFICATIONS to DELAY EQUATIONS -- Saturated

March 1996
Version 6.0 .... VEHICLE DELAY WITH RANDOM ARRIVALS
Chapter 14. DELAYS with Random arrivals

June 1996
Version 7.0 ... QUEUING with INTERCONNECTION and ACTUATION
Chapter 15. QUEUING with Signal INTERCONNECTION
Chapter 16. QUEUING with Signal ACTUATION

Version 8.0 ....DELAY FOR PEDESTRIANS *July 1996*
Chapter 17. DELAY for Pedestrian Exclusive and/or Concurrent at signalized intersections
Chapter 18. DELAY at mid-block signals : effects of cycle time
Chapter 19. PEDESTRIAN DELAY for non-uniform arrivals
Chapter 20. Developing a DELAY-BASED LOS for vehicles and peds

Version 9.0.... ACCELERATION AND FLOW of Vehicles and Pedestrians

Chapter 21. Startup Waves and Rolling Queue Delay
Chapter 22. Variable acceleration from a Queue
Chapter 23. Vehicle Flow past a Stop Line
Chapter 24. Total stopped and rolling delays
Chapter 25. Pedestrian Response times, speeds, flow rates

Subsequent sections will deal with delays and Levels of Service for both vehicles and pedestrians.

TABLE 1 FACTORS IN QUEUE LENGTH

* Peak Hour traffic Volume -- higher volumes mean longer queues
* Number of approach Lanes -- more approach lanes reduce queue lengths
* The length of short lanes -- the shorter, usually the longer the queue
* Amount of Green Time for each approach -- Increasing green time will usually reduce queue lengths
* Length of the Signal Cycle -- shorter cycles often reduce queue lengths
* Nearby intersections which are overloaded beyond capacity :
  - Downstream bottlenecks will cause traffic to back into the upstream intersection and cause queues and reduced flows.
  - Upstream bottlenecks will reduce the volume of traffic demand that wishes to get through, thus reducing downstream queues
* Traffic signal coordination could have the following effects :
  - Break up long queues into several shorter ones; reduce gridlock
  - Reduce stopped queue lengths, but with less efficient flow (applicable only for intersections which are below capacity)
  - Allow for a long queue to exist only very briefly, with gridlock protection thereafter.

...... TECHNICAL NOTATION AND SYMBOLS

Over the years, different analysts have used technical notations in their equations, such as X or V/C for the volume-capacity ratio. In the following analysis, the use of Greek symbols is avoided. All variables are designated by a bold faced symbol, such as **V/C** and **PHF**. Table 2 lists the technical symbols used in this report.

... TWO EXAMPLES OF TRAFFIC CONDITIONS in CALCULATIONS

During the presentation of the various equations for queue lengths, two examples will be offered to provide an empirical sense for the implications of the analysis. These two examples are listed in Table 3.

TABLE 2 SYMBOLS FOR TRAFFIC VARIABLES

UNITS

A = Combined Variable **A** = ... **(seconds per vehicle)**
$1/I - 0.68 * SP_o/U_o - t_a$

a = Acceleration rate for the first car .... **(MPH per second)**

B = Combined Variable **B** .... **(seconds)**
$T_r - 0.5 * (1/I - 1.38 * SP_o - U_o/b)$

b = average Braking rate .... **(MPH per second)**

C = Combined Variable **C** = ... **(vehicles per cycle)**
$V_t * CL/(3600 * N * PHF)$

CL = Cycle Length .... **(seconds)**

D = Combined Variable **D** = ... **(vehicles per cycle)**
$s * T_g/3600$

d = delay .... **(seconds)**

g/c = ratio of green time to cycle time

I = Flow intensity of approach volume........ **(vehicles/lane per sec)**

N = Number of Lanes at stop line

PHF = Peak Hour Factor =
(0.25 x Hourly Volume/Peak 15-minute volume)

Q_g = Queue length at beginning of Green .... **(vehicles)**

Q_i = Initial Queue at beginning of peak period .... **(vehicles)**

Q_x = Maximum Extension of the Queue .... **(vehicles)**

s = saturated lane capacity .... **(vehicles per hour)**

t_a = Startup response time .... **(seconds)**

T_g = Green Time .... **(seconds)**

T_r = Red Time .... **(seconds)**

U_o = Approach speed .... **(miles per hour)**

V/C = Volume/Capacity Ratio

v/s = v/s ratio of flow volume to capacity

V_t = Through Volume .... **(Vehicles per hour)**

TABLE 3 EXAMPLE A and EXAMPLE B ...... Benchmarks ...

EXAMPLE A ... Red Time = T_r = 50 seconds
Green Time = T_g = 50 seconds
Cycle Length = **CL** = 100 seconds
Lane Capacity = **s** = 1800 vph
Approach Flow = V_t = 900 vph
Number of Lanes = **N** = 1 lane
Peak Hour Factor = **PHF** = 1.00
V/C Ratio on approach = **V/C** = 1.00

EXAMPLE B ... Red Time = T_r = 60 seconds
Green Time = T_g = 30 seconds
Cycle Length = **CL** = 90 seconds
Lane Capacity = **s** = 1500 vph
Approach Flow = V_t = 500 vph
Number of Lanes = **N** = 1 lane
Peak Hour Factor = **PHF** = 1.00
V/C Ratio on approach = **V/C** = 1.00

Chapter 2. THREE VERSIONS of the WEBSTER QUEUING EQUATION

A clear understanding of queuing is complicated by the existence of three different queuing formulas (Table 6 ABC) – all traceable to the legendary F. V. Webster of England. The first Webster equation is generally obsolete and not useful, while the second Webster equation is used by CINCH as the basis for measuring the number of cars in the queue at the beginning of Green. The third Webster formula estimates the maximum extension of the queue or "back of queue" *during* the green phase. The queue extension is the more useful figure, since it indicates how far the tail of the queue extends towards the nearest intersection.

The United States currently has no established method for estimating the queue lengths at intersections. The *Highway Capacity Manual* is completely silent on the subject, while the *Transportation and Traffic Engineering Manual* (ITE) includes Webster's second formula for the queue length at the beginning of green. The Australian SIDRA model also has a method for estimating queue length. All these models assume an Instant Stop/Instant Start traffic flow -- with simplifications summarized in Table 4. The real world differences are listed in Table 5.

TABLE 4 THEORETICAL QUEUING MODELS : SIMPLIFICATIONS

[A] ...... All vehicles stop instantly
[B] ...... All vehicles accelerate instantly to cruise speed
[C] ...... Cars in the queue have no physical length
[D] ...... All cars queue on top of each other at the stop line
[E] ...... Cars are started off the bottom of the pile, at approximately two-second intervals
[F] ...... No account is made of pedestrian movements

TABLE 5 ACTUAL VEHICLE QUEUING : COMPLEX REALITIES

[A] ...... All vehicles stop gradually, taking 5 to 15 seconds typically
[B] ...... All vehicles accelerate gradually, with cars further back in the queue accelerating at slower rates
[C] ...... Cars in the queue stop with a physical spacing of 20-25 feet
[D] ...... All cars queue in individual lanes, one behind the other.
[E] ...... The queue grows towards oncoming traffic ("queue growback")
[F] ...... Drivers have a measured start-up response time of 1 second
[G] ...... Moderate queues may cause vehicles in Boston not to yield to pedestrians crossing concurrently.

TABLE 6A THE FIRST OF THREE WEBSTER EQUATIONS

Webster's initial queueing equation has been described as the <u>average</u> queue during the entire cycle. This concept is not very useful.

$$\text{WEBSTER \#1} = \text{average queue length} = \frac{\text{Flow x Delay}}{3600} \qquad [1]$$

For Example A, the HCM delay is 42 seconds, and the average queue length of 10.5 vehicles.

For Example A, the HCM delay is 54 seconds, and the average queue length of 7.5 vehicles.

TABLE 6B THE SECOND WEBSTER EQUATION for QUEUE LENGTH

The ITE *Transportation and Traffic Engineering Handbook* contains formulas for the calculation of queue length at the beginning of Green, as formulated by Webster in the 1950s. The CINCH model has included this ITE reference in its queue length calculations.

This second and more well-known Webster equation for queuing is based partly on delay and partly on the queuing arrival rate. There are two equations, and the queue length at the beginning of Green Q_g is whichever is the larger of :

$$Q_g = \frac{\text{Lane Volume * Red Time}}{3600} = I * T_r \qquad \{2A\}$$

or .....

$$Q_g = \frac{\text{Lane Volume (0.5 * Red Time + Delay)}}{3600} = I\,(0.5 * T_r + d) \qquad \{2B\}$$

Example A, {2A} is 0.25 * 50 =12.5 cars, {2B} is 0.25 * (25 + 42) = 17 cars
Example B, {2A} is 0.14 * 60 = 8.4 cars, {2B} is 0.14 * (30 + 54) = 12 cars

These equations measure :
* the maximum number of cars in the static queue
* the largest number of accumulated stopped vehicles
* the condition occurring at the beginning of green
* the average overflow queue length during a period of saturation

They do not measure :
* how far the queue continues to extend
* how the number of stopped vehicles decreases as the queue extends
* how the queue disappears at the maximum extension of the queue
* the maximum queue extension after a period of saturation

In theory, the second Webster equation would appear to underestimate queue lengths. Because the HCM tends to <u>overestimate</u> delay, the CINCH calculations will also overestimate queue lengths. These two errors often tend to cancel out, so that CINCH queuing calculations can produce comparable results to SIDRA and other Webster #3 formulas. Thus, use of CINCH to estimate queue extension, especially under saturated conditions, appears a reasonable "ballpark" measure.

TABLE 6C THE THIRD WEBSTER EQUATION

Webster's third formula for queue length calculates the maximum extension of the queue. This measure is critical to the operation of turning lanes and the potential for queues to extend into adjacent intersections. With different notation, this equation is used by JHK (1983) and Akcelik (1988).

$$Q_x = \frac{V_t * T_r}{3600*(1-v/s)} = \frac{T_r}{(1/I - 3600/s)} \text{ cars} = \frac{SP_o * T_r}{(1/I - 3600/s)} \text{ feet} \quad [3]$$

For Example A, Q_x = 50 / (4-2) = 25 cars = 25 * 22 = 550 feet
For Example B, Q_x = 60/(7.2-2) = 12.5 cars = 12.5 x 22 = 275 feet

This general formulation is used by the TRANSYT model to estimate queue extension after one signal cycle.

The third Webster equation is not usually attributed to Webster but comes directly from his definition of uniform delays for unsaturated intersections. It does measure the queue extension and has been adopted as part of the Australian (SIDRA) method. Similar equations were originally included in precursor reports (JHK 1983) to the 1985 *Highway Capacity Manual* but were unfortunately deleted in the final edition.

The SIDRA modifications apply primarily to overflow queues and will be discussed in Chapter 4 and 5.

The Third Webster Equation has several limitations :

* Residual queuing for saturated conditions was not included
* Added queues from variable arrivals was not included.
* All of the instant stop limitations listed in Table 3 above apply.

Because the Webster #3 equation includes the vital element of queue extension, it will serve as the basis for the WalkBoston queuing analysis, with adjustments to reflect the realities of traffic flow noted in Table 5.

Chapter 3 MODIFICATIONS TO THE WEBSTER #3 QUEUE EQUATION for UNIFORM ARRIVALS

The original notation for Webster #3 equation with uniform arrivals can be modified to reflect the input measures of approach volume **V_t**, the number of Lanes **N** and peak hour factor **PHF** .... as reflected in the center part of equation 1 below. The volume intensity in vehicles per second **$I = V_t/(3600*N*PHF)$** can also be included :

$$Q_U = \frac{V_t * T_r}{3600*N*PHF*(1-v/s)} = \frac{T_r}{(1/I - 3600/s)} \qquad \{4\}$$

This model is based on the following key assumptions :

A * UNIFORM ARRIVALS
B * UNSATURATED FLOWS, with no residual queue
C * FIXED TIMING, with no actuation
D * INSTANT ACCELERATION, INSTANT STOPPING
E * NO LINEAR LENGTH TO THE QUEUE
F * ALL QUEUED VEHICLES LEAVE FROM STOP LINE
G * STARTUP INTERVALS ARE TYPICALLY 2 SECONDS
H * NO LOSSES DUE TO TURNING VEHICLES, PARKING.....

