Transportation (2012) 39:1259–1280
DOI 10.1007/s11116-012-9395-8

Tradeoffs among free-flow speed, capacity, cost, and environmental footprint in highway design

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Published online: 12 April 2012
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Abstract This paper investigates differentiated design standards as a source of capacity additions that are more affordable and have smaller aesthetic and environmental impacts than modern expressways. We consider several tradeoffs, including narrow versus wide lanes and shoulders on an expressway of a given total width, and high-speed expressway versus lower-speed arterial. We quantify the situations in which off-peak traffic is sufficiently great to make it worthwhile to spend more on construction, or to give up some capacity, in order to provide very high off-peak speeds even if peak speeds are limited by congestion. We also consider the implications of differing accident rates. The results support expanding the range of highway designs that are considered when adding capacity to ameliorate urban road congestion.

Keywords Highway design · Capacity · Free-flow speed · Parkway

Introduction

Many analysts and policy makers have argued that building more highways is an ineffective response to congestion: specifically, that it is infeasible to add enough highway

An earlier version was presented to Third Kuhmo-Nectar Conference, Amsterdam, July 2008 and at the poster session of the University of California Transportation Center—Partners for Advanced Transit and Highways Conference, November 2008.

Electronic supplementary material The online version of this article (doi:10.1007/s11116-012-9395-8) contains supplementary material, which is available to authorized users.

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capacity in large urban areas to provide much relief. The argument is supported, for example, by examining the funding requirements for a highway system that is estimated to accommodate travel at acceptable levels of service. Such requirements come to hundreds of billions of dollars for the entire US (FHWA 2006). This observation, along with impediments to various other measures, has led Downs (2004) to conclude that there is little chance for a resolution to congestion problems in the US.

Much of the expense envisioned in such lists of needs is for new or expanded expressway routes.[1] Expressways are very intensive in land and structures, requiring great expense and disruption to existing land uses. Furthermore, the resulting road has significant environmental spillovers on surrounding neighborhoods. These spillovers, including air pollution, noise, and visual impact, are closely related to the size of the road and even directly to speed itself. Thus, there are significant tradeoffs when deciding what kind of free-flow speed should be provided by a given highway investment.

Many of the highest-capacity roads in US urban areas conform to the very high design standards of the federal Interstate system. These standards, which specify lane width, sight distance, grade, shoulders, and other characteristics (AASHTO 2005), are mainly dictated by two underlying assumptions: the road must be safe for travel at high speeds (typically 55–70 miles/h in urban settings), and it must be able to carry mixed traffic including large trucks. But does it make sense to build high-speed roads that will be heavily congested, and thus used at low speeds, for large parts of the day? And does it make sense to design every major route within a large network to accommodate large trucks?

One way to look at the problem is in terms of equilibration of travel times. In a heavily congested urban area, higher-quality roads become congested more severely than others, so that levels of service tend to be equalized (Pigou 1920, Downs 1962). When this occurs, the extra expense incurred to raise the design speeds on major roads has no payoff during congested periods, whereas anything to improve capacity has a huge payoff. Clearly, then, one key factor determining the ideal highway design will be the ratio of peak to off-peak traffic.

We can illustrate the tradeoffs by considering lane width. The standard 12-foot-wide lanes of US interstate highways provide safety margins for mixed traffic at high speeds, often in difficult terrain; but on most urban commuting corridors, there are fewer trucks and relatively flat terrain. Indeed, urban expressway expansions are sometimes carried out by converting shoulders to travel lanes and restriping all lanes to an 11-foot width. These have a disadvantage of slower free-flow travel, and perhaps of higher accident rates—although as we shall see the evidence on safety is mixed. The point here is that by squeezing lanes and shoulders, more capacity can be obtained at the expense of some other desirable features.

In this paper, we examine just a few of the tradeoffs involved by considering examples of pairwise comparisons between two urban highway designs, in which as many factors as possible are held constant. We first consider different lane and shoulder widths for a given highway type (expressway or signalized arterial). In the case of the expressway, this really amounts to a reconsideration of the "parkway" design that prevailed in the US prior to 1950.[2] We then compare expressways with high-performance unsignalized arterials. In

[1] See FHWA (2006), Ch 7. Of the $84.5 billion in investments in urban arterial and collector roads meeting defined cost-benefit criteria in this report, 53% is for freeways and expressways—of which about half is for expansion, half for rehabilitation or environmental enhancement (Exhibit 7-3).

[2] We do not attempt to account quantitatively for the truck restrictions, tighter curves, or nicer landscaping that may further enhance parkway amenities. We suspect that in some cases it would be optimal to restrict trucks altogether, but an explicit model of truck restrictions would be far more complex and is not undertaken here. Safety implications are discussed further in the section on "Safety".

each comparison, we characterize the range of conditions under which one or the other road provides lower travel time, or the same travel time at lower cost (including user costs).

We use a highly stylized model to illustrate the possibilities, rather than providing a design tool for a specific situation. Several of our simplifying assumptions, as described in the next section, probably understate the extent of congestion and thus understate the advantage of a higher-capacity but slower road.

Congestion Formation, Capacity, and Travel Time

We consider four determinants of travel time on a highway. First is free-flow speed, which for expressways is specified as a function of highway design including lane and shoulder widths. Second is slower speeds due to the rising traffic flow when it is still less than the highway's capacity; this is described by speed-flow curves. Third is further congestion delay due to queuing when demand exceeds capacity; this is described by a simple bottleneck queuing model. Fourth is control delay, which applies only to arterials with signalized intersections and which accounts for the effects of traffic signals. In this section, we derive peak and off-peak travel times from basic relationships involving these four determinants. In the next two sections, we relate the parameters of these relationships numerically to highway design parameters, and carry out the comparisons described earlier.

The first two determinants just described are summarized by a speed-flow function, $S(V)$, giving speed as a function of flow. We define it for $0 \leq V \leq V_K$, where V_K is the highway's capacity, defined here as the highest sustainable steady flow rate.[3] For expressways, speed decreases as the flow rate approaches the expressway's capacity. Figure 1 shows the speed-flow curves for various free-flow speeds for expressways, based on the 2000 edition of the Highway Capacity Manual (HCM) (Transportation Research Board 2000). The free-flow speed is $S^0 \equiv S(0)$. For signalized urban arterials, the HCM speed-flow curve is flat, i.e., the speed on the urban street remains at the free-flow speed for all flow rates up to the road's capacity V_K; this capacity is determined by the saturation flow rate at intersections (the maximum flow rate while the signal is green) multiplied by the proportion of the signal's cycle time during which it is green.

