Calibration and Application of a Simulation-Based Dynamic Traffic Assignment Model

The paper presents the calibration and application of a simulation-based dynamic traffic assignment (DTA) model on a portion of the City of Calgary road network in Alberta, Canada. The DTA model iteratively re-assigns flow to paths using the Method of Successive Averages (MSA), based on travel times obtained with a traffic simulation model. The original sub-network extracted from a regional planning model, was enriched by greatly increasing the number of zones. The DTA origin-destination (O-D) matrix was estimated from an extensive database of turning movement counts via a trip generation/distribution model and a matrix adjustment algorithm. The network topology was enhanced by the addition of an interchange and a more precise representation of arterial intersections, including traffic signal control plans. A set of one-hour turning counts was used to calibrate the DTA model by adjusting local parameters such as gap-acceptance values, as well as global parameters such as average vehicle length. The final model results were compared to an independent set of 15-minute turning-movement counts. The resulting R^2 values, which ranged from 0.91 to 0.96, lead to a high degree of confidence in the model results.

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Introduction

The purpose of this paper is to present an application and calibration of a simulation-based dynamic traffic assignment (DTA) to a part of the road network of the City of Calgary, Alberta, Canada. The dynamic assignment model used for this application (*1*, *2*, *3, 4*) uses an iterative approach that re-assigns flows to paths on each iteration based on path travel times obtained with a traffic simulation model. The traffic simulator was designed to produce reasonably accurate results with a *minimum* number of parameters and a *minimum* of computational effort (*5*, *6*). However, the underlying structure of the model has more in common with microscopic models (*7*, *8*, *9*, *10*, *11, 12, 13*) than with mesoscopic approaches (*14*, *15, 16*), as it is designed to capture the effects of car following, lane changing and gap acceptance. The simulation is a discrete-event procedure and moves individual vehicles. However, the procedure only calculates explicitly the time at which each vehicle crosses each node on its path: the actual vehicle trajectories are implicit in the solution of the model. This leads to a *drastic* reduction in computational effort relative to microscopic discrete-time approaches, where the computational effort is a function of the total travel time experienced by the drivers.

To the authors' knowledge, no calibration exercises of an iterative simulation-based DTA have been reported in the literature. The existing literature on the calibration of traffic simulation models, of which there is very little, offers some good guidelines and practical advice, but does not address the simultaneous calibration of route-choice and link flows. Hourdakis et al (*17*) reviewed and critiqued several recent calibration efforts involving traffic simulation models, and proposed a practical methodology of their own for linear networks.

They also automated part of their method by devising a means of communication between the commercial traffic simulation package AIMSUN2 (*18*) and the MINOS optimisation package (*19*). Although their methodology is limited to linear networks, and obviously does not address route choice, it offers some useful ideas for the manual calibration of more general networks. Earlier work by Rakha et. al (*20*) established a framework for model development and application that clearly distinguishes between the tasks of verification, validation and calibration, but does not venture very far into actual methodologies for these tasks. Jayakrishnan et al. (*21*) have discussed calibration of networks with route choice, but treat the calibration of the assignment as a separate problem from the calibration of the traffic simulation model.

Any attempt to calibrate a simulation model requires a means to compare the simulation output to corresponding empirical data. The literature on statistical methods for making such comparisons is well established (*22*), and some measures have even been suggested specifically for traffic simulation (*23*). Such measures provide a more accurate picture of the goodness of fit, but they cannot explain *why* the two sets of data are different, nor *how* to appropriately improve the simulation output. This has always been done in an *ad ho*c manner that relies on good judgement, a thorough understanding of the model being used, and an appreciation of the quality of the empirical data that is available. The work reported here does not provide a complete algorithm, or recipe, for the simultaneous calibration of path flows and link flows for general networks. It does, however, provide insight into the importance of route choice in the calibration process, and some general guideposts that may lead to more structured methods in the near future.

The paper is structured as follows. In the following section the issues addressed by the model are presented. A brief description of the simulation-based DTA model is given in the third section. The preparation of the O-D matrix for the DTA is described in the fourth section, followed by a discussion of other inputs and the calibration of the dynamic model in the fifth section. A conclusion regarding the validity of the model and the application ends the paper.

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Issues Addressed with the DTA Model

The City of Calgary, with a population of 900,000 in 2002, is located in southwestern Alberta, Canada. To alleviate traffic congestion in a growing community in southern Calgary, approval was obtained in 2001 to construct an interchange to replace a signalized intersection.

In order to analyze the traffic impact the new interchange could have on the community road network, the DTA model was developed to encompass the whole southern community of 47,000 persons. This was possible since this community is isolated from the rest of the city to the north by a river (Fish Creek) with a limited number of bridges. By including the whole community, the study could take into account the impact of traffic re-routing along the major roadways in the community, as a result of the new interchange. This was advantageous since the interchange design involved the re-alignment of existing roads, as well as construction of new roads, to connect existing and future commercial spaces on both sides of the interchange.

The main reason for choosing a dynamic model over a static model is that the former employ more realistic representations of traffic flow. As a result, dynamic models offer greater potential for capturing reality, and in particular, queuing and congestion. By modeling the delays and queuing due to traffic conflicts, dynamic models provide the transportation engineer with feedback on how traffic volumes and capacity can alter between different design options. Of particular interest were:

- Weaving movements between the signalized intersection and interchange entrance/exit ramps
- Weaving volumes along freeway lanes between ramps
- Queuing and spill-back at saturated signalized intersections
- Queuing and spill-back at freeway merge bottlenecks

Network plots of model outputs provide a visual aid on how interchange design options can impact traffic flow at the interchange as well as indicate where diverted traffic may be causing problems elsewhere in the network. Outputs such as congested speeds, weaving volumes and traffic volumes can be input into capacity analysis software for further analysis. This type of analysis is useful since it provides standard measures of level of service that quantify the quality of traffic flow in most instances. However, at locations where it is difficult to quantify level of service such as at weaving sections between a signalized intersection and interchange ramp, the DTA model itself provides useful insights into the potential capacity reductions at these critical components of the interchange design.