The WalkBoston queuing equation described in this chapter and Chapter 4 modifies Webster #3 with the following assumptions :

A * =UNIFORM ARRIVALS
B * +BOTH UNSATURATED AND SATURATED FLOWS
C * =FIXED TIMING, with no actuation
D * +GRADUAL ACCELERATION, GRADUAL STOPPING
E * +THE QUEUE HAS PHYSICAL LENGTH
F * +ALL QUEUED VEHICLES START UP FROM WHERE THEY STOP IN THE LINEAR QUEUE
G * +STARTUP INTERVALS ARE TYPICALLY 1 SECOND
H * =NO LOSSES DUE TO TURNING VEHICLES, PARKING

(Chapter 5 will assess the effects of random arrivals. Chapter 6 will assess signal interconnection and Chapter 7 will cover signal actuation.)

The WalkBoston equation makes 5 modifications to the Webster #3 equation, two of which tend to increase the queue length, while the other three tend to decrease the queue length (Table 7). The derivations for these adjustments are provided in Appendix A.

TABLE 7 SUMMARY of 5 Adjustments to the WEBSTER #3 Equation	EFFECT ON QUEUE LENGTH
#1* Allowance for 1-second response time on startup	Lower
#2* Allowance for physical Queue Growback	Higher
#3* Allowance for gradual braking	Lower
#4* Allowance for special arrival time of Car #1	Higher
#5* Allowance for Queue location of Car #1	Lower

The five adjustments of Table 7 are included in Equation [5], where t_a is the driver/vehicle response time on startup = 1 second.

$$Q_x = \frac{T_r + 0.5\,(1/I - 2\,SP_o/U_o - U_o/b)}{(1/I - 0.68{*}SP_o/U_o - t_a)} \qquad [5]$$

For Example A, $Q_x = (50 + 0.5(4 - 1 - 8)) / (4 - 0.5 - 1) = 19.0$ cars

For Example B, $Q_x = (60 + 0.5(7.2 - 1 - 8)) / (7.2 - 0.5 - 1) = 10.3$ cars

The differences from Equation (4) occur in both the numerator and denominator. In the *denominator*, the SPo/Uo term represents the physical growth of the queue, while the ta term is the startup reaction time of 1 second, which replaces the "toll booth" release time of 3600/s or approximately 2 seconds. This latter adjustment is quite significant, because the 20th car in the queue would begin to move 20 seconds after the green based on the empirical 1-second response time, in contrast to the startup time of 40 seconds predicted by the 2-second toll booth model.

In the *numerator*, the 0.5/I term indicates the average arrival of the first car in the middle of its time slot, rather than at the end. The SPo/Uo adjustment accounts for car #1 arriving at a distance zero from the stop line, not the spacing distance of SPo (20-25 feet). The last term in the numerator of Uo/b accounts for the time lost in gradual deceleration, whereby cars are delayed before coming to a full stop in the queue. Typically the effect is 3-4 seconds of delayed stopping.

Two additional calculations are useful for understanding the percentages of queuing under unsaturated queuing conditions. In both examples selected, the intersections are exactly at capacity – at the borderline of oversaturation.

The *percentage of cars stopping* is the ratio of the maximum queue extension to the total arriving cars per cycle:

$$\%\ \text{of CARS STOPPING} = \frac{Q_x}{I * CL} \times 100\% \qquad \{6\}$$

For Example A, %STOP = 19.0 / (0.25 * 100) * 100 = 76 %
For Example B, %STOP = 10.4 / (0.14 * 90) * 100 = 83 %

and the *percentage of time that the stopped queue exists* :

$$\%\ \text{TIME QUEUE EXISTS} = \frac{T_r + Q_x * t_a}{CL} \times 100\% \qquad \{7\}$$

For Example A, %TIME = (50 + 19.0 * 1) / 100 * 100% = 69 %
For Example B, %TIME = (60 + 10.4 * 1) / 100 * 100% = 70 %

Note that all of the calculations so far apply to an individual approach at an intersection. Each approach must be calculated separately.

Table 8 (with Figure 1) compares the results for four queueing equations for unsaturated conditions :

1. CINCH (HCM85, Webster #2, tollbooth model) {2A} {2B}
2. TRANSYT-7 (Webster #3, tollbooth model) {4}
3. SIDRA (Webster #3, tollbooth model) {8} below
4. WalkBoston (Webster #3, Batch processing model) {5}

SIDRA is basically TRANSYT plus an overflow term, to account for variable arrivals and saturation in one equation. As a result, SIDRA queues are usually higher than those for TRANSYT.

SIDRA's second term is shown below in equation {8}, which can be used for both saturated and unsaturated conditions.

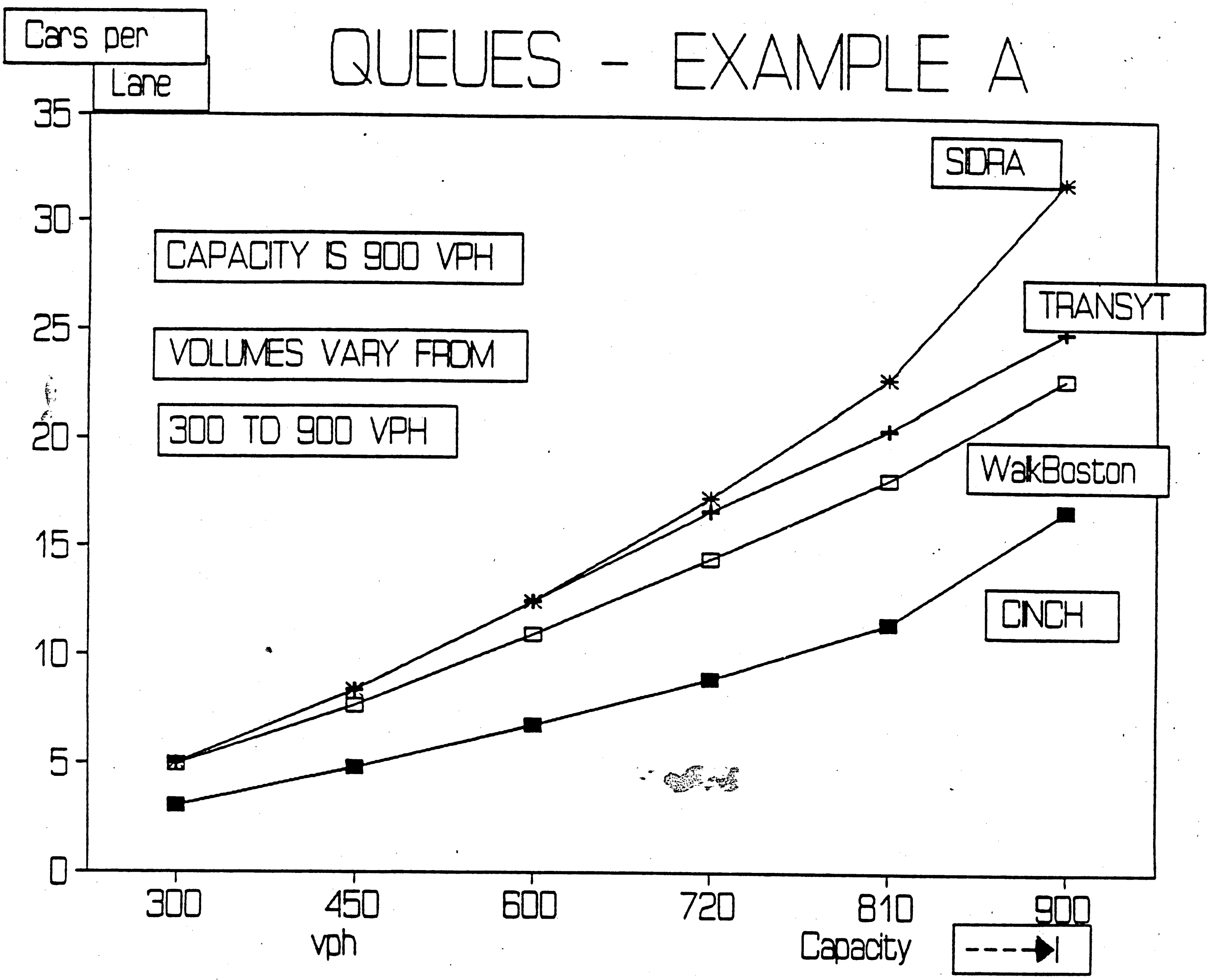
$$Q_{SIDRA} = \frac{T_r}{(1/I - 3600/s)} + 230*I*[(V/C-1)+\sqrt{(V/C-1)^2+12(V/C-x_o)/(900*I)}$$

{8}

The comparison in Table 8 applies to the benchmark case of Example A, with V/C ratios lof 1.00 or less. A batch-processing queueing equation such as WalkBoston will tend to have shorter queues for unsaturated conditions, while inclusion of queue growback will produce longer queues for saturated traffic. CINCH/HCM has shorter queues at low volumes because of its measure of queue at beginning of green and from its blanket 24% reduction (factor of 0.76) in delay attributed to a correction for stopped-only delay.

TABLE 8 COMPARISON OF FOUR QUEUEING EQUATIONS ....
for EXAMPLE A with Volumes Varied from 300 to 900 vph

Volume vph	V/C	Webster 2 CINCH	.... Webster #3 .... TRANSYT-7	SIDRA	WalkBoston
300	0.33	3.0	5.0	5.0	4.9
450	0.50	4.8	8.3	8.3	7.7
600	0.67	6.8	12.5	12.5	11.0
720	0.80	8.9	16.7	17.3	14.4
810	0.90	11.4	20.5	22.8	18.1
900	1.00	16.8	25.0	32.0	22.8



QUEUES - EXAMPLE A
Cars per
Lane
35
30
25
20
15
10
5
0
CAPACITY IS 900 VPH
VOLUMES VARY FROM
300 TO 900 VPH
SIDRA
TRANSYT
WalkBoston
CINCH
300
450
600
720
810
900
vph
Capacity

FIGURE 1

CHAPTER 4. SATURATED FLOWS with UNIFORM ARRIVALS

When the volume of approaching traffic exceeds the capacity of the intersection, the result is residual overflow traffic. This "leftover" traffic cannot be processed within the available green time and instead must be stop and be processed on the next cycle. These leftover vehicles become the first cars in the queue, and new arrivals must fall in behind them -- subject to the process of queue growth, growback and ultimate startup. With each successive cycle, this overflow queue becomes longer and longer, and with fixed traffic assignments, the queue will continue to grow throughout the standard 15-minute analysis period and throughout the peak hour.

In practical terms, queues do not grow to infinity -- as Webster presumed for both queues and delays in his 1950s formulations. Instead, rational drivers will see queues ahead or anticipate them and will seek alternate routes. For this reason, any queues and volumes in a system where one or more V/C ratios exceed 1.2 should be considered quite suspect, and the analyst should anticipate that traffic will redistribute itself to alternate routes or possibly not seek to pass through the area at all.

Many computer network models are based on link analyses and the concept that trip "demand" can be represented in a partially constrained way. Thus it is common for networks to assign large numbers of vehicles to intersections in excess of capacity. However, intersection capacity is analogous to "hitting the wall" and vehicle throughput cannot physically exceed the capacity. Any excess "demand" or actual arrivals must be stored in the queue. The more the V/C ratio exceeds 1.00 the more the intersection represents an actual or potential bottleneck -- a key metering point which will determine how much actual volume of traffic can be processed through adjacent upstream and downstream intersections.

For the purpose of elementary queuing analysis, all traffic volumes are presumed to be actual flows of arriving vehicles (unrestricted by other bottleneck locations) and that any excess traffic will automatically stop and accumulate in the overflow queue. For simplicity of analysis, all calculations will be based on either a 15-minute or 60-minute peak period, beginning with a zero overflow queue. The remainder of this chapter will be based on the following assumptions :

* UNIFORM ARRIVALS
* NO TURNING MOVEMENT OR CONFLICTS
* APPROACH TRAFFIC EXCEEDS CAPACITY
* NO TRAFFIC DIVERSIONS TO OTHER ROUTES

For most intersections, the peak 15-minute period represents 9 to 12 cycles. The procedure which will be followed in this chapter will be to assume that the total queue is composed on two parts -- the capacity queue which grows and then is processed through the intersection, and the overflow queue -- which includes any residual queue at the start of the measurement period and adds in the overflow accumulation, with allowance for queue growback.

This principal has been followed with the SIDRA queuing model, which retains the first term for uniform toll-booth queueing and adds a second term {8} to account for overflow queueing based on saturated conditions and variable arrivals.