The third determinant, queuing delay, may be approximated by deterministic queuing of zero length behind a bottleneck (Small and Verhoef 2007, Sect 3.3.3).This simplification ignores possible effects of spillover queues on other roads, and thus might understate the advantage of higher-capacity roads which have shorter queues. For the sake of concreteness, we assume the bottleneck occurs at the entry to the section of road under consideration, thus ignoring the interplay among successive bottlenecks; we also assume a simple bottleneck of constant capacity, foregoing the treatment of merges or other cases where capacity varies as a function of conditions. We further simplify by assuming the daily dynamic demand pattern in one direction can be approximated by two continuous periods,

[3] Cassidy and Bertini (1999) suggest that the highest observed flow, which is larger, is not a suitable definition of capacity because it generally breaks down within a few minutes—although Cassidy and Rudjanakanoknad (2005) hold out some hope that this might eventually be overcome through sophisticated ramp metering strategies. Our speed-flow function does not include the backward-bending region, known as congested flow in the engineering literature and as hypercongested flow in the economics literature, because flow in that region leads to queuing which we incorporate separately. See Small and Verhoef (2007, Sect 3.3.1, 3.4.1) for further discussion of hypercongestion.

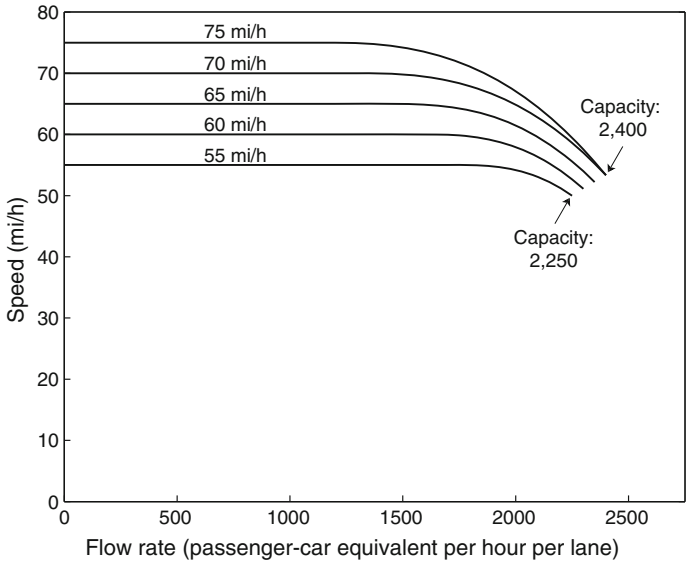

Fig. 1 Speed-flow curves for different free-flow speeds for expressways

"peak" and "off-peak", each with constant demand.[4] Suppose traffic wishing to enter the road arrives at rate V_o during an off-peak period of total duration F, and at rate V_p during a peak period of duration P and starting at time t_p. We describe here the case $V_o < V_K < V_p$ so that a queue forms during the peak period and vehicles leave the queue at the rate V_K. We also assume F is long enough that the queue disappears by the end of the off-peak period. The number of vehicles in the queue, $N(t)$, builds up at rate $V_p - V_K$ starting at time t_p, causing queuing delay $D(t) = N(t)/V_K$ to a vehicle entering at time t. At time $t_{p'} = t_p + P$, the end of the peak period, this delay has reached its maximum value, $D_{\max} = P \cdot [(V_p/V_K) - 1]$. The queue then begins to dissipate, shortening at rate $(V_K - V_o)$ until it disappears at time $t_x = t_{p'} + (V_p - V_K)P/(V_K - V_o)$.

The resulting queuing delay has the triangular pattern shown in Fig. 2. The average queuing delay to anyone entering during the peak period is

$$\bar{D}_p = \frac{1}{2} D_{\max} = \frac{1}{2} P \cdot [(V_p/V_K) - 1]. \tag{1}$$

The same average queuing delay affects those arriving between times $t_{p'}$ and t_x, which when averaged with the other off-peak travelers (who experience no queuing delay) produces the average off-peak travel delay:

$$\bar{D}_o = \frac{\bar{D}_p \cdot (x - P)}{F} = \frac{1}{2} \frac{P^2}{F} \cdot \frac{\left(V_p - V_K\right)^2}{V_K \cdot (V_K - V_o)} \tag{2}$$

where $x = t_x - t_p$ is the total duration of queuing (hence $x - P = t_x - t_{p'}$).

[4] Analyzing more daily time periods would of course increase precision, but would also make the model more location-specific. By using only two periods, we miss some congestion that is caused by queue buildups during shorter periods of higher demand, and thus may understate the advantage of higher-capacity roads. To compensate, we use a rather long 4-h one-directional peak.

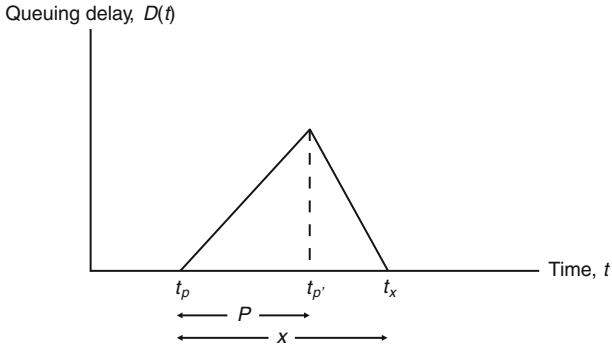

Fig. 2 Queuing delay for vehicles arriving at road entrance at time t

In addition to queuing delay, urban arterial drivers face "control delay" reflecting additional time lost slowing and waiting for signalized intersections. Let Z denote the number of signalized intersections on the urban street encountered on a trip of length L. Based on the HCM's procedure (see Appendix), the average control delay per vehicle for through movements, δ, is:

$$\delta = Z\left\{\frac{0.5C\cdot(1-g/C)^2}{1-[(V/V_K)(g/C)]} + 900T\left[\left(\frac{V}{V_K}-1\right)+\sqrt{\left(\frac{V}{V_K}-1\right)^2+\frac{8kV}{TV_K^2}}\right]\right\} \tag{3a}$$

where C is signal cycle length, g is effective green time, V is the volume of traffic going through the intersection, T is the duration of the analysis period (in hours), and k is the incremental delay factor. Time durations C, g, and hence δ are all conventionally measured in seconds. The first and second terms within the curly brackets in Eq. (3a) are known as uniform control delay and incremental delay, respectively. The uniform control delay assumes that vehicles arrive at a signal at a constant rate V, which results in queuing if the signal is red, and it incorporates the way this queue dissipates when the signal turns green. That dissipation depends on lane-specific saturation rates, which we are able to relate to the overall capacity V_K of the highway (see Appendix). The incremental delay accounts for the saturation level of the road as well as random arrivals, adjusted for the type of signal control through the factor k.