A Brief Review of the DTA Model Used

The dynamic traffic assignment used in this application is based on the notion of temporal network equilibrium, and uses and iterative solution algorithm. In an equilibrium approach, only pre-trip path choices are considered. However, the path choices are modelled as a decision variable and the objective is to minimize each driver's travel time. All drivers have perfect access to information, which consists of the travel times on all paths (used and unused) experienced on the previous iterations. The solution algorithm used here consists of two main components: a method to determine a new set of time-dependent path flows given the experienced path travel times at the previous iteration, and a method to determine the actual travel times that result from a given set of path flow rates. The latter component is referred to as the "network loading problem". Thus, the input to the network-loading problem is the set of time-dependent path flows, while the output is the set of time-dependent path travel times (as a function of the departure time from the origin). Any network-loading model will simultaneously yield the time-dependent link flows, travel times and densities. The network loading problem is solved in this model using a traffic simulator that implements the simplified traffic model developed by Mahut (*5*). This model moves discrete vehicles on a network defined at the level of individual lanes. The underlying mechanism of congestion in the model is the crossing and merging – collectively referred to as *conflicts* – of vehicle trajectories. Such conflicts arise at intersections, and also on roadways, due to lane-changing manoeuvres. The network loading model is described in more detail below.

The Dynamic Equilibrium Concept and a Solution Algorithm

The algorithm requires a set of initial path flows, which are determined by assigning all vehicles to the shortest paths, based on free-flow conditions. The mathematical statement of the dynamic equilibrium problem is not given here but may be referenced in the work of Friesz et al (*24*) or in earlier work on this DTA model (*1*, *2*, *3*). In an equilibrium-oriented approach, the aim of the model is to prevent drivers from using long paths and to divert them to the current shortest paths with the aim of equalizing the travel times from origins to destinations for each discrete departure interval.

A brief description of the algorithm is given in the following. The demand is subdivided into discrete time periods, called departure intervals, and denoted by t . Each interval t has the demand g_i^t for each O-D pair $i, i \in I$. The initialization procedure consists of an incremental loading scheme that successively assigns the partial demand for each interval t onto dynamic shortest paths. In the successive iterations the path input flows h_k^t , for each path $k \in K$, are determined by the method of successive averages (MSA), which is applied to each O-D pair *i* and departure interval t . Starting at the second iteration, the time-dependent link travel times obtained by the network loading model are used to compute new path travel times s_k^t , for each path *k* and each departure interval t (i.e., for used and unused paths). The travel time on the shortest path for O-D pair *i* and departure interval t is denoted u_k^t . For each iteration *l*, up to a pre-specified number of paths per O-D pair, *N*, the set of new shortest paths is added to the current set of paths and the volume assigned as the input flow to each path is g_i^t / l , $i \in I, \textit{all } t$. After that, for $l > N$, only the shortest among *used* paths is identified: no new paths are added, and the path input flow rates are redistributed. The algorithm may be stated as follows.

Dynamic MSA Equilibration Algorithm

- Step 0 $l = 1$; compute dynamic shortest paths based on free-flow travel times, and load the demands to obtain an initial solution; $l = l + 1$;
- Step 1 If $l \le N$, compute a new dynamic shortest path and assign to each path $k \in K$ the input flow g_i^t / l

If $l > N$, identify the shortest among used paths and redistribute the flows as follows:

$$\begin{aligned} h_k^{t\,(l)} &= h_k^{t\,(l-1)}\left(\frac{l-1}{l}\right)+\frac{g_i^t}{l}, \quad if \;\; s_k^{t\,(l-1)} = u_i^{t\,(l-1)} \\ h_k^{t\,(l)} &= h_k^{t\,(l-1)}\left(\frac{l-1}{l}\right) \quad otherwise \end{aligned} \qquad (1)$$

for $k \in K_i, i \in I \; and \; all \; t$

- Step 2. If *l* is less than a pre-specified maximum number of iterations, or $RGap \le e$, STOP; otherwise return to Step 1

While no formal convergence proof can be given for this algorithm, since the network loading map does not have an analytical form, a measure of the gap (as used in static network equilibrium models) can be used for qualifying a given solution. It is the difference between the total travel time experienced and the total travel time that would have been experienced if all vehicles had the travel time (over each interval t) equal to that of the current shortest path.

Hence:

$$RGap^{t\,(l)} = \frac{\sum_{i\in I}\sum_{k\in k_i} h_k^{t\,(l)} s_k^{t\,(l)} - \sum_{i\in I} g_i^t \; u_i^{t\,(l)}}{\sum_{i\in I} g_i^t \; u_i^{t\,(l)}} \qquad (2)$$

A relative gap of zero would thus indicate a perfect dynamic user equilibrium flow.

The Network Loading Model

The network loading problem is solved here using a traffic simulation model that moves individual vehicles according to the triangular flow-density relationship. In the case of a one-lane link, the model is the discrete-flow equivalent of the simplified kinematic wave model of Newell (*5*, *25*). Rather than computing a real-valued amount of flow that can pass a point (node) over a fixed time interval (as in Newell's model), this model calculates the real-valued duration of time that separates fixed-size increments of flow (individual vehicles) at a given point.