The WalkBoston equation utilizes the uniform batch-processing concepts of Equation {5} for the queue extension at the end of the 15 minute peak period, where the volume intensity is J vehicles per second per lane, where $J = s * T_g/CL$

$$Q_x = \frac{T_r + 0.5\,(1/J - 2\,SP_o/U_o - U_o/b)}{(1/J - 0.68 * SP_o/U_o - t_a)} \qquad \{9\}$$

For Example A, Q_x = 50 - 0.5 (4 - 1 - 8) / (4 - 0.5 - 1) = 19.0 cars
For Example B, Q_x = 60 - 0.5 (7.2 - 1 -8) / (7.2 -0.5 - 1) = 10.4 cars

The simple number of unprocessed vehicles over the 15-minute or 900 second period -- including an initial residual queue of Q_i -- is

$$900\,(I - J) + Q_i$$

For Example A at V/C = 1.2 ..... 900 * (.30 - 0.25) + 0 = 45 cars
For Example B at V/C = 1.2 ..... 900 * (.17 - 0.14) + 0 = 27 cars

while the simple number of unprocessed vehicles at the beginning of the last cycle in the peak period is

$$(900 - CL) * (I - J) + Q_i$$

For Example A at V/C = 1.2 ..... (900 - 100) * (0.30 - 0.25) + 0 = 40 cars
For Example B at V/C = 1.2 ..... (900 - 100) * (0.17 - 0.14) + 0 = 24 cars

As noted in Chapter 3, the adjustment for queue extension includes both growback and startup time, and the primary adjustment occurs in the denominator. The queue at the beginning of Green needed to be multiplied by a factor which reflected more than the arrival rate I, in the form

$$\frac{1}{(1/I - 0.68 * SP_o/U_o - t_a)}$$

Similarly, for an overflow queue at the beginning of the Red phase, the effect on the overall queue extension involves a similar adjustment for growback and startup.

The effective overflow queue with growback and startup is

$$Q_{ov} = \frac{(900 - CL) * (1 - J/I) + Q_i/I}{(1/I - 0.68 * SP_o/U_o - t_a)} \qquad \{10\}$$

Therefore, complete description of uniform queue extension is :

UNSATURATED
V/C < 1.00

$$Q_x = \frac{T_r + 0.5\,(1/I - 2\,SP_o/U_o - U_o/b)}{(1/I - 0.68 * SP_o/U_o - t_a)} \qquad \{11\}$$

SATURATED
V/C > 1.00

$$Q_x = \frac{T_r + 0.5\,(1/I - 2\,SP_o/U_o - U_o/b) + (900 - CL)*(1 - J/I) + Q_i/I}{(1/I - 0.68 * SP_o/U_o - t_a)} \qquad \{12\}$$

Example A at $V/C=1.2$, $Q_i=0$, $Q_x = 26 + 800/6 / (3.3 - 1.5) = 99$ cars
Example B at $V/C=1.2$, $Q_i=0$, $Q_x = 13 + 800/6 / (6.0 - 1.5) = 43$ cars

This equation can be considered complete for both saturated and unsaturated queuing with uniform arrivals for the queue extension at the end of the last cycle of a 15-minute peak period. The influence of variable arrivals will be addressed in the next chapter.

CHAPTER 5. QUEUING WITH VARIABLE OVERFLOW

The last step in development of a queuing analysis is to account for variations in arrival times. At issue is how statistical randomness can affect queue lengths. Two general types of variability are

(a) simple bunching of traffic so that queue lengths differ from cycle to cycle, and

(b) variations which cause vehicles to be delayed more than one signal cycle.

Webster did include the 90th and 95th percentile probabilities of queue length, to adjust for (a) above, while the primary emphasis of HCM and SIDRA has been to address type (b) variations, especially as they relate to delay. Using Example A as a guide, with 25 arrivals per cycle, a Poisson analysis of probable arrivals would demonstrates the following :

Probability of	That the Total Arrivals are no more than ....
30%	22
50%	25
75%	28
90%	31
95%	33

Thus an increment of an extra 25% in the calculated queue length would provide for a confidence level of 90-95% probability of queue length within any given cycle. Such estimates would be most useful for estimates of signal operation under *unsaturated* conditions.

The second aspect of queue variability relates to the overflow aspects of queuing when V/C ratios are close to 1.00 Figure 2 illustrates the growth in queues as intersection capacity is approached and exceeded. The queue growth is composed of two substantially linear sections representing uniform arrivals, with the break point occurring at **V/C** of 1.00. The dotted line illustrates a representative allowance for variable (Poisson) arrivals. In effect, varied arrivals result in a few cycles which are overloaded, as well as others which are underutilized. The total overflow queue will thus be larger than for the simple case of uniform arrivals.

Typically queues serve as sponges to absorb variations in traffic arrivals. In practice, a traffic volume of 1350 vehicles per hour is exactly that, not a variable. The peak 15 minute period results in an equivalent flow rate of 1500 vph for **PHF** = .90. There may be variations within the 15 minute or one hour period, but not in the average value used for capacity calculations.

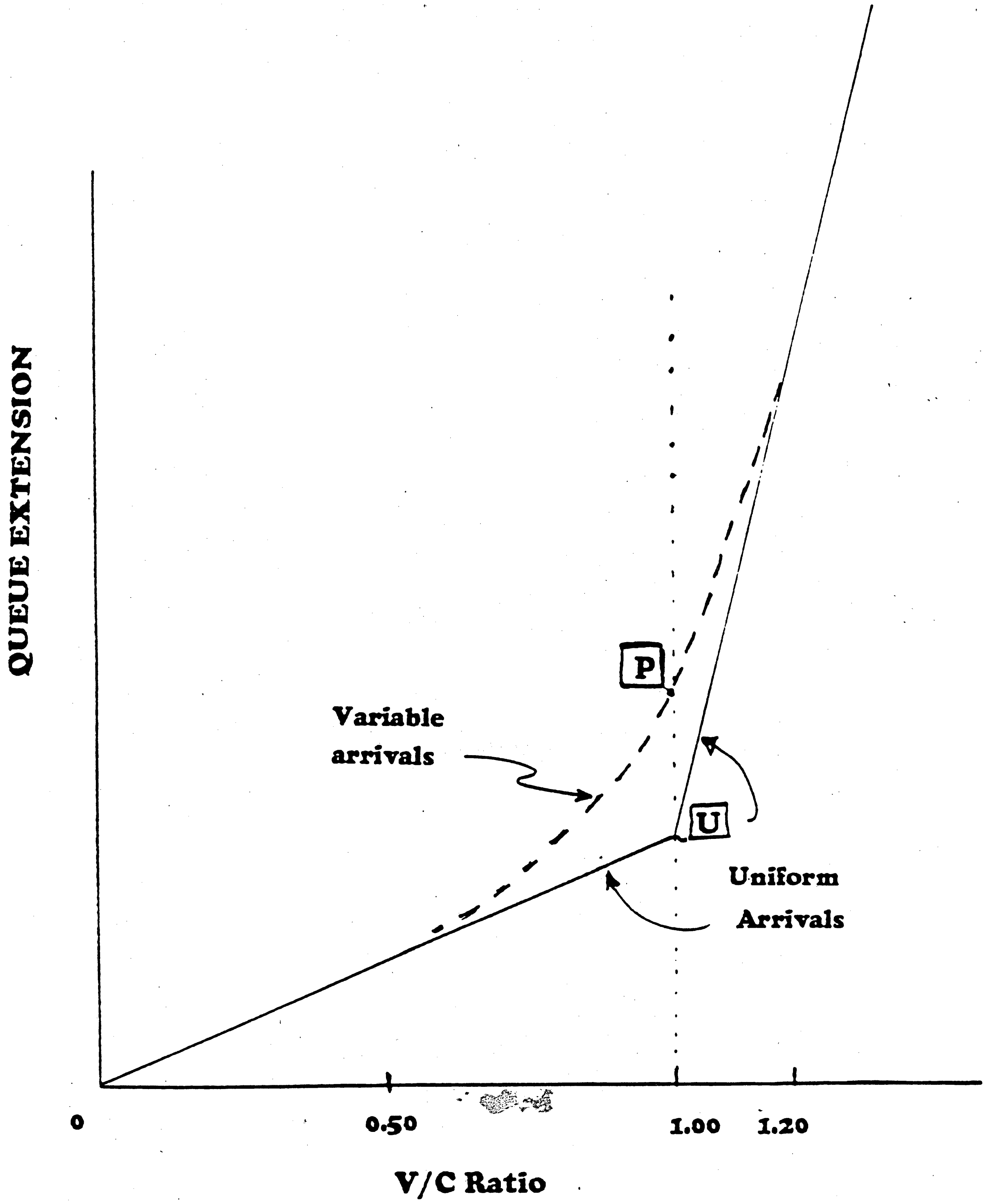

QUEUE EXTENSION
P
Variable
arrivals
U
Uniform
Arrivals
0
0.50
1.00
1.20
V/C Ratio

FIGURE 2

The most common statistical model is the Poisson distribution, which carries with it a certain small but finite probability of zero vehicle arrivals or of double or triple the average arrival rate. However, we can assume an average number of arrivals (such as a 1500 vph rate) during the 15-minute peak period, and assess the impact of variable arrivals on queue lengths within that 15-minute period.

Under this scenario, the effect of variable arrivals on total queue extension will be zero if many of the extra arrivals occur early in the 15-minute period, because the queue will be longer than average initially but will return to average at the end of the period. However, if there are few arrivals initially, the intersection could be operating under capacity – with associated efficiency losses. This loss of efficiency will mean that later vehicles will fall into a longer queue.

Therefore, the challenge is to estimate the probability of early "non-arrivals" of vehicles and the magnitude of the inefficiency they produce. Example A has 25 average arrivals every cycle (and a uniform queue extension of 19 cars). A Poisson distribution over zero to 50 vehicles in one cycle shows that the total number of accumulated gaps or voids is 2 cars in the first cycle. For the 15-minute peak period of 9 cycles, the probabilities are found to correspond to a partial Pascal's triangle, so that the gaps/voids after 15 minutes are 3.9 or 20 % of the uniform queue extension.

These calculations were made for the case of **V/C** = 1.00. Figure 2 shows this overflow point [P] as 20 % higher than uniform queue length [U]. The maximum difference between uniform and variable arrivals for both queuing and delay occurs at capacity, and this difference diminishes to zero as the **V/C** ratio becomes more distant from 1.00. The SIDRA analysis presumes that any overflow effects are non-existent for **V/C** less than about 0.67, while queues/delays are asymptotic to uniform arrivals for **V/C** values much greater than 1.00.

The solution is to generate a mathematical solution which maximizes the effect of variable arrivals at **V/C** = 1.00 -- using 20 % for the 15-minute peak, while reducing the adjustment to zero for **V/C** very much higher or lower than 1.00. A formula which achieves this goal is

$$\frac{0.2 * V/C * Q_u}{1 + 50\,(V/C - 1)^2}$$

where Q_u is the queue extension for uniform arrivals at **V/C** = 1.00. The effects of this adjustment are summarized in Table 9.

TABLE 9 OVERFLOW QUEUE ADJUSTMENT FACTOR	
V/C	**Adjustment** $\frac{0.2 * V/C}{1 + 50\,(V/C - 1)^2}$
0.00	0 %
0.33	0.2 %
0.50	0.7 %
0.66	2 %
0.90	12 %
1.00	20 %
1.10	15 %
1.20	8 %
1.50	2 %

The results appear reasonable, but empirical verification of arrivals and queuing over 15 minute periods would be useful to demonstrate the validity of the basic assumptions, including the Poisson arrivals.

With this final adjustment for variable arrivals, the WalkBoston equation for queue extension becomes :

SATURATED and UNSATURATED ......

$$Q_x = \frac{T_r + 0.5\,(1/I - 2\,SP_o/U_o - U_o/b) + (900 - CL) * (1 - J/I) + Q_i/I}{(1/I - 0.68 * SP_o/U_o - t_a)}$$

$$+ \frac{0.2 * V/C}{1 + 50\,(V/C - 1)^2} * \frac{[\,T_r + 0.5\,(1/J - 2\,SP_o/U_o - U_o/b)\,]}{1/J - 0.68 * SP_o/U_o - t_a)} \quad [13]$$

where $(1 - J/I)$ is zero if $I < J$, where $J = s * T_g/CL$

Example A at $V/C = 1.2$, $Q_x = 99 + 0.2 * 1.2/(1+50(0.2)(0.2) * 19 = 101$ cars
Example B at $V/C = 1.2$, $Q_x = 43 + 0.2 * 1.2/(1+50(0.2)(0.2) * 10 = 44$ cars

The full results of saturated and unsaturated analysis of Example A are shown in Table 10 for the 4 equations, with the results shown graphically in Figures 3 and 4.