Recall that due to our assumption that V_K limits the flow upstream of the roadway (because queuing occurs at the entrance to the road when $V_p > V_k$), V is equal to the queue discharge rate V_K during the time period from t_p to t_x. At all other times, V is equal to the off-peak volume, V_o. Thus, for peak travelers and for those off-peak travelers who experience queuing at the bottleneck, the control delay for each vehicle is obtained by setting $V = V_K$ and $T = x$ in (3a):

$$\delta_p = Z\left[0.5C(1-g/C) + 900x\sqrt{(8k)/(xV_K)}\right]. \tag{3b}$$

For all other off-peak travelers, the control delay is given by (3a) with $V = V_o$ and $T = F - (x - P)$:

$$\delta_o = Z\left\{\frac{0.5C\cdot(1-g/C)^2}{1-[(V_o/V_K)(g/C)]} + 900(F-x+P)\left[\left(\frac{V_o}{V_K}-1\right)+\sqrt{\left(\frac{V_o}{V_K}-1\right)^2+\frac{8kV_o}{(F-x+P)V_K^2}}\right]\right\} \tag{3c}$$

We now combine all four sources of delay and add them over vehicles. Peak travelers, of whom there are V_pP, experience speed $S(V_K)$ while moving. Adding control delay and queuing delay yields total travel time in hours:

$$TT_p = V_p P \cdot \left[\frac{L}{S(V_K)} + \frac{\delta_p}{3600} + \bar{D}_p \right]. \tag{4}$$

Among off-peak travelers, a portion numbering $V_o \cdot (x - P)$ travel at speed $S(V_K)$ while moving, whereas the remainder, numbering $V_o \cdot (F - x + P)$, travel at speed $S(V_o)$. Again adding control and queuing delay, their total travel time is:

$$TT_o = V_o \cdot \left[(x - P)\left(\frac{L}{S(V_K)} + \frac{\delta_p}{3600} \right) + (F - x + P)\left(\frac{L}{S(V_o)} + \frac{\delta_o}{3600} \right) + F\bar{D}_o \right]. \tag{5}$$

We now need to make explicit assumptions about the number of days per year for which our analysis applies. It is common in large urban areas to have severe congestion on weekends, but for a shorter period than on weekdays. Therefore, we assume that the peaking analysis applies to regular work days (255 days/year) plus half the remaining days (55/year), and we denote the total travel time for all drivers on each of these days as

$$TT_w = TT_o + TT_p \tag{6}$$

It is assumed that the remaining 55 days have traffic volumes typical of the off-peak periods during work days. For simplicity we call them "Sundays" but they actually represent various parts of the 110 annual non-work days. Thus, by definition, each driver on a "Sunday" has travel time $L/S(V_o)$, so that total travel time for all drivers on a Sunday is:

$$TT_s = (F + P) \cdot V_o \cdot \frac{L}{S(V_o)} \tag{7}$$

With these assumptions, we can express the average daily traffic (ADT) as

$$ADT = (310/365) \cdot (PV_p + FV_o) + (55/365) \cdot (P + F) \cdot V_o \tag{8}$$

The annual total and average travel time for all vehicle trips are, respectively:

$$TT_{all} = 310 \cdot TT_w + 55 \cdot TT_s \tag{9}$$

$$\overline{TT}_{all} = \frac{TT_{all}}{365 \cdot ADT} \tag{10}$$

Comparisons between Designs with Equal Construction Cost

In this section, we make two comparisons of roads with "regular" and "narrow" designs, one for expressways and one for signalized urban arterials (which we shall refer to interchangeably as urban streets). In each case, we hold constant the total width of the roadway so there is very little construction cost difference between the two roads in each comparison. We also hold fixed the distance between the two halves of the road, so we need only consider one half, carrying traffic in one direction.[5] This comparison enables us to focus on the two primary factors that distinguish these designs from each other: travel time and safety. We discuss safety in the "Safety" section; in this section, we focus solely on travel time. By choosing designs for which trucks are permitted, we minimize possible

[5] We ignore the difference in cost due to converting part of the paved shoulders in the "regular" design to vehicle-carrying pavements in the "narrow" design; since the largest component of new construction cost is grading and structures, this difference should be minor. We also ignore any differences in maintenance cost that may occur because vehicles on narrow lanes are more likely to veer onto the shoulder or put weight on the edge of the pavement (AASHTO 2004, p. 311).

differences in cost or safety due to rerouting or separation of truck traffic. In actual implementation, some of the changes analyzed here might be done only during rush hours, thereby lessening the impacts on ease of truck travel.

The 2000 Highway Capacity Manual (henceforth HCM) provides methodologies for determining road capacities, free-flow speeds, and indeed the entire speed-flow functions for expressways and urban streets with different specifications. As described in Appendix, we use this information to determine the values V_K, $S(V_K)$, and $S(V_o)$ appearing in equations (3)–(5).

Our specifications are summarized in Fig. 3 and Table 1. First, consider expressways. We specify Expressway R (the "regular" design) with two 12 ft lanes in one direction, a 6 ft left shoulder, and a 10 ft right shoulder, bringing its total one-directional roadway to 40 feet (Fig. 3a). These are the minimum widths recommended for "urban freeways" by AASHTO (2004) except we have added two feet to the left shoulder. Expressway N (the "narrow" design) has three 10 ft lanes, a 2 ft left shoulder, and an 8 ft right shoulder. As shown in Table 1, this road's narrower lanes and shoulders lead to a lower free-flow speed and thus a lower capacity per lane compared to Expressway R; but its total capacity (V_K) is higher since it has more lanes.

Next, consider signalized urban arterials. Here we compare two high-type urban arterial streets, each with the same number of signalized intersections and the same one-directional road width (38 ft). Following the lane and median width recommendations by AAHSTO (2004), the "regular" urban street (Urban Street R) has two 12 ft lanes in one direction for through movement, a 6 ft left shoulder and an 8 ft right shoulder (Fig. 3b). At signalized intersections, the entire median (consisting of the left shoulders of *both* directional roadways) is used for a 12 ft exclusive left-turn lane, which therefore occupies the same lateral space as the left-turn lane facing it in the opposite direction. The rightmost through lane is a shared right-turn lane, and the right shoulder width remains at 8 ft. We assign this urban street a speed limit of 55 miles/h.

The "narrow" urban street (Urban Street N), by contrast, has three 10 ft lanes in one direction for through movement, a 2 ft left shoulder, and a 6 ft right shoulder—the latter serving to minimize the impact of improperly stopped vehicles on traffic flow. At signalized intersections, the right shoulder width is reduced to 3 ft; the additional roadway plus the median are used to provide for an exclusive left-turn lane of 10 ft while maintaining three 10 ft through lanes, one of which is a shared right-turn lane.[6] We give it a speed limit of 45 miles/h.