In this model, congestion is caused by the conflicts between vehicle trajectories. A conflict exists between two vehicles when, given their positions at one moment in time, their desired arrival times to the same downstream position violate a constraint that specifies the minimum time separation between the vehicles at that point. Conflicts can arise at nodes and on multi-lane links. In order to satisfy a minimum headway constraint, it must be decided which vehicle is to precede the other, and thus which vehicle is to be delayed. A relatively simple gap-acceptance model has been implemented to determine precedence between vehicle conflicts at nodes. Conflicts on links between vehicles changing lanes are resolved by giving precedence to the vehicle that is further ahead, i.e., without using a gap-acceptance model. The gap-acceptance model employed at nodes consists of a linear probability (density) function that takes a single *critical-gap* parameter, *G*. The probability increases from zero to one over the domain $(G/2, 3G/2)$. A second linear function is used to capture what is often referred to as the *gap decay* phenomenon, which captures driver impatience as waiting time for an acceptable gap increases. This is addressed with a second linear function, based on a *critical-wait* parameter, *W*, which increases from zero to unity over the domain $(W/2, 3W/2)$. It should be noted that the second function is not a gap decay function: it is simply a second precedence probability function. The actual precedence probability is taken as the maximum over the values provided by the two linear functions, as follows:

$$P = \max\left[\min\left(\frac{g}{G}-0.5,1\right),\min\left(\frac{w}{W}-0.5,1\right)\right] \qquad (3)$$

where:

P = the probability that the lower priority vehicle precedes the higher priority vehicle
g = the available gap
w = the relative waiting time between the two vehicles at the node

Once a conflict has been identified and resolved, and the appropriate delay has been calculated, this delay (or a residual portion of it) may propagate upstream, from one vehicle to another, against the direction of the traffic flow. The propagation of delay is modelled through a simplified car-following model that is consistent with the triangular flow-density relationship.

This traffic model is solved in a very efficient way using an event-based (discrete-event) algorithm, which requires only a fraction of the calculations that are normally executed by a traditional time-step simulation approach. In an event-based simulation, each temporal process modeled is associated with a specific sequence of events, and each event is associated with a real-valued point in time. For instance, an event may be associated with a change of phase at a traffic signal, or the time at which a vehicle enters an intersection.

In contrast to continuum traffic models and static assignment models, most traffic simulators model the movement of vehicles on individual lanes. How drivers utilize the available lanes of a roadway can have a significant, even drastic, impact on both the total delays experienced and how these delays are distributed (spatially and temporally) in the network. Since the traffic model is lane-based, each link must be defined by a number of lanes. Each lane is furthermore defined by an access code that determines which classes of vehicles may use the lane (e.g., taxi, bus, HOV, etc…). A length and speed limit furthermore define each link. At each node (intersection) of the network, a movement is defined from each incoming link to each outgoing link. Each movement is defined by an access code and a saturation flow rate per lane. The lanes that may be used to execute each movement must also be specified, along with the traffic control plans at signalized intersections.

Unlike most micro-simulation models, the network definition does not require geometrical information such as lane width, turning angles, and the dimensions of intersections. Moreover, vehicle trajectories within links are modelled implicitly rather than explicitly. Thus, each driver must choose the lanes by which he/she will enter and exit a link just before actually arriving to the link and, once on the link, the choice cannot be re-considered. The principal argument behind using such an approach is that it is sufficient to model only mandatory lane changes in order to reproduce the general congestion patterns resulting from a given set of path flows. Mandatory lane changes are those that must be made in order to exit and enter each link on the lanes permitted for the associated turns. The lane-choice rules consider the permitted lanes over a sequence of downstream turns when some of the lanes immediately downstream of the driver are queuing and some are not. This logic allows a driver to join the queue if necessary, or to by-pass it if his/her path does not go through the head of the queue.

Development of the DTA Trip Table

The origin-destination matrix is an essential input to any dynamic assignment model. The City of Calgary currently simulates the morning peak hour traffic demand by using a traditional four-stage static assignment model using the EMME/2 software package. The level of detail of the zone system and network in this regional model is adequate for planning major road and transit needs over the medium and long-term time horizons. However, the zone system and network of this regional model was not of sufficient detail to provide a trip table suitable for the DTA model. For this reason, a sub-area static model was built for the area south of Fish Creek that expanded the number of zones in the regional model from 12 to 77. The detail of the road network was also expanded to include key local roads as well as all roadways with a classification higher than a local roadway.