TABLE 10 COMPARISON OF FOUR QUEUEING EQUATIONS ....
for EXAMPLE A with Volumes Varied from 300 to 1350 vph

Volume vph	V/C	Webster 2 CINCH	.... Webster #3 .... TRANSYT-7	SIDRA	WalkBoston
300	0.33	3.0	5.0	5.0	4.9
450	0.50	4.8	8.3	8.3	7.7
600	0.67	6.8	12.5	12.5	11.0
720	0.80	8.9	16.7	17.3	14.4
810	0.90	11.4	20.5	22.8	18.1
900	1.00	16.8	25.0	32.0	22.8
945	1.05	21.6	27.6	38.9	41.0
990	1.10	28.3	30.6	47.2	59.6
1080	1.20	48.1	37.5	67.8	100.6
1170	1.30	77.4	46.4	93.1	148.4
1260	1.40	118.3	58.3	123.8	204.1
1350	1.50	173.4	75.0	161.7	269.5

A review of the results of Figures 3 and 4 shows that the TRANSYT-7 model is very limited and effectively useless for any network containing a saturated intersection. Generally, the SIDRA, CINCH and WalkBoston equations are fairly close, within a band of +/- 25%. CINCH varies in some cases from having the lowest unsaturated queues to the highest saturated queues. SIDRA has the highest unsaturated queues but is in the middle range of the 4 formulas for saturated conditions.

These four queuing equations have been applied to specific field conditions, specifically to 38 intersections in the surface street corridor of the Central Artery project in Boston. Based on the most recent September 1995 TRANPLAN forecasting models, link and turning movement volumes were analyzed for both V/C and for queuing distance. The results for two locations are summarized in Figures 5,6,7 & 8 : Dewey Square at Atlantic Avenue and Summer Street and at Purchase and Summer streets, for both year 2010 AM and PM forecast conditions. Both intersections are shown with 3-phase operation on 3 approaches, with 100-second cycles.

For the case of Dewey Square, both AM and PM, the WalkBoston queue predictions were very close to TRANSYT-7 -- which is to be expected since both are based on the Webster #3 formula and the V/C ratios are 1.00, which is the limit of effectiveness of the TRANSYT-7 model. CINCH and SIDRA predict queue lengths almost twice as large.

At Purchase and Summer, the V/C ratios are quite high -- in the range of 1.40 to 1.50. The queues are quite large, especially for the major flow on Purchase Street. TRANSYT results are out of its useful range and should be disregarded. SIDRA and WalkBoston results are quite close, while the largest queues are recorded by CINCH. As noted earlier, one would have expected CINCH to underpredict queues, but because it is based on a delay model, CINCH may be predicting large queues because of excessively high values generated by the HCM delay model.

By analogy to the Volume-to-Capacity ratio, a queuing formula can be defined as the ratio of

$$\text{QUEUING-CAPACITY Ratio} = \frac{\text{PREDICTED QUEUE}}{\text{QUEUE SPACE AVAILABLE}}$$

At Dewey Square, Northbound Atlantic Avenue has space for 12 cars to queue, before the nearest crosswalk at Essex Street becomes blocked by the tail of the queue. With estimated queues extensions of 15-18 cars, the Q/C ratio is 1.25 to 1.50 but the solution could be signal coordination or shorter signal cycles.

At Purchase Street, the queue lengths are about 200 cars (WB, SIDRA) to 300 cars (CINCH), with an available space of about 30 cars. The Q/C ratio is in the range of 6 to 10, which implies very severe congestion, difficulty of coordinating signals to avoid gridlock, and a dense queue of 4500 to 6500 feet. If the signals are coordinated to at least avoid gridlocking of intersections along the way, the end of queue could extend another 1,000 feet. All of these assumptions presume that

* There are no turning movement conflicts or conflicts with pedestrians or parked cars

* There are no traffic diversions to other routes.

In practical terms, given the street network of Boston, it is unlikely that queues of more than 1 mile in length would be maintained on a regular basis. A redistribution of this traffic -- or even its disappearance from the local area -- is a probable alternative to the extremely long queues calculated all models for high V/C ratios.

The influence of pedestrian volumes on capacity and queue length must also be developed in more detail. At Purchase and Summer Street, the Purchase Street corridor is now crossed by over 10,000 pedestrians in both the AM and PM peak hours, while the traffic flow on Purchase Street is only about 1,000 vph now and is projected to double in the future. The conflict between the high volume of pedestrians and cars along Purchase Street is likely to contribute to even longer queues than one mile or alternately lead to more relocation of traffic to other routes.

The Highway Capacity Manual (Table 9.11 in '85 edition, 9.11b in '94 edition) includes adjustment factors for vehicle capacities when flows conflict with pedestrians. The presumption is that the conflict only occur for turning traffic and that there is no jaywalking. The HCM recognizes a rate of 400 pedestrians an hour as being in a "high" degree of conflict with cars. The Purchase Street crosswalk which carries 8500 pedestrians an hour in the morning includes a significant jaywalking contingent which has a significant effect of vehicle flows. Field measurements of pedestrian-limited saturated lane volumes would be appropriate to develop an empirical knowledge basis and provide for a basis for adjustment factors.

=============== END OF VERSION 1.1 ============

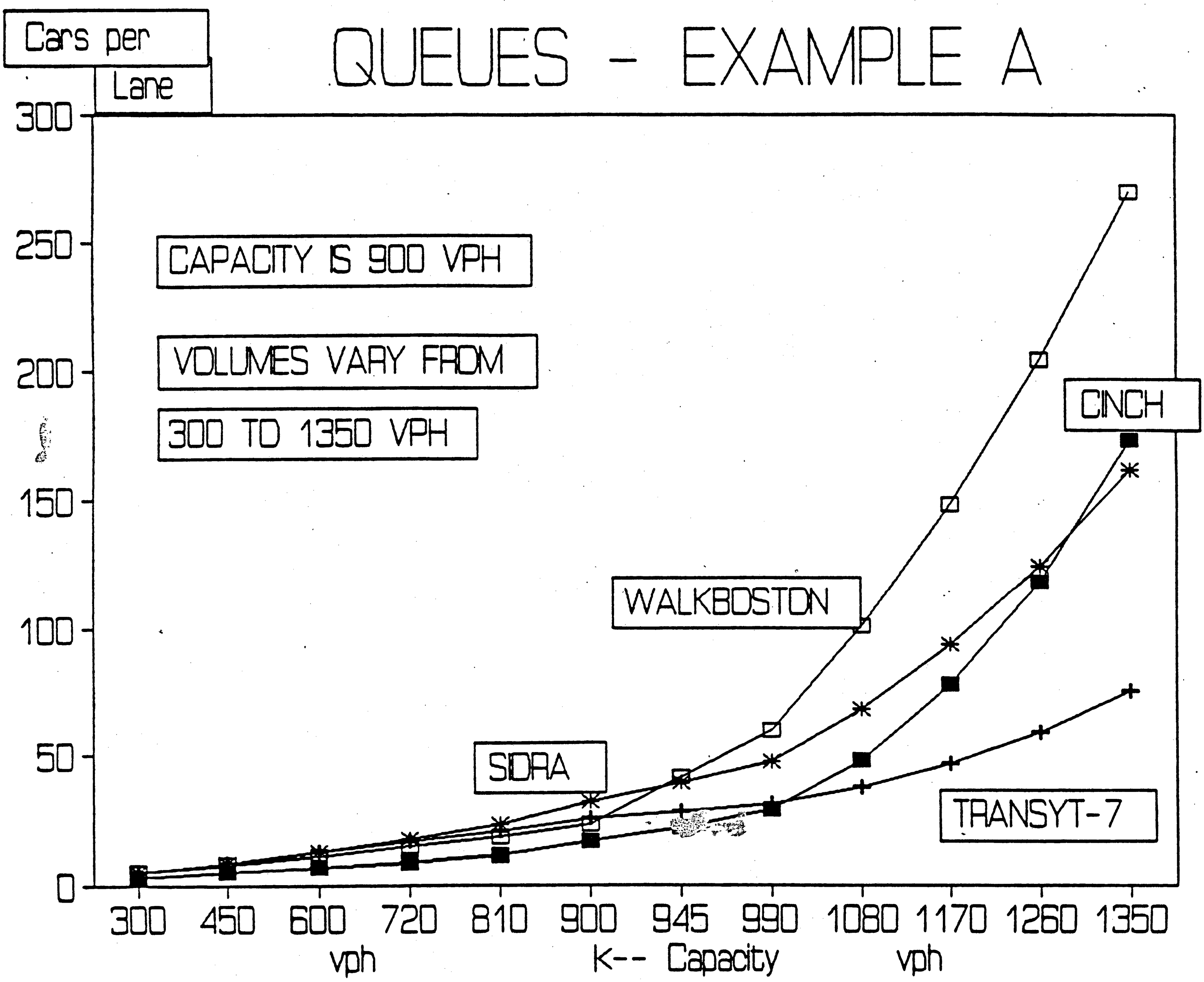

QUEUES - EXAMPLE A
Cars per
Lane
300
250
200
150
100
50
0
CAPACITY IS 900 VPH
VOLUMES VARY FROM
300 TO 1350 VPH
CINCH
WALKBOSTON
SIDRA
TRANSYT-7
300 450 600 720 810 900 945 990 1080 1170 1260 1350
vph
K-- Capacity
vph