These assumptions enable us to derive free-flow speeds and intersection delays by following procedures in the HCM, as explained in Appendix A (Online Resource). Table 1 shows selected results. The free-flow time advantage for a trip of $L = 10$ miles is 0.77 min for the "regular" compared to the "narrow" expressway; and it is 1.17 min for the "regular" compared to the "narrow" arterial. Recall that each pairwise comparison is of two roads occupying the same width and hence with nearly identical construction costs.

Using the specifications listed in Table 1, we can calculate average travel times for a range of traffic volumes and time-of-day distributions. We consider the 16-h daily period 6 am–10 pm, during which about 95 % of all trips take place (Hu and Reuscher 2004,

[6] Note that for the urban streets in Fig. 3b, the total two-directional roadway width at the intersection itself is less than the sum of those of the two separate one-directional roadways, because the left turn lanes in both directions share the same lateral space. That is, the width of the two directional roadway includes only the width of one, not two, left turn lanes. For the "regular" design this is $2 \times (12 + 12 + 8) + 12 = 76 = 2 \times 38$, whereas for the "narrow" design it is $2 \times (10 + 10 + 10 + 3) + 10 = 76 = 2 \times 38$; hence both are described as having a 38-foot one-directional roadway.

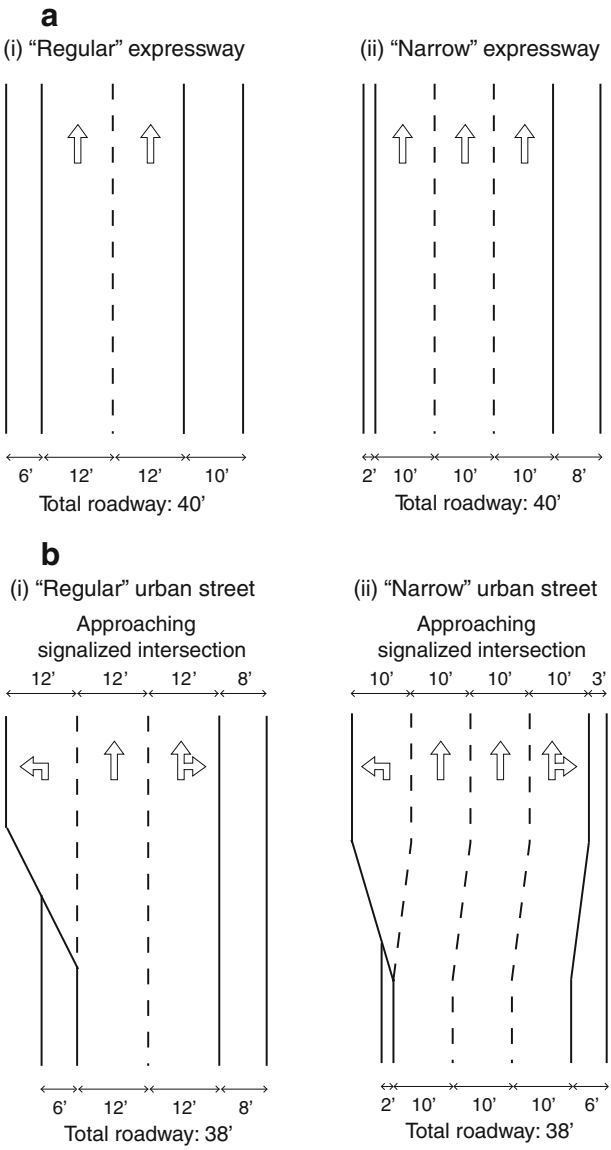

Fig. 3 Example **a** expressways (one direction), **b** urban streets (one direction)

Table 28), and analyze a one-directional roadway with a 4-h peak period ($P = 4$) and a 12-h offpeak period ($F = 12$).[7] We report results for two alternate assumptions about

[7] The calculations are done with each period continuous (i.e. 6–10 am peak, 10 am–10 pm offpeak). We found it makes a negligible difference if the peak is in the afternoon so the offpeak period is split into two parts. We also computed results for a two-peak scenario with each peak period equal to 2 h, representing a case where the traffic is evenly distributed during the morning and afternoon peaks. The two-peak scenario is briefly addressed at the end of this and the next sections, and both results are described in detail in Appendix B of the Online Resource.

Table 1 Specifications for examples (all one direction)

	Expressway R: regular lanes and shoulders	Expressway N: narrow lanes and shoulders	Urban St R: regular lanes and shoulders	Urban St N: narrow lanes and shoulders
Parameters				
Number of lanes	2	3	2	3
Lane width (ft)	12	10	12	10
Left shoulder width (ft)	6	2	6	2
Right shoulder width (ft)	10	8	8	6
Total roadway (ft)	40	40	38	38
Length (miles)	10	10	10	10
Proportion of heavy vehicles[a]	0.05	0.05	0.05	0.05
Driver population factor[a]	1.00	1.00	1.00	1.00
Peak hour factor[a]	0.92	0.92	0.92	0.92
Interchanges/signals per mile[a]	0.50	0.50	0.50	0.50
Signal cycle length (s)[a]	–	–	100	100
Effective green time (s)[a]	–	–	70	70
Speed and capacity				
Free-flow speed (miles/h)[b]	65.5	60.4	51.5	46.8
Speed at capacity (miles/h)[b]	52.3	51.2	51.5	46.8
Capacity per lane (veh/h/ln)[c]	2,113.76	2,067.98	1,165.33	1,087.64
Total capacity, V_K (veh/h)[c]	4,227.51	6,203.94	2,490.96	3,486.86
Travel time				
Free-flow travel time (min)	9.16	9.93	12.03	13.20

[a] In most cases, the default values recommended by the HCM are used; see Appendix A in Online Resource. The recommended default value for interchanges per mile is used for expressways, and we assume a comparable number for signal density on urban streets. The signal cycle length is based on the HCM's default value for non-CBD areas (see Exhibit 10-16 of the HCM). A relatively high effective green time is chosen

[b] For expressways, the HCM calculates average passenger-car speed based on total flow rate using passenger-car equivalents (pces) for heavy vehicles. Our calculations are based on the average speed of passenger cars

[c] For urban streets, "capacity per lane" is based on the capacity of lanes which allow only through movement. See Appendix A in Online Resource for how total capacity is calculated for both expressways and urban streets

peaking, one with a modest amount ($V_p/V_o = 1.25$) and one with a large amount ($V_p/V_o = 2.0$); see Appendix B of the Online Resource for evidence that this covers a reasonable range of conditions.