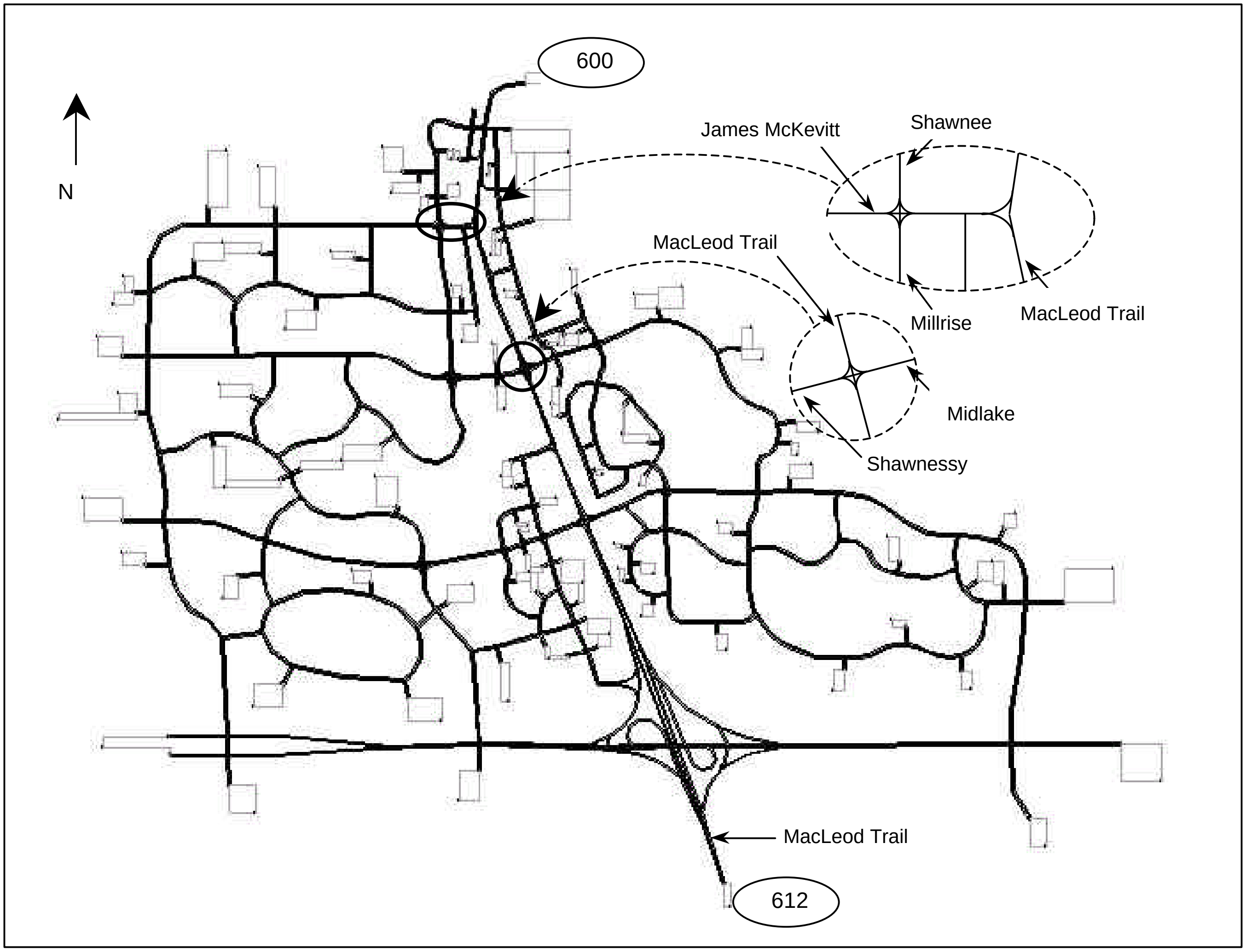
An extensive intersection turning movement count survey was carried out in 2001 to collect traffic volumes at 72 intersections between 6:00AM and 9:00AM. Analysis of the traffic counts indicated that morning peak hour traffic occurred from 7:00AM to 8:00AM. Traversal auto vehicle trip matrices were output from the regional planning model to provide external trip matrices by trip purpose for the sub-area model.

Using the City's GIS database, the land use data for each of the sub-area zones was obtained and entered into the sub-area model databank. Trip generation and distribution models were developed to provide the auto vehicle travel demand models.

For the EMME/2 network eventually used for the DTA application, link capacities were quantified based on the saturation flow volume of the link, using the empirically estimated capacity of the downstream intersection. The posted speed limit and the link capacity were used to determine the appropriate volume delay function to use for each link in the static assignment model. The first part of the calibration process consisted of calibrating the trip generation and distribution model to the link volume contained in a series of screen-lines within the study area. This part of the calibration process was to ensure that the magnitude of traffic entering and exiting the MacLeod Trail corridor (see Figure 1) was within 10 percent of the observed counts.

The second calibration step consisted of fine-tuning the network to obtain a close approximation of simulated turn movements at MacLeod Trail intersections as well as the first adjacent major intersection of the cross streets on either side of MacLeod Trail. This involved refining the location of centroid connectors and applying a limited number of turning movement volume delay functions. The final calibration task involved modifying the 2001 trip table using a matrix adjustment algorithm. The algorithm [which is the gradient method of Spiess (*26*)], which is implemented in the EMME/2 software package, modified the trip table to minimize the difference between the intersection counts and simulated turn movements at the key intersections within the

Figure 1 - Final network plot showing key intersections and zones



corridor.

Calibration of The DTA Model

This section describes the calibration exercise that was conducted for the DTA model, which consisted of adjusting parameters within the traffic simulation model which in turn provided the travel times to the route-choice algorithm. The data requirements of the DTA model beyond the imported data from the static assignment model network are also discussed.

Running the DTA Model

The key parameter in defining a dynamic traffic assignment is the discretization interval, denoted by *t* in the above discussion on dynamic equilibrium, which is the temporal resolution of the dynamic assignment. These intervals will be referred to as *departure intervals* in the following, as the assignment related to these intervals describes decisions made by drivers at the time of departure from their respective origins. The assignment of the demand (trip table) to the paths for each O-D pair remains constant over each departure interval, but may change from one interval to the next. Other parameters required for defining an execution of this dynamic assignment model on a particular scenario are the maximum number of paths for each O-D pair, the aggregation interval for the output statistics, and the maximum number of iterations.

Six departure intervals were used for the 1-hour (7:00AM to 8:00AM) matrix of the Calgary sub-network, each one 10 minutes long. The output statistics were aggregated by 4-minute intervals and the model was run for forty iterations. The MSA algorithm redistributes progressively less flow with each subsequent iteration, so that eventually it must converge by definition. The relative gap was seen to be virtually unchanging (less than 0.1% per iteration) for this application by around thirty iterations. By forty iterations, it was clear that no more improvement towards equilibrium was being made, or would likely be made with further iterations. The outputs of the model include link statistics (flow, density, travel time) aggregated by exiting turn, as well as the flows assigned to the paths by the routing algorithm for each departure interval.

Building the DTA Model

The sub-network and corresponding trip table described above were imported from the planning software package (EMME/2) into this model (implemented in a prototype software package which is currently called *DTASQ*). Since vehicle conflicts at intersections are the main cause of delay in a traffic simulation model, intersection parameters are the critical inputs in defining this kind of network. The network model was relatively quick to build since a good portion of the network data was taken directly from the planning model.