FIGURE 3

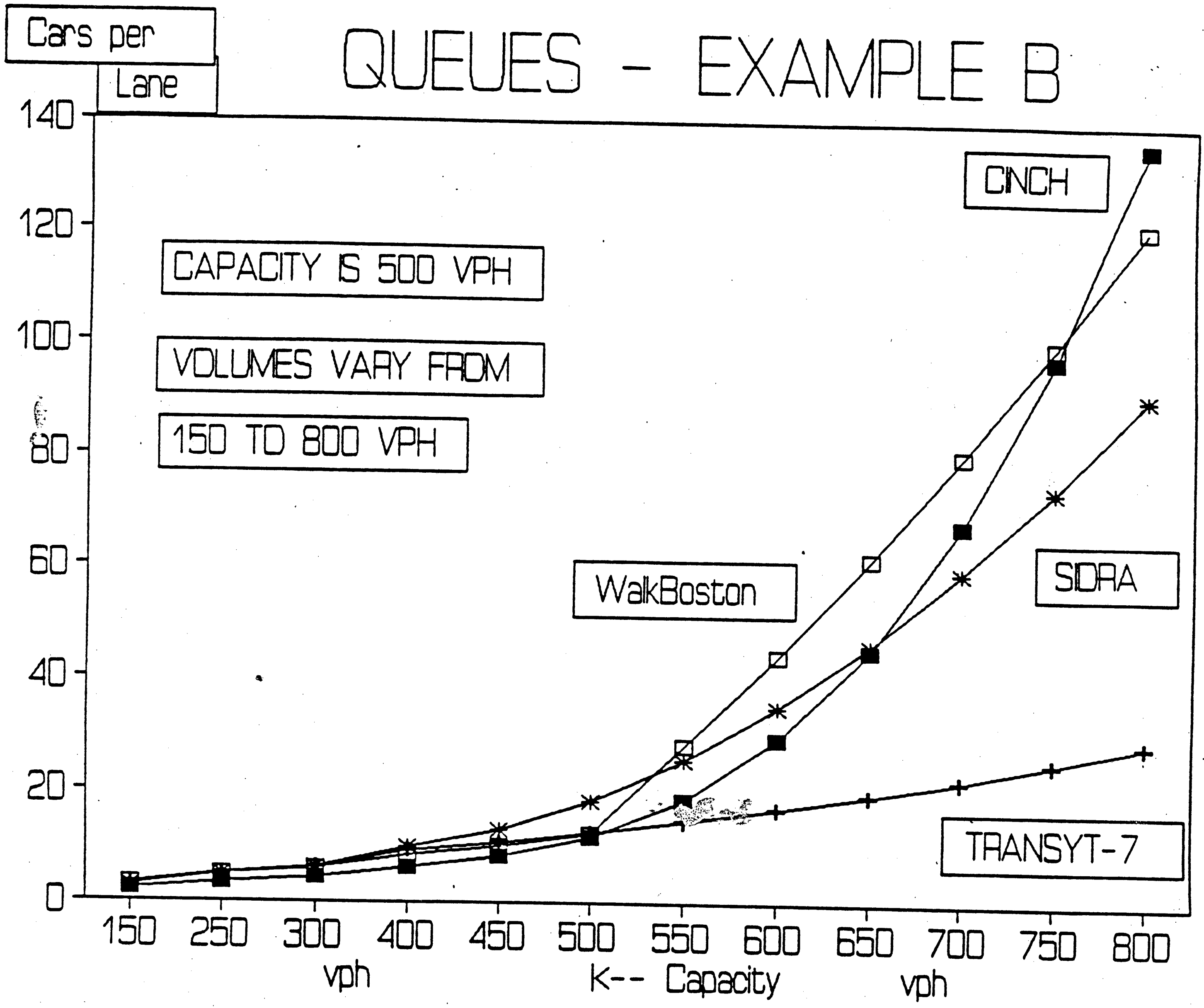
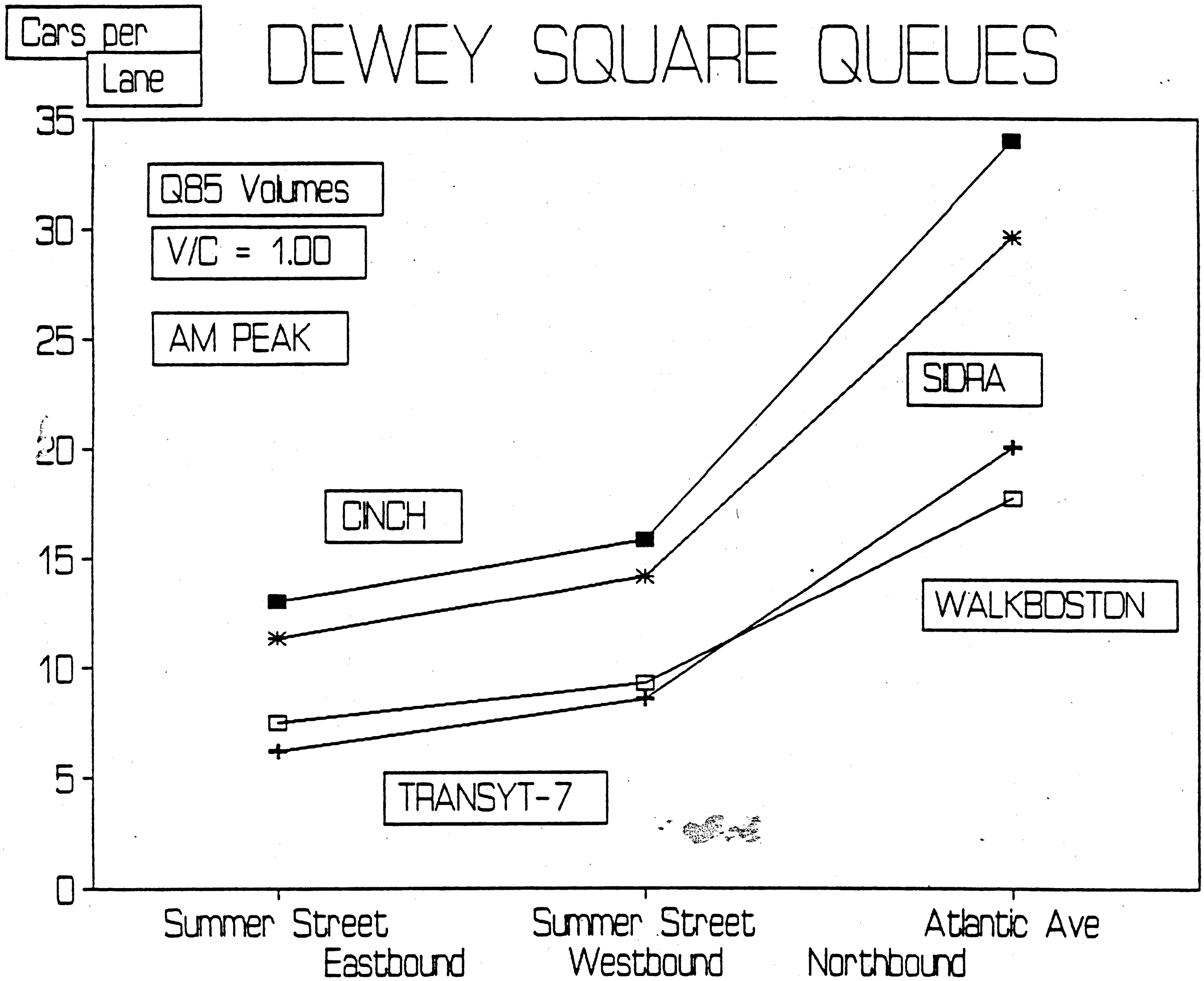
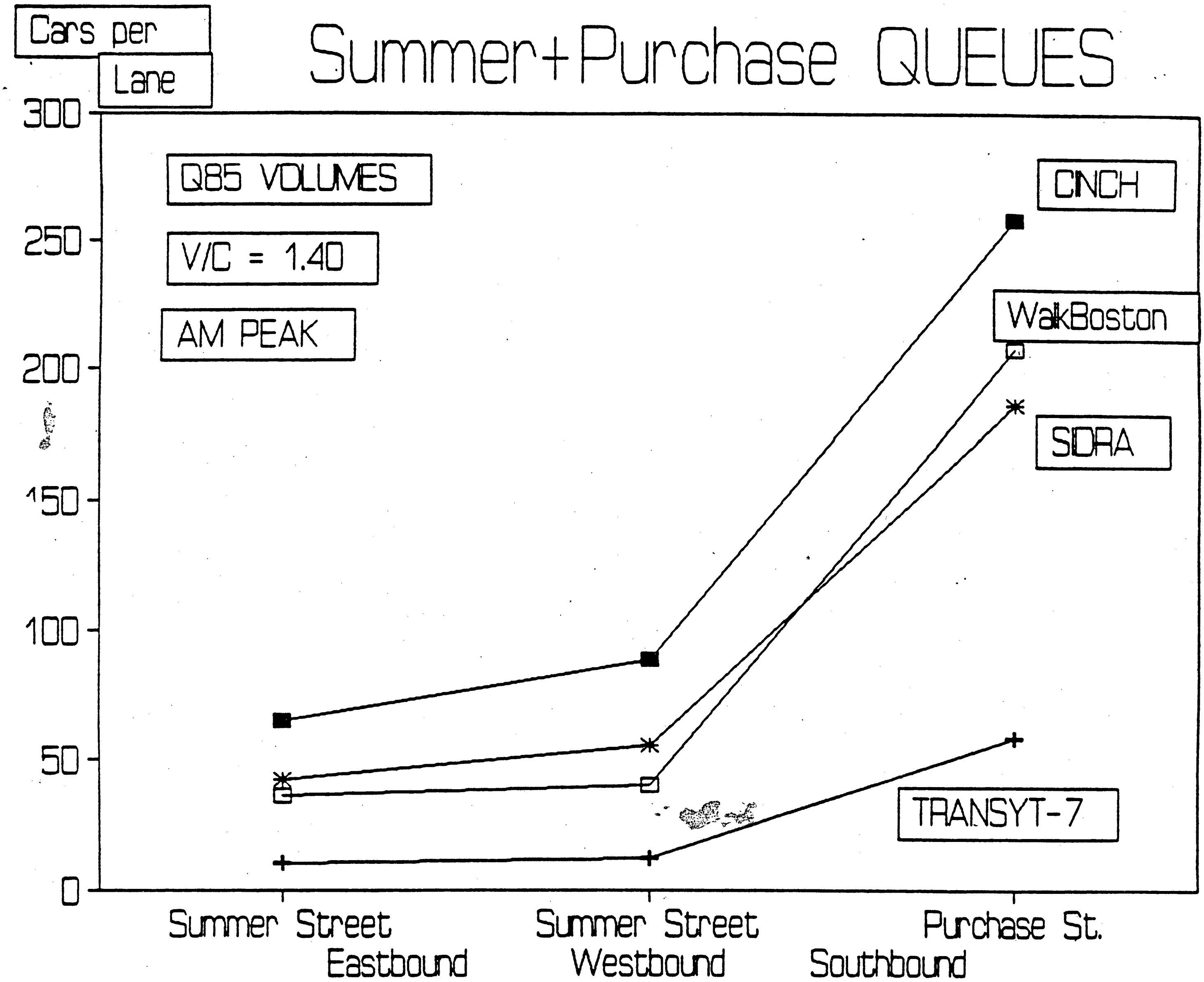

Cars per
Lane
QUEUES - EXAMPLE B
140
120
100
80
60
40
20
0
CINCH
CAPACITY IS 500 VPH
VOLUMES VARY FROM
150 TO 800 VPH
WalkBoston
SIDRA
TRANSYT-7
150
250
300
400
450
500
550
600
650
700
750
800
vph
K-- Capacity
vph

FIGURE 4



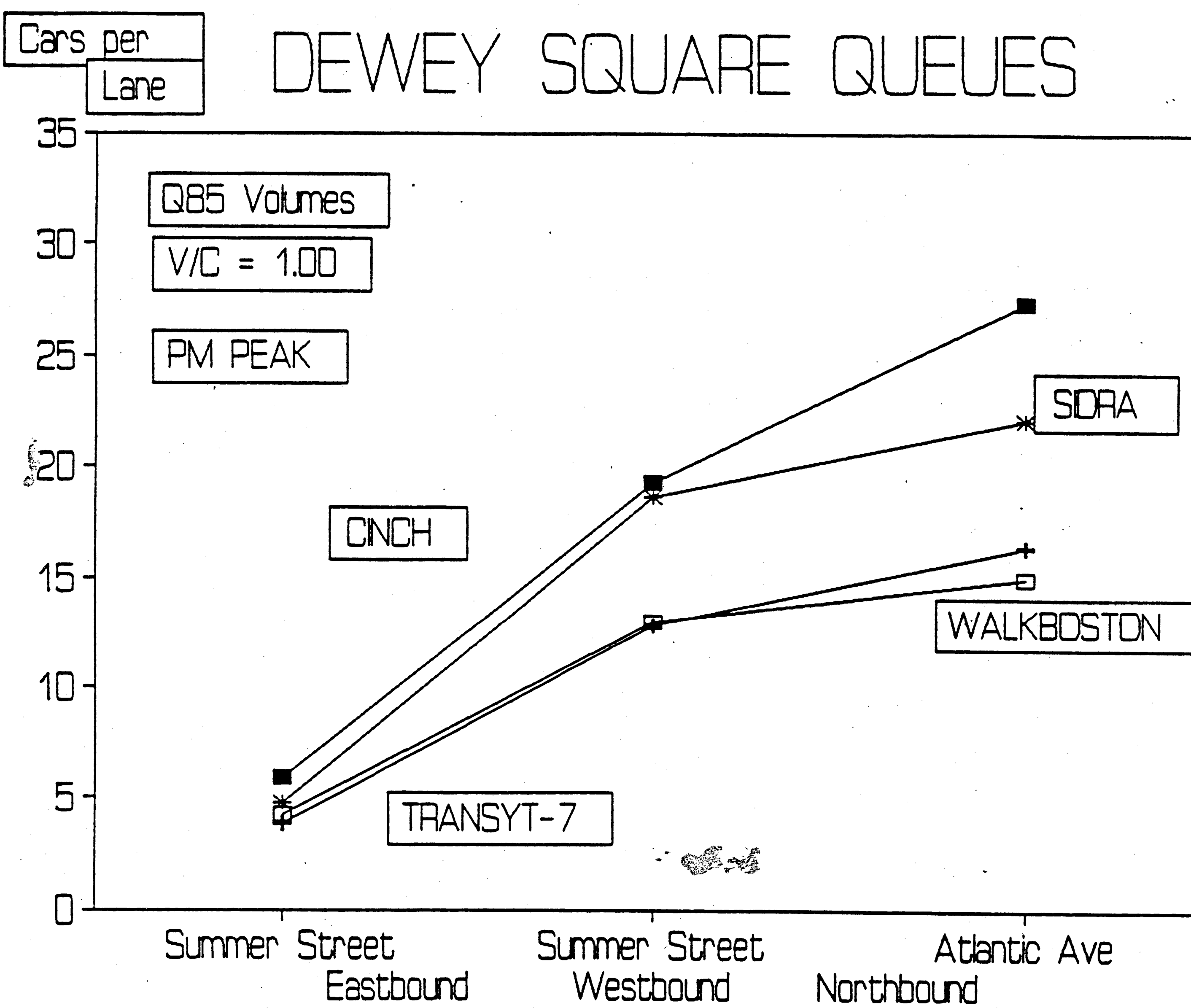
Cars per Lane
DEWEY SQUARE QUEUES
35
30
25
20
15
10
5
0
Q85 Volumes
V/C = 1.00
AM PEAK
SIDRA
CINCH
WALKBOSTON
TRANSYT-7
Summer Street Eastbound
Summer Street Westbound
Atlantic Ave Northbound