Figure 4 shows the resulting average travel times for the four different road designs under different values for average daily traffic (see Eq. 8), and the ratio of peak volume (V_p) to off-peak volume (V_o). (Times are shown for ADT up to the value that would leave a queue remaining at the end of the off-peak period.) We see from Fig. 4a that when $V_p/V_o = 1.25$, the "regular" freeway experiences queuing when ADT exceeds 56,984, but queuing does not occur on the "narrow" freeway for ADT values up to 65,000 because the latter has a higher capacity. Once queuing begins, the increase in average travel time is so marked that the average travel time on the "regular" freeway begins to exceed that of the "narrow" freeway when ADT is just a little higher than the value at which queuing begins.

Some of this travel-time increase can be attributed to the lower speed when the lanes become more crowded, but most of it is due to queuing delay. In the case of the signalized urban arterials, the "regular" and "narrow" urban streets experience queuing when ADT exceeds 33,576 and 47,001, respectively. The average travel time on the "regular" urban street starts to exceed that of the "narrow" urban street when ADT is greater than 33,147.

Figure 4b shows the average travel times for the four highway types when $V_p/V_o = 2$. With much higher traffic volumes during the peak hour compared to the previous scenario, queuing now begins at lower values of ADT. Once queuing begins, average travel time on the "narrow" design increases at a lower rate compared to the "regular" design because the former has more capacity and thus discharges vehicles from the queue at a higher rate.

Thus even though the "regular" roads have slightly shorter average travel times (compared to the "narrow" roads) when traffic volumes are low, this advantage is quickly erased when they experience queuing—all the more so when V_p/V_o is large, since then more vehicles experience queuing, the duration of the queue is longer, and fewer vehicles reap the advantages of higher free-flow speed.

We can also calculate the values of ADT and V_p/V_o for which the difference in average travel time between the "regular" and "narrow" designs is zero. Figure 5a, b show this (and other) contour lines for expressways and urban streets, respectively, plotted so that a positive number favors the "narrow" design. In both figures, the "narrow" design has shorter average travel times compared to the "regular" design in the region to the right of the "0" contour line. For the example expressways, the lowest value for the difference in average travel time (i.e., the largest possible advantage for the regular design) is −0.77 min, which occurs under free-flow conditions for both expressways. For the urban streets, the lowest difference is −1.17 min.

We observe that the "narrow" design is strongly favored under all conditions in which there is appreciable queuing. Most strikingly, the advantage of the "narrow" design increases extremely rapidly with traffic. By contrast, the advantage of the "regular" design for light traffic volumes is very modest and increases very slowly as traffic decreases. This is because the "narrow" design's advantage depends on queuing, whereas the "regular" design's advantage depends on the difference in free-flow speeds, which is quite small. While the specific numbers depend on our particular examples, these broad features result from well-established properties of highway design, and so are quite general.

As a sensitivity analysis, we performed the same calculations for a two-peak scenario with each peak period equal to 2 h, representing a case where the traffic is evenly distributed during the morning and afternoon peaks. If we assume that the queue from the first peak period dissipates completely by the time the second peak period begins, in this scenario there are two smaller triangles with half the height of the single triangle in Fig. 2, resulting in total queuing delay equal to half that of the one-peak scenario. Control delay on signalized urban streets is lower as well. The results are still qualitatively similar to those of the one-peak scenario, but the "narrow" road's advantage is realized at a slightly higher ADT and differences in average travel time between the "regular" and "narrow" designs are smaller. Details are provided in Appendix B of the Online Resource.

Comparison between Expressways and Arterials with Equal Capacities

As seen in the previous section, signalized urban arterials tend to have lower free-flow speeds and capacities than expressways. However, arterials also tend to have lower construction costs. Thus, it is useful to consider when it can be more cost-effective to build a

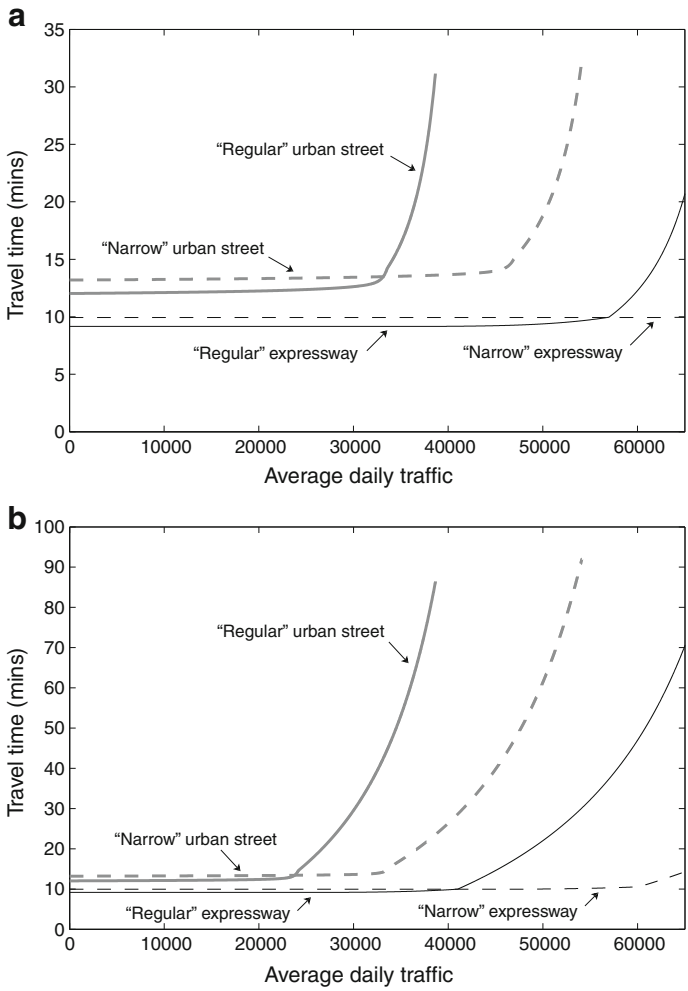

Fig. 4 Average travel times for **a** $V_p/V_o = 1.25$, **b** $V_p/V_o = 2$

lower-speed, less expensive arterial instead of an expressway. We consider only new facilities, whose merits are still hotly debated even in heavily built-up areas; the principles considered here could also apply to reconstructions of aging facilities, but the cost parameters would be so specific to a particular project that its results would have little generality.[8]

In order to make realistic comparisons, we now consider a very high type of arterial, considerably higher than those of the previous section: namely, one that is divided, is uninterrupted by traffic signals, and has driveway or side-street access no more than once

[8] Nevertheless, we performed the same analysis using reconstruction costs of existing lanes, which are lower than the construction costs of new alignment shown in Table 3. The results are qualitatively similar: using reconstruction costs favors the narrow roads somewhat less in small cities and more in large cities. The sources used are the same as those listed in Table 3; see Appendix C of the Online Resource for more details.