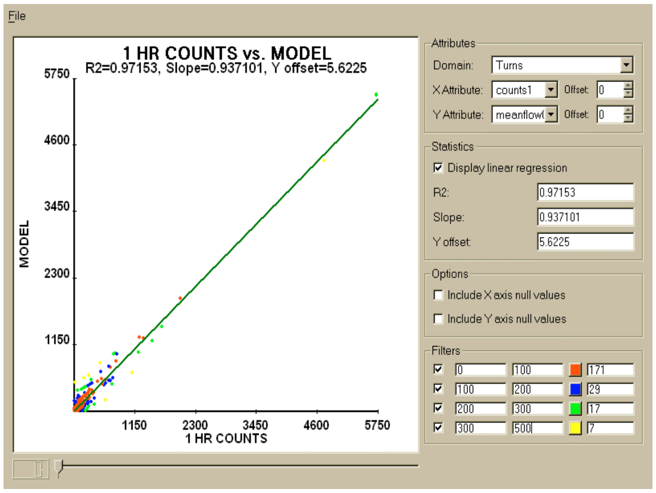
Once imported into the *DTASQ* software a number of modifications were made through the graphical-user interface (GUI). Aerial photographs were used to code the number of lanes on the approaches to fifteen major intersections in the network. The interchange in the lower-central part of the network was also coded in this way. Signal timing plans for eighteen intersections were also input through the GUI. The parameters (attributes) imported with the EMME/2 network were the length, number of lanes and free-flow speed of each link. Along with the intersection modifications and signal timing data, the number of lanes and the saturation flow rate on each turning movement in the network were entered to complement the imported network data. Lastly, the model required the definition of gap-acceptance parameters for “uncontrolled conflicts” at intersections. Uncontrolled conflicts occur at signalised intersections, highway merges and roundabouts. These conflicts can be identified automatically in the software and assigned default gap-acceptance values. These parameters are key to the calibration process.

Figure 1 shows the final network after the necessary modifications were made to the imported EMME/2 network. The two zones identified at the top (north end) and bottom (south end) of the network are connected by the main north-south artery, MacLeod Trail. The key nodes in the analysis are the intersections of (1) MacLeod Trail and Shawnessy/ Midlake, (2) MacLeod Trail and James McKevitt and (3) James McKevitt and Millrise/Shawnee.

Empirical Data

The model was calibrated using a set of one-hour turning movement counts comprising some 250 turns. These counts were imported as turn-based (movement-based) attributes and were compared with the model results using a scattergram. The final scattergram (after calibration) is shown in Figure 2. After calibration, the model outputs were compared with an independent set of 15-minute counts representing 156 turns. Since the parameters used to calibrate the model – primarily gap-acceptance values – were static in nature, it was decided to use the one-hour counts for calibration, and the 15-minute counts to independently verify the calibration afterwards. In this application, a ‘warm-up’ matrix was not used to precede the one hour matrix that was prepared for the study. As a result, the first few minutes (not more than 10) of output were not usable as the network was still filling up. The hourly simulated flows reported in the data are in fact averaged only over the period 7:10-8:00 in order not to be influenced by this issue.

Figure 2 - Scattergram of 1-hour turning volumes (250 turns)



Domain: Turns
X Attribute: counts1 Offset: 0
Y Attribute: meanflow(Offset: 0
☑ Display linear regression
R2: 0.97153
Slope: 0.937101
Y offset: 5.6225
☐ Include X axis null values
☐ Include Y axis null values

☑	0	100	🟥	171
☑	100	200	🟦	29
☑	200	300	🟩	17
☑	300	500	🟨	7

The most common difficulty with the data used in any calibration effort is the consistency between the network, O-D matrix and empirical counts. In this exercise, an O-D matrix obtained originally from a household survey was adjusted using aggregate traffic counts in order to represent current year conditions. The network and traffic control data represented the ‘ideal’ network conditions as far as possible. However, during the calibration work, it was realised that there had been a construction zone in the sub-area network during the period when the traffic counts had been made. Since no data could be obtained regarding the construction work, the resulting capacity reductions could not be incorporated into the model. Some of the most significant discrepancies in turning flows in the final calibration are on turns that are known to have been affected by the construction work.

Another consideration is that the duration of a construction project tends to have an impact on whether the capacity reduction will result in changes to drivers' route-choice behaviour. If the construction had only been short term, it would not have been appropriate to simply include the capacity reduction in the model because the drivers in the model would have found alternate paths, resulting in lower congestion than would be observed in reality.