FIGURE 5



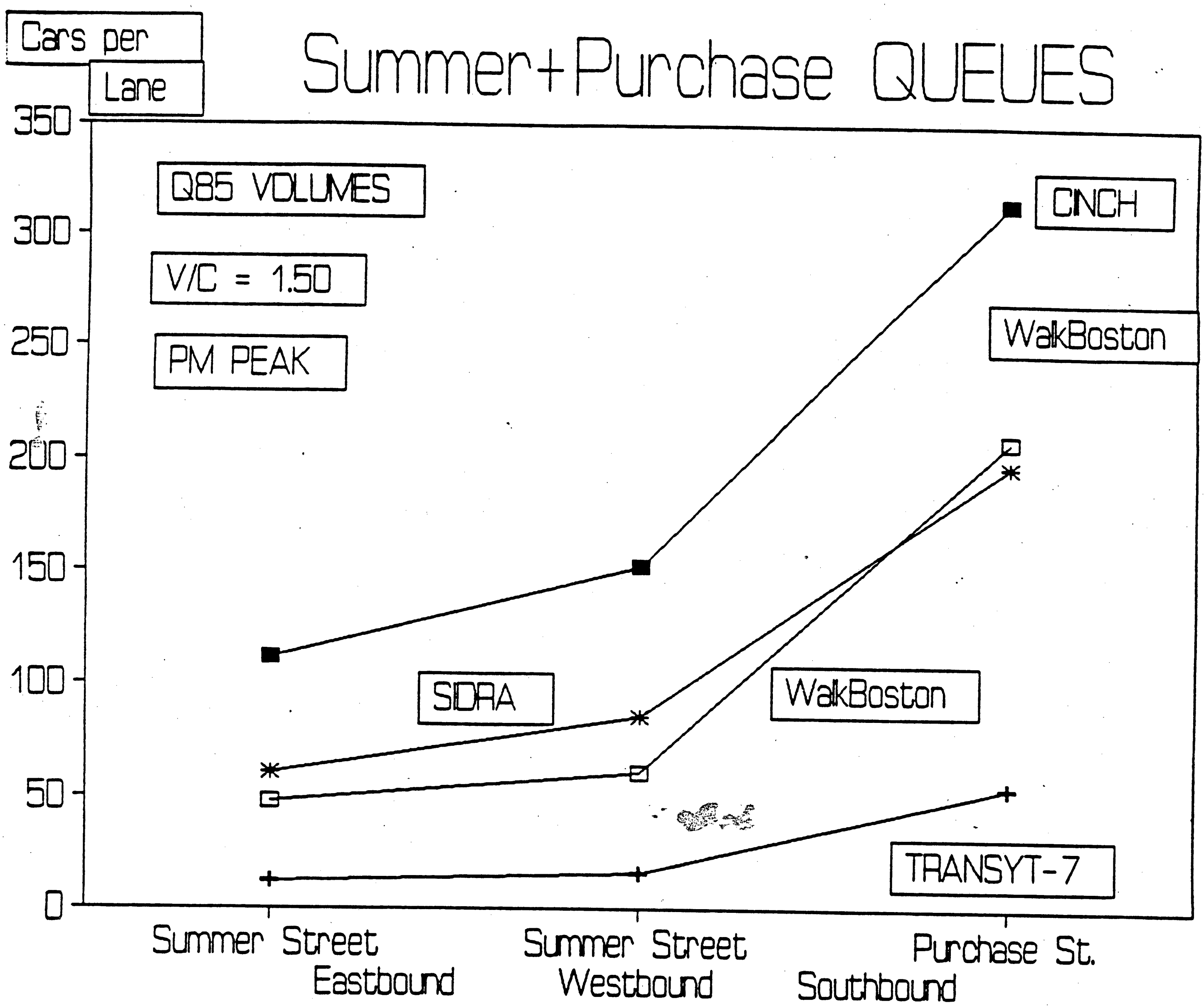
Cars per
Lane
Summer+Purchase QUEUES
300
250
200
150
100
50
0
Q85 VOLUMES
V/C = 1.40
AM PEAK
CINCH
WalkBoston
SIDRA
TRANSYT-7
Summer Street Eastbound
Summer Street Westbound
Purchase St. Southbound

FIGURE 6



Cars per
Lane
DEWEY SQUARE QUEUES
35
30
25
20
15
10
5
0
Q85 Volumes
V/C = 1.00
PM PEAK
CINCH
SIDRA
WALKBOSTON
TRANSYT-7
Summer Street Eastbound
Summer Street Westbound
Atlantic Ave Northbound

FIGURE 7



Cars per
Lane
Summer+Purchase QUEUES
350
300
250
200
150
100
50
0
Q85 VOLUMES
V/C = 1.50
PM PEAK
CINCH
WalkBoston
SIDRA
WalkBoston
TRANSYT-7
Summer Street Eastbound
Summer Street Westbound
Purchase St. Southbound

FIGURE 8

APPENDIX E

SHORT LANES
+
TURN BLOCKAGE

SHORT LANES IN HIGHWAY CAPACITY CALCULATIONS

A Discussion of Practical Applications of Uniform Flow and Stochastic Models to Short Lanes and Turn Blockage Issues.

Prepared by :
Stephen H. Kaiser
Transportation Engineer
Cambridge, Mass.

Version 3.0

November 1992

SUMMARY PAGE

This report on short lanes and the effects of Turn Blockage is intended to present both analysis and research on the implications of short lanes on intersection capacity. It also translates the data and calculations into a "plug-in" equation for the Effective Volume of approach vehicles, so that this volume can be substituted for the unadjusted value in conventional capacity calculations.

The Effective Volume represents an upwards adjustment due to turn blockage which is similar to adjustments in approach volumes made as a result of passenger car equivalents, such as truck volumes reducing the capacity of intersections by effectively increasing the equivalent numbers of cars on the approach.

Two methods are reviewed : a uniform arrival model and the Swedish model which is statistically based. Comparison of the results with an empirical probability tree shows that the Swedish equation is indeed accurate, while the uniform arrival model gives acceptable results for extremes of short lanes or total absence of turn blockage, but can produce 10-15% errors when the short lanes are almost adequate to the task.

The concept of Blocking Ratio is introduced as a measure of likely turn blockage, and the sensitivity of the analysis to partially loaded phases is provided for by inclusion of the v/s flow ratio.

Technical notation is summarized on the inside back cover.

Version 3.0 November 1992

SHORT LANES IN HIGHWAY CAPACITY CALCULATIONS

Short lanes are a physical constriction on intersection approaches. Capacity is reduced when queues grow long enough to interfere with access to another lane. Queue growth accelerates when vehicles that might use another lane now become concentrated in one lane. More signal time is required to process this longer queue.

The term "short lanes" refers to conditions of turn blockage or "lane blockage" -- when the length of queued traffic exceeds the length of turn slots or added turning lanes. For any type of traffic capacity analysis (including simple critical lane applications) the number of approach lanes at the stop line may be misleading and result in unduly optimistic results.

In terms of ultimate significance, short lane effects can easily have an impact on through capacity of 10-20%, and in worst cases can impede capacity by 30-50%. Unless the traffic engineering community comes to grips with the effect of short lanes, there is no valid reason for anyone to go beyond simple critical lane estimates. No one would be justified in many other miniscule adjustments for trucks, grades, motorcycles, etc.

Two situations of informal short lanes can result in greater intersection capacity. The most common situation is a "pocket slot" whereby a left turning vehicle stops at mid-intersection or at a median break, while other vehicles swing to the right and go around. In such cases, one car may not block but two or more will. Such situations of 1- or 2-car pocket slots are quite common in the city. The second situation involves a wide lane approach to an intersection, so that two small cars may share the same lane and permit a free right turn from the lane. The best example is Boston's Western Avenue Bridge and Soldiers Field Road near the Harvard Business School.

There are numerous examples of short lane conditions in the Cambridge area, such as Alewife Brook Parkway at both Route 2 and Rindge Avenue, Fresh Pond Parkway inbound at Mt. Auburn, Central Square, and O'Brien Highway & Prison Point Bridge. Other locations include the Hartwell Avenue jughandle in Lexington and Route 1 and 1A in Revere.

Generally, the best work on short lanes has been done in the 1977 Swedish Capacity Manual, although application guidelines and procedures have yet to catch up with the theoretical equations. The goal of this report on Short Lanes is to develop a methodology for turn blockage effects to normal traffic capacity calculations.

HOW CAPACITY IS AFFECTED BY SHORT LANES

Short lanes can produce a mixed queue which includes blocked turning vehicles as well the regular lane groups. During the green phase, the blocked vehicles move into their proper lane and leave behind gaps or voids in the traffic stream. These voids or "phantom vehicles" consume valuable green time and represent inefficient operation of the overall intersection.

For example, if the queue for a through traffic lane is composed of 10% turning vehicles, an additional 10% in green time will be required to service the through move -- compared to the typical design condition when turn blockage is presumed not to occur.

The significance of this heretofore ignored factor is sufficiently large to require its inclusion in any comprehensive traffic capacity analysis beyond the Planning Method.

EXAMPLES

The most common example of turn blockage is a short left or right turning lane which becomes blocked by the queue of mainline traffic. Other examples are heavy volumes of left turn traffic which spill over into the through traffic lanes, parking on an approach, a near-side bus stop near the stop line and a lane flare (a general traffic lane added shortly before the stop line).

HISTORY OF SHORT LANES ANALYSIS

The earliest reference to short lanes I have found is the Swedish Capacity Manual (Petersen & Imre, Feb. 1977). This report referred to "holding space", L_1, which defined the capacity of the turn (when full of turning vehicles) and considered the effect on the holding space of a queue in an adjacent lane. Both situations resulted in the assumed saturation lane flow s_1, being limited to a lower value than an unrestricted lane.

The next appearance of short lanes occurred in an interim report of February 1983 -- "NCHRP Signalized Intersection Capacity Method" -- by JHK Associates. Short lanes were discussed briefly on pages 4-3 and 4-4, but clear application guidelines were not included.

Between this 1983 report and the official 1985 Capacity Manual, any discussion of short lanes was dropped. The only fragment of short lane existence in HCM85 is hidden in Figure 9-2, which requires that analysts should record the length of the storage bay for right or left turns, Ls . Regrettably, the length of storage bays and the short lane effect is never used in any of the Chapter 9. I regard this matter as the single most significant omission in the Chapter 9 methodology.

Independent of these national and international efforts, I produced a 25-page "Signalized Intersection Capacity Manual" in April 1983. It was an attempt to simplify Circular 212 while also including an analysis of short lane queuing, which I observed to be critical at the Hartwell Avenue jughandle and at Route the 9 ramps near I-495 in Westboro. Several times in subsequent years, update drafts dealing with turn blockage were prepared, but not until November 1989 did I circulate a 13 page draft to the TRAFFC committee, entitled "Short Lane Implications for Intersection Capacity". I regard the original 1983 report as Version 1.0 of Short Lane analysis, while the 1989 document (dealing primarily with lane flares) is Version 2.0. Logically, the current 1992 report is Version 3.0.

RESPONSES TO SHORT LANE PROBLEMS

If short lanes adversely affect capacity, there are several possible responses :

* Lengthen the turn lane

* Shorten the signal cycle

* Provide more green time to the affected phase.

* Provide TSM or other methods to reduce traffic volumes or find alternate routes.

IDENTIFICATION OF SHORT LANE PROBLEMS

In practical terms, traffic analysts should have two general types of questions about short lane effects and intersection performance :

* Is the length of the storage lane or slot adequate for the traffic demand? How much longer should it be to avoid turn blockage?

* How much traffic volume can be served through a short lane of fixed length? ..... And how much additional traffic demand is blocked and caught in a mixed queue?

The existing conditions case is one of traffic impact on an existing location of known geometry. Slot lengths and volumes are fixed, signal timing is a variable, and we seek an overall V/C ratio and LOS. In the case of design, we may wish to alter slot lengths and and adjust volumes.