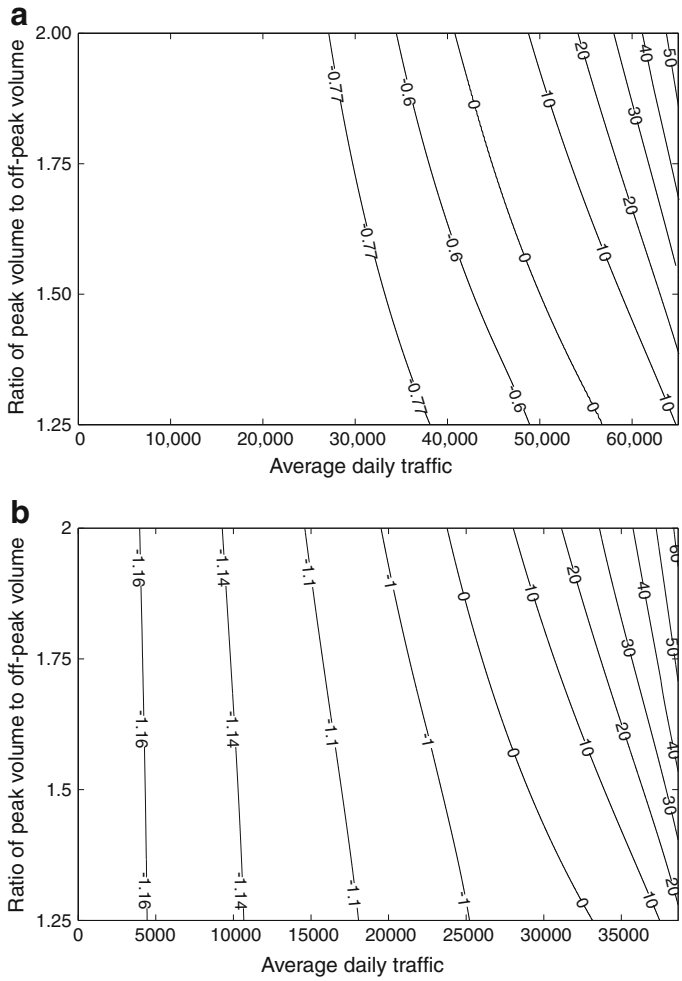

Fig. 5 Contour map of the difference in average travel time **a** "regular" expressway minus "narrow" expressway, **b** "regular" urban street minus "narrow" urban street

every two miles. This meets the HCM definition of "multilane highway" and differs from an expressway by allowing some access by other than entrance and exit ramps; but it has grade-separated intersections for all major crossings. Examples include the Arroyo-Seco Parkway (SR-110) in the Los Angeles region, Storrow Drive in Boston, and most of Lake Shore Drive in Chicago.[9] Since not all intersections are grade-separated, drivers on minor roads may have to take a longer route to one of the grade-separated crossings; this additional delay could be accounted for in our model with some additional assumptions, but we think it would not make enough difference to be worth the extra complication. In the case

[9] Lake Shore Drive includes six signalized intersections (only five going southbound) within its 15-mile length, for an average spacing of over two miles; but all the signals are within a central section about 2.4 miles in length. This highway opened in 1937 (Chicago Area Transportation Study 1998).

of Storrow Drive and Lake Shore Drive, cross traffic is less of an issue due to the fact that they both run along a river or lakefront, so there are few streets that cross all the way through.

We wish to examine total costs, including construction and travel-time costs, of a network of expressways versus a network of unsignalized arterials, each with the same capacity. We therefore compare the "regular" expressway from the previous section with an unsignalized arterial with similar characteristics. Using the procedures outlined in Chapters 12 and 21 of the HCM, this unsignalized arterial has a capacity of 3,945 veh/h—only seven percent less than that of "Expressway R" of Table 1. To equalize capacities, then, requires that the arterial network have 1.07 times the number of lane-miles as the expressway network—implicitly assuming the network is large enough to ignore indivisibilities. [10] We therefore consider again a road section of $L = 10$ miles, but multiply the arterial construction cost by 1.07. We assume that roads in each network provide access to the same origin and destination points, and that traffic volumes are distributed proportionally throughout a given network so that average travel times for a ten-mile trip are the same everywhere. (We also assume travel distances are the same for both networks, thereby ignoring the arterial network's advantage in having more total road-miles, thereby providing more access points and therefore reducing some trip distances.)

The HCM free-flow speeds are 65.5 and 59.9 miles/h for the expressway and arterial, respectively (exclusive of any signal delays). There is an anomaly, however, in comparing their speeds at capacity: the HCM formulas imply a slightly *lower* value for the expressway than for the arterial (52.3 versus 54.9 miles/h), due to the fact that speed falls less steeply with flow for unsignalized arterials than for expressways (see Exhibits 21-3 and 23-2 of the HCM). While this situation could possibly be explained by a disruptive effect of more rapid accelerations and decelerations on the expressway, we take the more conservative stance that it is an anomaly resulting from different chapters of the HCM being developed by different research groups. Thus, we equalize the speeds at capacity by increasing the rate at which arterial speed falls with traffic volume (see Appendix A in Online Resource).

The free-flow travel time on the expressway is 9.16 min, compared to 10.02 min for the unsignalized arterial network (difference: 0.86 min). At positive traffic volumes, the expressway's speed advantage gradually erodes. Once capacity is reached (at the same travel volume, due to our equalizing capacities), queuing sets in, with queuing delay identical for the two road networks.

Figure 6 shows the resulting contour lines of the difference in travel times (average travel time on the expressway minus average travel time on the arterial network) for a range of V_p/V_o and ADT values. We can see that the expressway always has shorter travel times up to an ADT of 65,000 (beyond this, the queuing period is longer than the 12 h off-peak period). The kink seen in the -0.6 contour line indicates the ADT at which queuing begins for the corresponding value of V_p/V_o.

Even though the expressway has shorter average travel times, we need to weigh the annual travel time savings against amortized construction costs since the arterial has lower construction costs. We examine this for the case where peak volume, V_p, is equal to $1.05 \times V_K$, so that each road network experiences a small amount of queuing, We again consider two values of V_p/V_o: 1.25 and 2. Under these conditions, average travel time on the expressway is 0.68 min shorter than that on the arterial network for $V_p/V_o = 1.25$, and 0.54 min shorter for $V_p/V_o = 2$. We value these time savings at \$10.50 per vehicle in our

[10] Of course, the idea of building an entire network is an idealization, made here solely in order to account for the different capacities of different design options.