Observations and Analysis

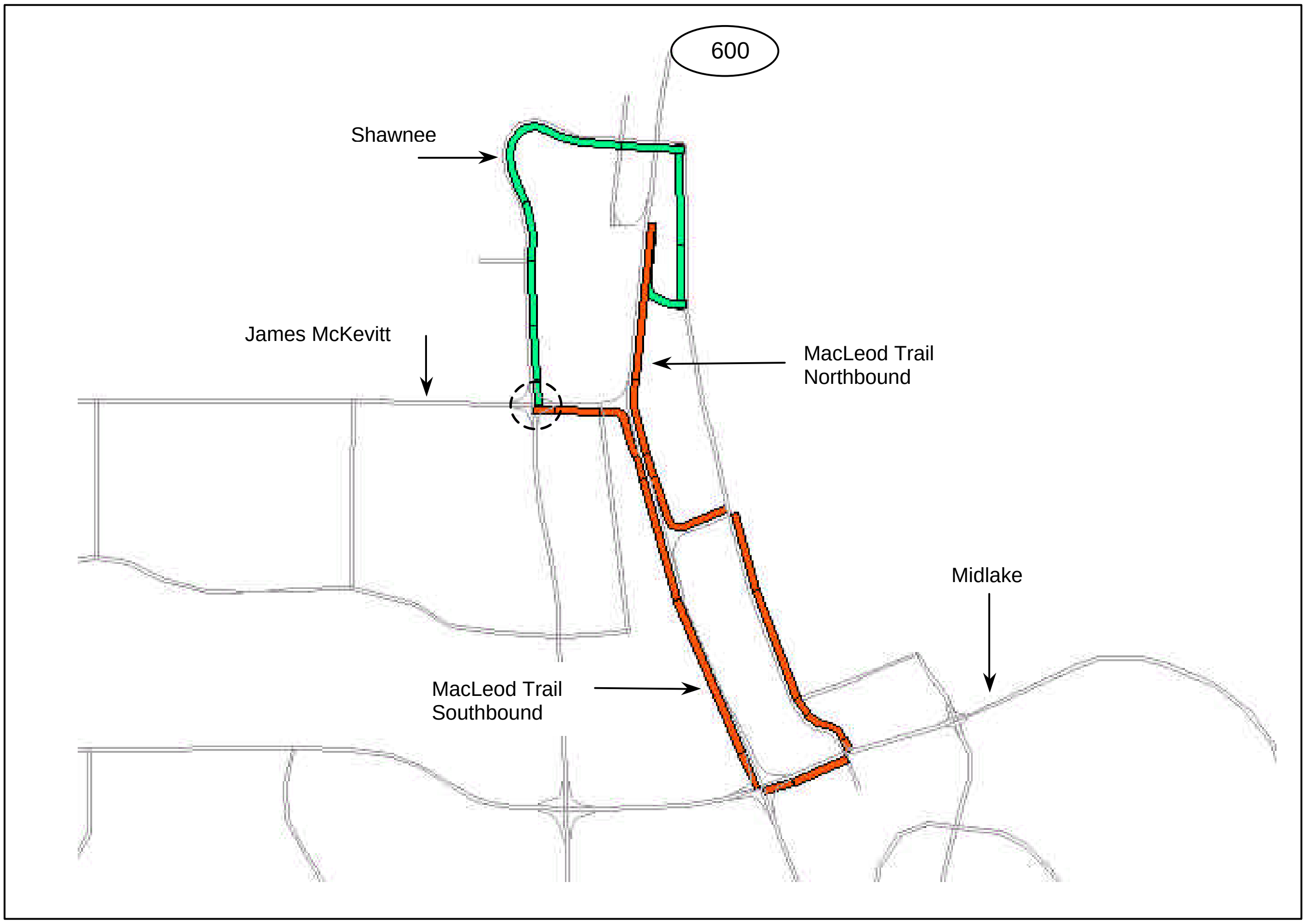
The key aspect of the approach employed here was to use the observed discrepancies between empirical counts and model outputs to identify partial paths that were under or over utilized, rather than trying to address the discrepancies one turning movement at a time. It was often the case that an underutilized path and an overutilized path (leading to the same destination) emerged from the same node, suggesting that the discrepancies on the two paths were due to a single cause. Since the assignment model is based on (approximately) equilibrating path times, rather than on pre-defined turning proportions, the cause of such discrepancies had to be explicitly identified. This was generally the most challenging aspect of the calibration work. The calibration methodology was thus fairly systematic in that one – or at most a few – plausible causes were identified that could simultaneously explain a number of discrepancies (between simulated and empirical counts).

These causes were identified through logical reasoning, albeit *inductive* reasoning, rather than *deductive* reasoning. As a result, it was not possible to produce a formula or recipe for how to calibrate such a model based on what was learned. What can be put forth conclusively from this work, in the way of calibration methodology, is that using the observed discrepancies to identify paths, and then looking for specific causes that might correct several paths simultaneously, is an effective strategy that should be further explored and refined.

The one-hour counts indicated that at the intersection of James McKevitt and Millrise/Shawnee, rather high flow was predicted by the DTA model leaving the intersection to the east and going south on MacLeod Trail, while too little flow was leaving northbound by Shawnee. Identification of other turning movements showing significant discrepancies with the empirical data, and some intuition on the part of the City of Calgary staff, led to the identification of the two paths shown in Figure 3 leading from the intersection of James McKevitt and Millrise/Shawnee to the northbound exit from the network on MacLeod Trail. As it was suspected that the discrepancies in the flows leaving this intersection could be primarily due to flows exiting to this zone, the path flows destined for this zone were plotted on the network.

Figure 4(a) shows the plot of these path flows for the sixth departure interval, i.e., for vehicles leaving their respective origins between 7:50 AM and 8:00 AM. The width of each link is proportional to the sum of the flows assigned to all paths that use the link. Numerical values are shown next to certain links. It is seen that the flow going north on Shawnee is less than twenty five percent of the flow going east on James McKevitt. Knowledge of the real-world traffic routing habits, and intersection turning movement counts indicated that the majority of the flow should in fact be using the path

Figure 3 – Two paths from James McKevitt and Shawnee to zone 600



through Shawnee.

The counts also indicated that the model was predicting a high right-turn flow from MacLeod Trail onto James McKevitt, and a correspondingly low flow on the through movement continuing southbound on MacLeod Trail. Further investigation revealed rather low predicted flows on the southbound MacLeod Trail approach to Shawnessy/Midlake, despite free-flow conditions on the three downstream (exiting) legs of this intersection.

Since only the left-turn movement (onto Midlake) was operating at capacity, this indicated that there was significantly more demand than capacity (or 'supply') for the left-turn movement, so much so that the vehicles were spilling *over* laterally (as well as spilling *back* upstream) out of the left-turn lane and cutting off capacity for the through movement.

As this does not occur in reality at this intersection, it was assumed that the model was overestimating the demand for this turning movement, since the parameters related to the capacity of this turn appeared to be correct. This discrepancy was associated with the O-D flow originating at the north end of MacLeod Trail and destined for the south end of this road.