When the capacity model for short lanes is put into a spreadsheet, it will be possible to do both existing and design analysis at almost the same time, by freezing or altering variables.

VEHICLE ARRIVALS : Uniform or Stochastic?

Short lane conditions can be evaluated under conditions either of uniform traffic arrivals or of statistically varied arrivals ("stochastic"). There is a small statistical possibility that the first 10 cars arriving at the stop line will all be going straight or all be turning. Traffic models which assume uniform arrivals ignore any such statistical variations. The mixture of turning and non-turning vehicles is a complex distribution, but warrants inclusion because short lanes do not take effect suddenly. The impact is gradual, so that blocked turns may occur for up to half the signal cycles, even if the turn lanes are almost adequate to handle the flows.

This report considers the effects of both uniform and statistically variant vehicle arrivals to compare the predicted results of both models. Each is also compared to a very simple system of vehicle arrivals, lane distribution and turn slot lengths, for which a simple probability tree can be developed as a manual check. These probability trees will also be used as a check on the spreadsheet models for the uniform and statistical short lane models.

The key to all short lane calculations is a determination of whether or not any queue from a lane group blocks out the movement of another. The critical queuing distance is the measure of the onset of lane blockage : it is a function of approach volumes, the effective red Time, the approach volume, peak hour factor, number of lanes on the approach and the static density of the queue, measured in the average vehicle space (typically 20-22 ft.).

In traffic flow terms, the short breaks up the Red Phase into a period when there is no lane blockage and standard traffic queuing applies and a second regime when a turn or lane is blocked from access.

Similarly, the Green Phase is split into two regimes : an initial standard phase when traffic passes through the intersection in conventional flow, and a second regime where the voids are introduced into the flow of the through lane and/or the turn lane -- caused by vehicles in a shared lane branching out into the short lane and leaving voids behind in the traffic flow.

These voids can be seen as "phantom vehicles" which pass over the stop line at part of the saturated flow, but do not count for vehicles actually moved. In other words, the phantom vehicles carry a capacity penalty but no service benefit.

In essence, short lanes have only a negative effect on capacity. Any inclusion of short lanes in the analysis will either show no change or a worsening in level of service. In particular, the effectiveness of Right-Turn-On-Red is diminished significantly by the blockage of right turns.

Consider the case of a free right turn lane on a one-lane approach.

* There are 12 cars going straight and
* 8 cars turning right, and
* The critical queue length is 6 cars to begin blocking,
* THUS 4 cars would turn right before the turn was blocked. AND the final queue would be 16 cars, not 12.
* During the green phase, 12 cars would travel straight through, using the green time required to process 16 cars.

Effectively, 1/3 more Green Time is required to process the traffic -- compared to the normal case when short lane effects are ignored.

Short lane capacity warns us against the error of considering only the number of lanes at the stop line. Lane blockage teaches us that capacity is measured in terms of the efficiency of queuing when the signal is RED. Capacity calculations should be based upon the critical lane queues, not the critical flows. The extra length in the queue represents an effective flow of vehicles, adjusted to be higher than the actual flow. The process is similar to the requirement that a truck represents more than one passenger car equivalent, except that we adjust the demand volume, not the saturated lane capacity.

Driver reaction is important in short lane situations also. Generally, vehicles turning into a blocked lane will queue in the immediately adjacent lane, while drivers using the unblocked lanes may have a choice of lanes. We will assume that when possible, drivers will pick the lane with the shortest queue. In the calculations which follow, all queues will grow equally in each lane (a balanced queue) except for the unusual situation where the turning volume exceeds the through movement -- in which case an uneven queue develops.

SIMPLE TURN BLOCKAGE OF A LANE OR SLOT

The simplest short lane occurs for a through movement blocking a free right turn. We will assume that the right turn does not back up or impede capacity in any way. In normal critical lane analysis, the right turn volumes would be ignored.

The length of the short lane is the blocking distance or D_R. During the peak hour, the volume sufficient to begin blockage, B_{Vol}, is

$$B_{Vol} = \frac{3600 \times D_R}{CL \times SP_J} \qquad \{1\}$$

where CL = the signal cycle length

SP_J = the spacing per stopped car (approx. 22 ft.) ("jam spacing")

The right turns which are NOT blocked become

$$V_{RB} = \frac{V_R \times B_{Vol}}{V_S} \qquad \{2\}$$

The effective through volume is the sum of :

$$V_S + V_{RB} = V_S + V_R - \frac{V_R \times 3600 \times D_R}{V_S \times CL \times SP_J} \qquad \{3\}$$

Including an allowance for the peak hour factor, the effective volume V_{eH} becomes :

$$V_{eH} = V_S + V_R - \frac{PHF \times V_R \times 3600 \times D_R}{V_S \times CL \times SP_J} \qquad \{4\}$$

This equation for the Effective Volume now provides a practical method for considering short lane effects in traffic capacity calculations. Instead of the specified through volume V_S, we use V_{eff} in the capacity calculations.

THE SWEDISH STATISTICAL FLOW EQUATION

The phenomenon of short lanes is summarized in the Swedish equation #55 of Dam Beagan's Fall 1991 Draft Chapter 9 materials. This equation contained 3 typos, which have been corrected in the discussion below. Some changes have also been made in the Swedish/Beagan notation to reduce conflicts with other typical notation in U.S. traffic analysis.

Equation #55 shows the number of vehicles which initially can move into a potentially blocked turn lane, when short lanes restrict free access to these lanes. The equation in question is expressed in terms of V_{free}, the volume of traffic making its way through the short lane and is defined in terms of a summation of factorials and exponents.

$$V_{free} = \sum_{i=1}^{n-L} i \{{}^{i+L-1}Cr_i\} P^i (1-P)^L + \sum_{i=n-L+1}^{n} i \{{}^{n}Cr_i\} P^i (1-P)^{n-i} \quad \{5\}$$

where n = the total number of all vehicles arriving at the stop line in the lane adjacent to the short lane

L = the number of non-turning vehicles that can arrive at the stop line in the lane before entry to the turning lane is blocked

${}^xCr_y$ = the combination of x objects taken y at a time, defined as x! / ((x-y)! * y!)

x! = x factorial

P = the decimal fraction of turning traffic vs. total traffic in the lane adjacent to the short lane

No summary formula for application was presented, such as the "plug in" formulas in the HCM85. It is not clear how to apply the formula or what use its results can be used for.

Equation {5} can be reduced to practical application by recognizing that the total volume in the lane include the volume of turns which are blocked, which is the total right turn vehicles less V_{Free}. Thus the effective through volume is

$$V_{eff} = V_S + V_R - V_{Free} \quad \text{per cycle} \qquad \{6\}$$

and

$$V_{eff} = V_S + V_R - V_{Free} \quad \text{per hour} \qquad \{7\}$$

Now we can compare the two values of V_{eff} from equations {4} and {7}. First, we need an empirical check to determine the exact statistical likelihood of arriving vehicles, as a check to equation {7}. The method is a simple probability tree, using a simplified example whereby $V_S = V_R$ and the percentage of turning traffic is 50% or 0.50. The probability tree looks like this :

```
                                                                     CAR NUMBER

       ___ S ___                     ___ R ___                           1
      /         \                   /         \
     S           R                 S           R                         2
    / \         / \               / \         / \
   S   R       S   R             S   R       S   R                       3
  /\   /\     /\   /\           /\   /\     /\   /\
 S R  S R    S R  S R          S R  S R    S R  S R                      4
 Λ Λ  Λ Λ    Λ Λ  Λ Λ          Λ Λ  Λ Λ    Λ Λ  Λ Λ
SRSRSRSR   SRSRSRSR  SRSRSRSR   SRSRSRSR                                 5
.........  .........  .........  .........                  <-- R BLOCKED
```

The diagram could continue on to additional levels, multiplying by two each time. We can see what we normally know that the probability of the first two vehicles turning right is 1 in 4 or 0.25, which we already know. If If the turn lane is 3 car lengths long, the probability of the first car turning right unblocked is 1/2, and sequentially after that 2/4, 4/8, and 7/16 where the total arriving vehicles **n** is 4. The total number of right turning vehicles not blocked is 1 15/16. For the case of n =5, the probability sequence is 1/2, 2/4, 4/8, 7/16, and 11/32, for a total of 2 9/32. A half dozen other simple combinations were tried, and the results compared with Equation {5}, the Swedish statistic formula. In all cases, the answers were <u>identical</u>, confirming the accuracy of the statistical equation.

With the accuracy of Equation {5} confirmed, the next step was to develop a spreadsheet template which would calculate the Effective volume based on Equations {5} and {7}. Unfortunately, most spreadsheets cannot handle factorials, so a lookup table was used to handle all factorials up to 30. It also became possible to include equation {4} for uniform arrivals and compare the two results. A sample comparison is shown below, for 120 cycles an hour.

n	L	V_{eff} from Eq. {4} (Uniform)	V_{eff} From Eq. {5} (Statistical)	Difference
4	3	240	250	3.0%
5	3	300	326	8.2%
6	3	360	416	15.6%
7	3	480	516	7.4%
8	3	600	622	3.7%
9	3	720	733	1.9%
10	3	840	848	1.0%
11	3	960	965	0.5%
12	3	1080	1083	0.03%

The greatest differences occur right at the point when the uniform- arrival model would predict turn blockage would begin : with 3 vehicles queued going straight and 3 vehicles seeking to make the right turn. Uniform arrivals would say no blocking occurs, but the statistical model estimates an added queue and capacity penalty of 15.6%. As the number of arriving vehicles increases relative to a fixed short lane, the difference between the two equations drops to insignificant levels, because turn blockage becomes the prevailing conditions, not a statistically occasional occurrence. Similarly, as vehicle arrivals decreases below the blocking limit, the statistical chance of blockage drops off and the two equations give virtually the same result.

In conclusion, it appears that the uniform arrival equation {4} is simpler and more easily subject to manual calculation, but the availability of computer spreadsheets allows for us to easily incorporate the statistical complications of Equation {5}.

THE MULTIPLE LANE APPROACH

For the general case of more than one lane on the approach, the volume of unblocked turns V_{Free} is increased by a factor of **N**, the number of lanes :

$$V_{eff} = V_S + V_R - \frac{PHF \times V_R \times 3600 \times N \times D_R}{V_S \times CL \times SP_J} \quad \{8\}$$

It should be noted that the multilane equations {7}and {8} assumes that all queueing occurs evenly. There are many situations where queuing is not even, especially if the right turn flow is quite heavy. In these circumstances, we can define and may be able to measure the queuing "unevenness" U which is the ratio of the typical queue length in the turning lane compared to the average queue length in other through lanes. If the right lane queue tends to be shorter than the left lane queues, U would be less than 1.0 and less blockage would occur.

Now equation {8} for the general multilane case becomes :

$$V_{eff} = V_S + V_R - \frac{PHF \times V_R \times 3600 \times N \times D_R}{V_S \times CL \times SP_J \times U} \quad \{9\}$$

The danger with equation {9} is the uncertainty in the Unevenness factor. When considering a new or extensively modified intersection, there may be no useful empirical evidence at the site to use in setting the queue unevenness. Probably the best approach is to investigate certain key examples of uneven queuing and turn blockage, such as Alewife and the Hartwell Avenue jughandle. It does appear that the amount of unevenness depends on the total volume flow in all lanes as well as the proportion of through vs. turning traffic. Drivers may also out of courtesy (or by not wanting to be honked at) may avoid being the car that causes the block and tend to stay out of the lane next to the short lane.

It is also possible for uneven queues to occur next to the short lane, if the approach volumes are comprised of free-flow traffic generally staying to the right, or resulting from a prior merge which concentrated traffic either to the right or left.

THE BLOCKING RATIO

The onset of turn blockage and equation {9} can be simplified by the definition of a new term, the Blocking Ratio, as ϕ (Phi) or

$$\phi = \frac{CL \times SP_J \times V_S \times U}{3600 \times D_R \times N \times PHF} \qquad \{10\}$$

The Blocking Ratio is a measure of whether or not turn blockage occurs with uniform arrivals. If $\phi > 1.0$, blocking is likely to occur, while if $\phi < 1.0$, the queues are not long enough to cause turn blockage. In effect, when $\phi = 1.0$, this is the point of onset of turn blockage and short lane effects can become significant. If $\phi < 1.0$, we do not need to worry about short lane effects for uniform arrivals.