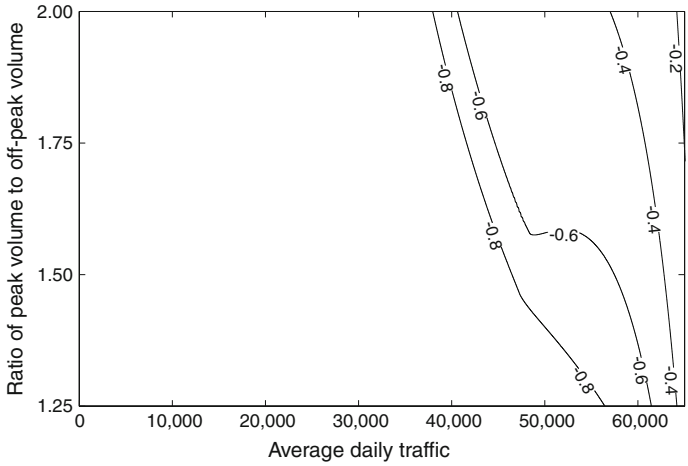

Fig. 6 Contour map of the difference in average travel time (expressway network minus unsignalized arterial network)

base case, with plus or minus 30 % as low and high cases.[11] The resulting aggregate travel-time cost savings of the expressway versus the arterial network are shown in Table 2.

We now turn to construction costs. Table 3 presents estimates of construction costs for expressways and other principal arterials compiled by Alam and Kall (2005) for the US Federal Highway Administration's Highway Economic Requirements System (HERS) model, based on samples of actual projects. These figures show that construction cost per additional lane on new alignment is typically 23–31 % lower for an arterial than for an expressway, with the smaller differences applying to larger urban areas. We restrict our consideration to urban areas of more than 200,000 people.

However, the cost differences from the HERS model are likely to overstate those applying to our comparison for two reasons. First, we are considering a higher type of arterial than the average in the sample. Second, these figures are based on averages for traffic conditions prevailing on actual roads built; since the expressways are likely handling more traffic than the arterials, their costs are likely to reflect some design features motivated by this higher traffic; thus the HERS cost differences may overstate the differences that would occur for a given (fixed) set of traffic conditions such as we consider here. Therefore, we assume that the applicable costs for our high-type arterial are midway between those for expressway and other principal arterials shown in Table 3. The resulting costs are shown in the last column of the table.

Using the costs in Table 3, the amortized construction cost per lane-mile is calculated as $r \cdot C_{ROW} + [r/(1 - e^{-r\lambda})] \cdot C_{other}$, where C_{ROW} and C_{other} are right-of-way and other construction costs, r is the interest rate (assumed to be 7 %, as recommended by the Office of Management and Budget for cost-benefit analyses of transportation and other projects (1992)), and λ is the effective lifetime of the road (assumed to be 25 years). This amortized

[11] According to Small and Verhoef (2007, Sect. 2.6.5), the value of time for work trips is typically estimated as 50% of the wage rate, which would be about $10.50 per hour for 2009 (BLS 2010, Table 1, reporting mean hourly wage for civilian workers). We assume these value of time studies apply to the entire vehicle, although authors are often ambiguous. Values of time are higher for work trips than for others, but occupancies are lower; we assume these two factors balance out between peak and off-peak travel so assign them both the same value of time.

Table 2 Travel-time cost savings of the expressway for $V_p = 1.05V_K$ (in thousands of 2009 dollars/year)

	$V_p/V_o = 1.25$	$V_p/V_o = 2$
Base value of time: $10.50	2,601	1,494
Low value of time: $7.35	1,821	1,046
High value of time: $13.64	3,379	1,941

Table 3 Construction costs per lane on new alignment in urban areas (in thousands of 2009 dollars per mile)

Urban area population (1000 s)	Expressway		Other principal arterial		High-type arterial	
	Total	% ROW	Total	% ROW	Total	% ROW
200–1,000	14,507	3.0 %	10,072	5.7 %	12,289	4.1 %
>1,000	18,152	18.3 %	13,871	18.3 %	16,012	18.3 %

Source: See text for last two columns. Other columns computed as follows. Roadway costs are from Alam and Kall (2005, Table 9). Non-roadway costs other than right of way (*i.e.*, engineering, environmental impact and mitigation, intelligent transportation systems, urban traffic management, and bridges) are from multipliers for roadway costs, in Alam and Kall (2005, Table 13). Right-of-way (ROW) costs are from Alam and Ye (2003, Table C-10) in the case of urban areas of 200–1,000 thousand, and from the multiplier 0.39 as recommended by Alam and Kall (2005, Table 13) in the case of urban areas of more than 1 million. All costs have been adjusted from 2002 dollars to the average price level in the years 2007–2009 using the Federal Highway Administration's Composite Bid Price Index (FHWA 2003) and National Highway Construction Cost Index (FHWA 2011). This average price level was used because actual construction costs were very volatile during those years

cost is then multiplied by the number of lanes (2), the length of the road (10 miles), and the relative number of roads (i.e., 1.00 expressway, 1.07 arterials) to obtain the total amortized construction cost of the compared road sections. Table 4 shows the results for six different cases governing the construction-cost differential between arterials and expressways. The "base" cost differential reflects costs given in Table 3 (for the larger two sizes of metropolitan areas). The "higher" and "lower" differentials are 1.5 and 0.5 times the base differential.

We can compare the difference in travel-time cost in Table 2 to the difference in construction cost in Table 4 to see which road is favored under different scenarios. Figure 7 illustrates the total annual cost differential (expressway minus arterial) for urban areas of more than 1 million people, where negative numbers indicate that the expressway has lower total annual costs because its travel time savings outweigh its higher construction costs. The expressway's advantage can be clearly seen when values of time are assumed to be high and construction cost differentials are low. However, there are several scenarios where the arterial is favored, especially when ratio of peak traffic to off-peak traffic is high.

As in the previous section, we considered an alternative "two-peak" scenario with two 2-h peaks instead of a single 4-h peak. In this scenario, queuing occurs at the same point for both roads. Even though average travel times rise more slowly under the two-peak scenario compared to the one-peak scenario once queuing begins, the difference in average travel times between the expressway and the arterial network is the same under both scenarios for a given ADT and V_p/V_o (see Appendix B of Online Resource). Therefore, our conclusions in this section hold for the two-peak scenario as well. We also ran sensitivity analyses with two different interest rates for amortized construction costs: a lower rate of 5 % and a

Table 4 Difference in amortized construction costs for equal-capacity expressway and arterial networks (in thousands of 2009 dollars/year)

Cost differential	Urban area population (1,000s)	
	200–1,000	>1,000
Base	2,330	1,673
Low (×0.5)	1,165	837
High (×1.5)	3,495	2,510

higher rate of 9 %. Since higher interest rates widen the gap in construction costs, the arterial is favored under more situations when interest rates are higher.