This was verified by plotting the path assignment for this O-D pair on the network, as seen in Figure 5(a), which shows the path flows for the sixth departure interval. This figure shows that over half of the flow for this O-D pair leaves MacLeod Trail at James McKevitt, only to return to MacLeod Trail further south in order to exit the network. Once again, such behaviour is not consistent with the real-world traffic conditions on this network. A distinguishing characteristic of the path in Figure 3 via James McKevitt is that it consists of several left turns, while the alternative path has only one. Moreover, the left turns on the former path have very high opposing through flows, and thus the attractiveness of this path may be highly sensitive to the left-turn gap-acceptance parameters. One possible explanation was thus that these parameters were too low, which would result in higher 'effective' capacities for these turns, and thus lower delays.

This could explain the most significant discrepancies observed in the network. As noted above, the left turn from MacLeod Trail to Midlake was so over-saturated that the through flow was being significantly impeded, causing the southbound flow to use unrealistic circuitous paths.

Increasing the values of the left-turn gap-acceptance parameters had a significant impact on the resulting assignment, as shown in Figures 4(b) and 5(b). The new assignment yielded a closer correspondence with the empirical turning counts, and produced much more realistic path assignments. Figure 4(b) shows the new path flows destined for the northbound exit by MacLeod Trail in the vicinity of the intersection of James McKevitt and Millrise/Shawnee, with a roughly 2:1 ratio in favour of the northbound route via Shawnee. Figure 5(b) shows the new path flows originating at the north end of MacLeod Trail and

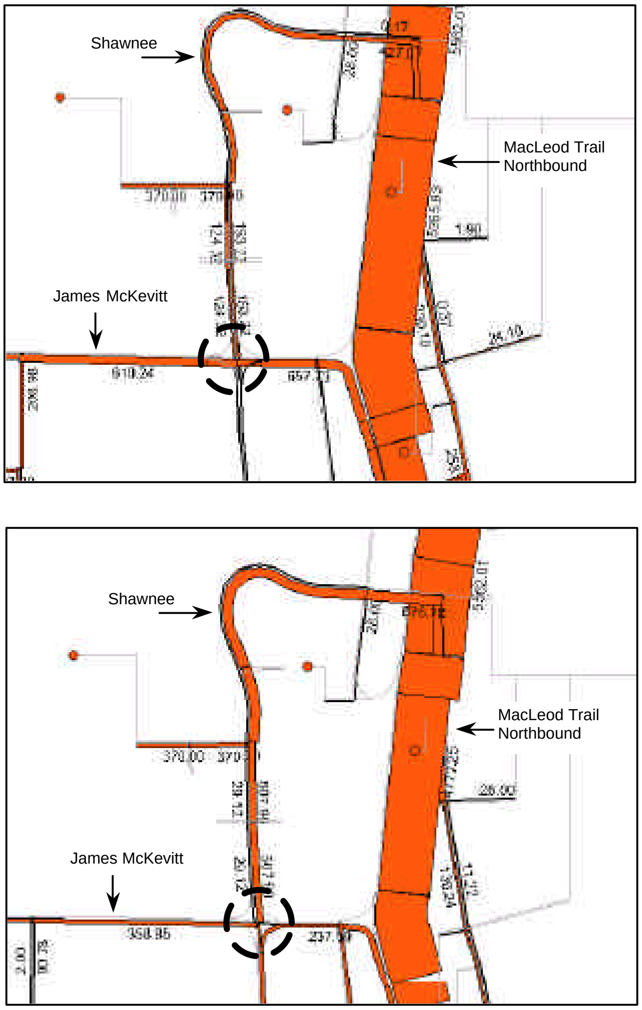


Figure 4 – Interval 6 path flows to zone 600 (a) before calibration (b) after calibration

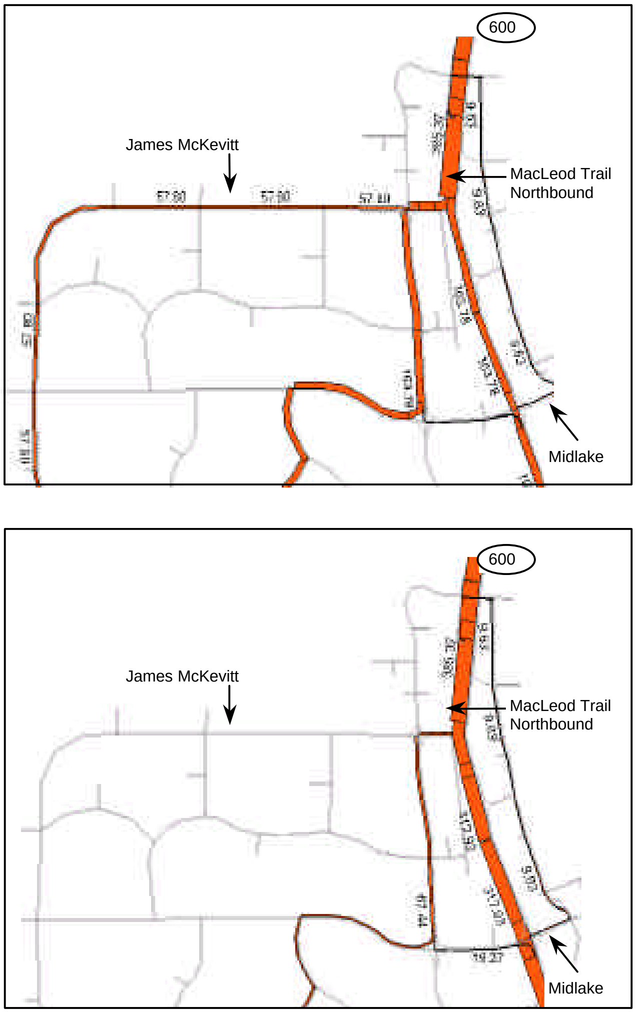
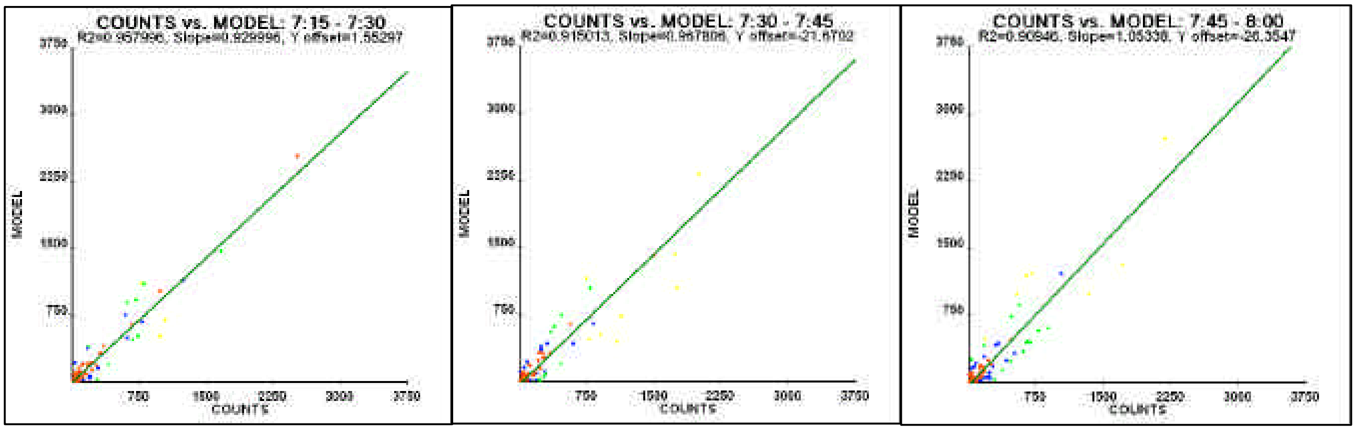