Now equation {9} can be rewritten to include the Blockage Ratio,

$$V_{eff} = V_S + V_R - \frac{V_R}{\phi} \qquad \{11\}$$

Note that equation {11} is meant to be used for all calculations which normally ignore short lane effects, so that V_{eff} replaces V_S as the approach volume. Furthermore, under no circumstances is V_{eff} ever to be less than V_{eff}.

EQUATIONS ADJUSTED FOR V/S

Equation {6}and {7} for the Effective Volume has been based on fully loaded phases, whereby v/s = 1.0 . The dense queue forms and then clears out, with the red phase coming immediately after the clearance of the queue. In other words, there is no assumption that at the end of the phase there will be a period of free flow traffic typical of signal phases which are not fully loaded. Any free-flow arrivals would not be subject to turn blockage -- so the equations need to be adjusted to account for v/s < 1.0 when moves are not fully loaded.

The best way to understand the implications of dense queue flow and free flow is to consider a simple model with a 120 second cycle, composed of 60 seconds of green and 60 seconds of red. (clearance time is ignored). All arrivals are uniform at 900 vph or one car every 4 seconds and 30 cars every cycle. The lane capacity is 1800 vph or one car every 2 seconds.

At v/s = 1.0, the dense queue of 30 vehicles moves through on green and there are no free flow arrivals served. For v/s = 0.67, the green time is 90 seconds and the red time is 30 seconds, so typically 15 vehicles would depart in a dense queue and 15 would move through as free flow. The extreme condition is v/s = 0.50, when all the traffic moves on green and there is no red time, no queuing and no blockage.

The 1983 NCHRP report includes a short lanes equation which was sensitive to v/s, but was not carried over into the formal Highway Capacity Manual in 1985. The Critical Queueing Distance D_c was defined as the average expected queue in feet for uniform arrivals :

$$D_c = \frac{SP_J \times V_S \times [illegible]}{3600 \times N \times (1.0 - v/s)} \qquad \{12\}$$

The NCHRP equation {12} can be compared with the simple v/s model above with the following results :

v/s	Simple Model	NCHRP Equation
1.00	660	∞
.80	495	1320
.67	330	500
.57	250	190
.50	0	0

The NCHRP equation is very non-linear and appears very wrong at v/s values near 1.0. Clearly, turn blockage is a function of v/s, but there is something terribly wrong with the NCHRP equation. Since the equation was dropped in the 1983-1985 period, we can presume that its errors were recognized at that time. Unfortunately, no replacement formulae were proposed in either HCM85 or any subsequent literature I have seen.

This discussion of v/s effects tells us that likelihood of turn blockage using the Blocking Ratio ϕ can be accurate only under the assumption of fully loaded phases (v/s = 1.0), as well as uniform arrivals. The relationship between v/s and dense queue length was empirically derived from the simple model case and applied to equation {11} to yield

$$V_{eff} = V_S + V_R - \frac{V_R\,(2\,v/s - 1)}{\phi\,(v/s)} \qquad \{13\}$$

The obvious problem with this equation is how to apply it to a capacity calculation -- when we do not know what v/s is. The proper approach is an iterative one, making at least two calculations :

(1) Use Equation {13} to obtain V_{eff} assuming v/s = 1

(2). After the first pass at estimating capacity, take the v/s value obtained and reapply it to Equation {13}
Take the new V_{eff} and calculate capacity again.

Obviously, all of these calculations are complex and inevitably require a computer. The computer can do the recalculations rapidly, as well as factorials and a host of other adjustments. Using a spreadsheet, it will be possible to include V_{eff} into any other capacity calculation, in place of the volume figures which presume no short lane effects.

EQUATIONS FOR LANE FLARES

A Lane Flare typically occurs for the purpose of expanding the number of lanes in a movement. Typically, an example might be the flaring out of a roadway from 2 lanes to 3 lanes of through traffic at an intersection, or flaring of an exit ramp from one ramp to 2 lanes. The right hand lane can be a shared lane with right turns included, but the primary difference between a short turning lane and a short flared lane is that a flared lane includes vehicles making the primary through movement.

With lane flares, the best approach is to consider queue growth and the point at which the short lane flare reduces the number of lanes available for queuing. First, we need to understand queue growth under conditions of uninterrupted long lanes, with vehicles at uniform spacing and speed.

Queue growth occurs faster than the free flow arrival rate because the queue grows back towards the oncoming traffic. With a Jam Spacing of 22 feet, every new vehicle joining the queue will have traveled 22 feet less than the car immediately in front. At 30 MPH, this represents half a second, and for a lane flow of 600 vph, the free flow spacing of 6 seconds would mean that cars are added to the queue every 5.5 seconds. Including the busiest 15 minutes reflected by the Peak Hour Factor **PHF**, the arrival time would be :

$$\underline{\textit{PEAK ARRIVAL TIME}} = \frac{3600 \times N \times PHF}{V_S} - \frac{SP_J}{Vel} \quad \{14\}$$

The queue growth rate in feet per second can be found by inverting Equation **{14}** :

$$\underline{\textit{QUEUE GROWTH RATE}} = \frac{SP_J}{\left[\dfrac{3600 \times N \times PHF}{V_S} - \dfrac{SP_J}{Vel}\right]}$$

$$= \frac{SP_J}{3600 \times N \times PHF/\,V_S \;-0.5} \quad \{15\}$$

The queue length at any time **T** is

$$\underline{\textit{QUEUE LENGTH}}\;(T) = \frac{SP_J \times T}{3600 \times N \times PHF/\,V_S \;-0.5} \quad \{16\}$$

At the end of the Red Time, TR, the light turns green and the front of the queue begins to move out. Meanwhile, cars continue to add to the end of the queue. A "startup wave" is created by the typical driver response time of 1 second. The wave speed is 22 ft / 1 sec. or 15 MPH. Because the end of the queue grows at a maximum rate of 22 ft. every 1.5 seconds or 10 MPH, the queue shortens faster at the front than it grows at the rear. Eventually, the queue disappears when the start-up wave catches up to the end of the queue (the stopping wave). For an approach volume of 600 vph, the stopping wave speed would be only 22 ft / 5.5 sec = 4 fps = 2.7 MPH.

The maximum extent of the static queue occurs at a Wave Time of T_W, measured from the beginning of Green until the queue finally dissipates. Meanwhile, the queuing process has been occurring since the beginning of the Red phase or over a time period $T_R + T_W$.

The startup wave passes through the same queue length in time T_W. The wave speed is 22 fps, so the queue length is 22 T_W. These two equations can be solved to eliminate T_W, with the result that

$$\underline{\textit{FINAL QUEUE LENGTH}} = \frac{SP_J \times T}{3600 \times N \times PHF / V_S - 0.5} \qquad \{17\}$$

EMPIRICAL VERIFICATION

In the late 1980s, I measured an average vehicle spacing of 22 feet for Alewife Brook Parkway in Cambridge at two locations. In prior years, 25 feet has been a typical value, but the widespread use of smaller cars has resulted in denser vehicle packing in queues. The 1983 NCHRP report suggested a spacing of 20-ft., but only in rare samples did I find vehicle spacings as low as 20 ft.

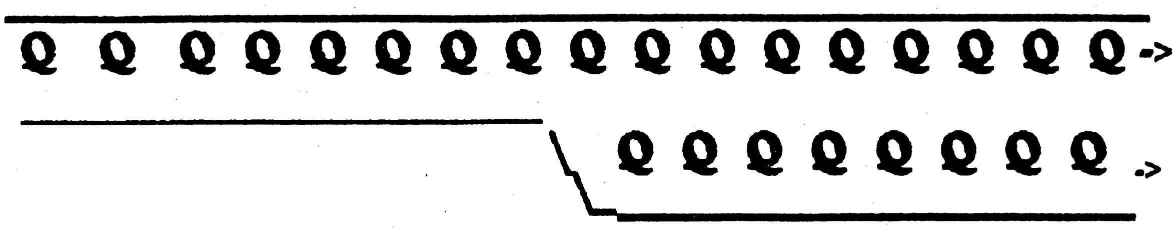
Measurements of the stopping wave were made in the Alewife area under congested conditions (1800 vph rate when moving) and vehicles were stopping at Rindge Avenue at an average interval of 1.5 seconds. This coincides with the arrival rate of one car every 2 seconds, less 0.5 seconds for the backward growth of the queue.

Similarly, at Alewife the startup wave was measured at an average of one second per vehicle. Each driver responds and begins movement of his vehicle with a one-second overall response time.

Another check can be made when queue lengths are relatively large and uninterrupted by driveways. Based on the signal timing at the Alewife and Rindge signal in 1985, equation {14} would predict that the startup wave would catch up with the stopping wave at a distance of 2,000 feet from the stop line. Indeed, field observations confirmed this estimate. Beyond this point traffic tends to move at a fairly steady crawl, without the distinct stop-start characteristics of traffic approaching an intersection.

QUEUE GROWTH WITH LANE FLARES

With the understanding of queuing as described above, we can now apply the queue growth equations to a <u>Lane Flare</u> :



| <----- Stage 2 Queuing ------->|<----------- Stage 1 Queuing --->|

The queuing occurs in two stages, first using all lanes and second when queuing occurs in one fewer lanes. :

$$\underline{\textit{QUEUE GROWTH RATE}}\ (1) = \frac{SP_J}{3600 \times N \times PHF/\,V_S \; - 0.5} \qquad \{18\}$$

and after lane blockage occurs,

$$\underline{\textit{QUEUE GROWTH RATE}}\ (2) = \frac{SP_J}{3600 \times (N-1) \times PHF/\,V_S \; - 0.5} \qquad \{19\}$$

The final queue length with blocking Ψ (Psi) becomes

$$\underline{\textit{FINAL QUEUE LENGTH}}\ \Psi = \frac{SP_J \times TR - 3600 \times PHF \times DR / V_S}{3600 \times (N-1) \times PHF / V_S - 1.5} \quad \{20\}$$

while the calculation of final queue length Π (Pi) ignoring any blocking effects is :

$$\underline{\textit{NO BLOCK QUEUE}}\ \Pi = \frac{SP_J}{3600 \times N \times PHF / V_S - 1.5} \quad \{21\}$$

where **N** is the number of approach lanes at the stop line.

The effective through volume increases proportionally to the growth in queue length due to blocking. The Effective Volume used in capacity calculations becomes :

$$V_{eff} = V_S \times \frac{\text{TOTAL QUEUE}}{\text{NO BLOCK QUEUE}} = V_S \times \frac{\Psi}{\Pi} \quad \{19\}$$

Through the use of equations {17}, {18} and {19}, it is possible to calculate the effective approach volume for a lane flare with short lane effects.

TECHNICAL NOTATION

B_{Vol} = the lane volume necessary to begin blockage of the short lane

CL = the signal cycle length, in seconds

D_C = Critical Queuing Distance (NCHRP)

D_R = the length of the right turn lane

D_S = the length of the right hand flare lane

L = same as D_R

N = the number of through lanes measured at the stop line

P = the decimal fraction of cars in the through lane which are turning right

SP_J = the Jam Spacing or density of cars in a stopped queue

T = any time T seconds

T_R = duration of the Red Phase

T_W = time for the startup wave to reach the end of the queue

U = Uneveness of right lane queues, compared to other lanes

V_{eff} = The effective or equivalent volume resulting from less efficient queuing with short lanes

V_{Free} = The volume of traffic which can turn right before turn blockage begins

V_R = The hourly flow volume of right turns

V_{RB} = The volume of right turns NOT blocked

V_S = The hourly flow volume of through traffic

Vel = Velocity of approaching free-flow traffic

Π = (Pi) Queue length if blocking is ignored (no short lanes)

$\emptyset$ = (Phi) Blocking Ratio (see Eq. {10})

Ψ = (Psi) Total Flare Queue Length with blocking

ΩΩΩΩΩΩΩΩΩΩΩΩΩΩΩΩΩΩΩΩΩΩΩΩΩΩΩΩΩΩΩΩΩΩΩΩΩΩΩΩΩ

APPENDIX F

TECHNICAL REPORT ON

INTERIM ACCESS DESIGN

MAY 1985

Consent Agenda #5 S-500

Transmiting communication from Robert W. Healy, City Manager, relative to Awaiting Report Item Number Seven, regarding the W.R. Grace site.

In City Council October 21, 1996

Referred to the Environment Committee and Traffic and Transportation Committee on the motion of Councillor Duehay,

Copies sent to Councillor Duehay and Councillor Davis mc