Figure 5 – Interval 6 path flows for O-D 600-612. (a) before calibration (b) after calibration

Figure 6 – Scattergrams for three 15-minute periods (156 turns)



destined for the south end, at the intersection of MacLeod Trail and James McKevitt. Only 18% of these vehicles now turn right at James McKevitt, and the most circuitous path that continues west on James McKevitt is no longer used at all. Another parameter that was used to calibrate the model was the “passenger car unit” (pcu) length, which was decreased from 5.56m (or a jam density of 180 pcu/km/lane) to 5.0m (200 pcu/km/lane).

After calibrating the data against the one-hour counts, the turn flows were compared to an independent set of 15-minute counts located at 13 intersections and representing 156 turns. The scattergrams for the three intervals from 7:15AM to 8:00AM are shown in Figure 6.

It was to be expected that the discrepancies would increase simply because of the shorter time windows used for comparison. It was thus very encouraging to find that the scatter is still well contained on these three plots, with R^2 values ranging from 0.91 to 0.96. Based on this calibration, the DTA model can be used to simulate travel demand in 2004 as discussed in Section 2, where the intersection of MacLeod Trail and Shawnessy-Midlake will be replaced by an interchange. This would be implemented by updating the road network and importing a 2004 O-D trip matrix into the DTA model.

Conclusions

An iterative dynamic traffic assignment (DTA) model that seeks to approximate dynamic equilibrium conditions using a traffic simulator was run on a sub-network of the City of Calgary, Canada, consisting of 734 links, 314 nodes and 77 zones. The DTA was calibrated using a set of 250 one-hour turning movement counts, and the results were later compared to an independent set of three consecutive 15-minute counts on 156 turns (at 12 intersections). Path travel times were calculated by the routing algorithm by considering link travel times aggregated by exiting turn. The routing logic was thus sensitive to differential queuing on multi-lane links that depended on both the demand for, and the capacity (supply) of, each turning movement.

The calibration methodology was to first identify partial paths that were under- or over- utilised, through analysis of the discrepancies between simulated and empirical turning movement counts, rather than addressing the discrepancies one turning movement at a time. In this way, changing the value of a single well-chosen parameter could simultaneously reduce the discrepancies on several turning movements, sometimes on the order of ten or more. The calibration thus focused on errors in the assignment as the primary cause of the discrepancies between simulated and empirical counts, and attempted to improve the assignment only through the adjustment of the simulation-model parameters. By exploiting the assumed relationship between route-choice and travel time (i.e. equilibrium assignment), this work presents a first attempt at the simultaneous calibration of path and link flows.

It was found that traffic parameters, such as gap-acceptance parameters, can have a significant impact on route choice in an iterative equilibration approach to DTA.

Despite the lack of empirical knowledge about path flows in the network, it was found that qualitative judgement of the path flows assigned by the model, based on first-hand knowledge of the network, could be used to identify which parameters to adjust.

Nevertheless, the calibration method presents a strong case for the usefulness of path-based empirical data, as might be available through technologies such as GPS and cell phone tracking. The possible availability of such data in the future suggests that the underlying path-based approach to calibration employed in this work could eventually be embedded in an automated procedure.

The fact that the calibration effort simultaneously improved both the turning movement flows (by comparison with the traffic counts) and the path flows (by comparison with first-hand knowledge of the network) strongly testifies to the validity of the traffic simulation model and the equilibration method used for routing. After calibrating the model using a set of one-hour turning-movement counts, the results were compared to an independent set of 15-minute turning counts, over three consecutive time intervals, using scattergram plots. Given that the shorter time interval for the counts would in itself result in reduced R^2 values, the rather modest reduction that was observed was a very encouraging result. It should be noted that this calibration was executed fairly quickly with the intention of obtaining a *reasonable* fit to the empirical data. In consequence, we consider these to be *very* promising results, and strongly recommend that the methodology be tested on networks of differing size and connectivity. The City of Calgary expects to continue working with this simulation-based DTA model and to test its validity on future projects before and after completion.

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