Safety

Conventional wisdom is that many of the smaller-footprint design features considered here would increase traffic accidents. We consider in Appendix D of the Online Resource whether increased accident costs would be likely to alter the results found so far.

The conventional wisdom relies on several posited uses of extra lane or shoulder width: to accommodate inattention, to maneuver in case of a near-accident, or to make emergency stops. But there are compensating behaviors that tend to offset these advantages, including higher speeds (Lewis-Evans and Charlton 2006), closer vehicle spacing, and a tendency to use wide shoulders for discretionary stops which are quite dangerous (Harwood et al. 2007). These are examples of the well-known hypothesis of Peltzman (1975) that safety improvements are partially or fully offset by more aggressive driving.

It is difficult to draw definitive conclusions about the relative importance of these offsetting factors because statistical studies are ambiguous. This is partly because there are many unmeasured road attributes, such as age and hazardous terrain, that are closely correlated with design features. Another problem is that these comparisons sometimes hold constant the posted speed limit but not the frequency of access points, just the opposite of the conceptual comparison relevant to this paper.

Thus, it is an open question whether the "narrow" road designs considered here would in fact reduce safety. Probably the result would depend on factors that vary from case to case, especially the speeds chosen by drivers.

This suggests a possible strategy of accompanying such roads with lower speed limits and/or other measures to discourage speeding. Would speed-control measures be accepted by drivers? Our reading of the guidelines on highway design suggests that such measures are more likely to be accepted when the road design is modest, because drivers then intuitively understand the rationale for them. This is also a conclusion of Ivan et al. (2009). There is some supporting evidence from Europe for the possibility of implementing effective speed control, especially in Germany, the Netherlands, and Denmark (e.g. Mirshahi et al. 2007). Sweden, the Netherlands, Spain, and the UK have even carried out on-road demonstration studies of such on-vehicle systems that limit speed, with encouraging results (Liu and Tate 2004).

If smaller-footprint roads were also automobile-only, a possibility mentioned earlier though not analyzed here, they probably would have further safety advantages. For example, Lord et al. (2005) show that on the New Jersey Turnpike, which has two parallel roadways of which one is for cars only, accident rates are higher in the lanes that allow trucks. Fridstrøm (1999) finds that overall injury accident rates are nearly four times as responsive to amount of truck travel than to amount of car travel.

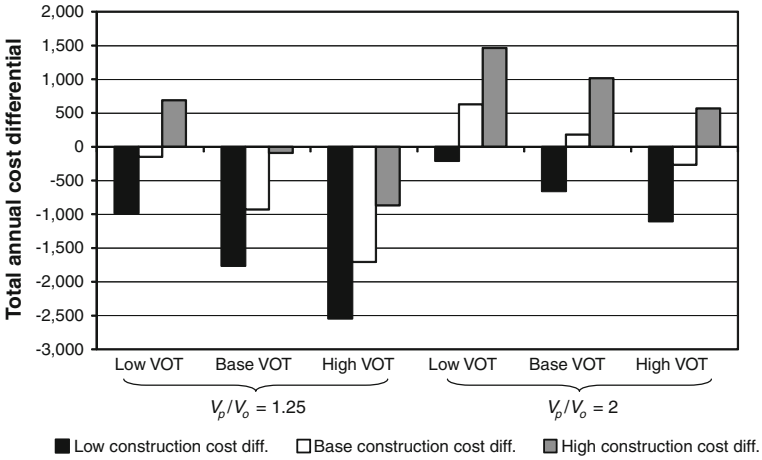

Fig. 7 Total annual cost differential (expressway minus arterial network): $V_p = 1.05V_K$, urban population >1 million (in thousands of 2009 dollars/year). *Note*: Travel time savings are calculated with the base value of time (VOT) as \$10.50/hour, with ±30 % as the high and low values. The base construction cost differential is \$1,673 thousand, with ±50 % as the high and low values. Negative numbers indicate that the expressway has lower total annual costs

In the Online Resource, we present two illustrative calculations in order to assess how safety costs might alter the comparisons comparisons of the previous two sections. First, we examine the implications for our "regular" versus "narrow" expressways comparison if, as suggested by Bauer et al. (2004) and Kweon and Kockelman (2005), narrower lanes and shoulders increase accident rates by 10 %. Applying standard parameters for costs of accidents, we find there that these extra costs would very rarely reverse the advantage of the "narrow" design.

Second, we examine implications for our comparison of equal-capacity expressways versus arterials. We apply the finding of Kweon and Kockelman (2005) that expressways of Interstate standards are associated with a 46.1 % and a 14.7 % decrease in fatal and injury crash rates, respectively, compared to other limited-access principal arterials. We calculate that for the example networks in our comparison, accident costs would be higher for the low-footprint design by \$5.4 million and \$3.9 million for the lower and higher congestion cases, respectively, which would offset the advantage of an arterial compared to an expressway.

Thus, we think our conclusions about regular versus narrow expressways are robust to plausible variations in accident costs, but not those about expressways versus arterials. For this reason, any move to replace expressways with arterials in metropolitan planning would need to be accompanied by a thorough analysis of accidents and would best be part of a comprehensive approach to using speed control or other measures to address safety.

Conclusion and discussion

It seems that the intuitive arguments made in the introduction hold up under quantitative analysis. For both freeways and signalized urban arterials, squeezing more lanes into a

fixed roadway width has huge payoffs when highway capacity is exceeded by even a small amount during peak periods, whereas the payoff from the higher off-peak speeds offered by wider lanes and shoulders is very modest. The advantage of the road with narrower lanes is accentuated when the ratio of peak to off-peak traffic is large. Meanwhile, the savings in travel-time costs offered by a network of expressways, compared to an equal-capacity network of high-type unsignalized arterials, is smaller than the amortized value of the extra construction costs incurred under certain conditions (generally, when the value of time is low, the construction cost differential is high, and when the ratio of peak volume to off-peak volume is high). However, if accident costs are included, arterials may no longer have a cost advantage since expressways are associated with substantially lower accident rates, especially for fatal crashes.

Of course, the pairwise comparisons presented here do not come close to depicting the full range of relevant alternatives for road design. And because so many properties of highways are site-specific, results comparable to ours cannot be assumed to apply to any particular case without more detailed calculations. Furthermore, we did not account for fuel consumption and emissions, which depend on speed, grade, etc., or for effects on truck routing, or for induced travel, which is important when comparing roads of different capacities; instead, we chose comparisons that minimize the likely impact of these factors, for example by choosing networks of equal total capacity when comparing freeways and arterials. Nevertheless, we think these results provide guidance as to what types of designs deserve close analysis in specific cases, and they may provide guidance for overall policy in terms of the type of road network to be planned for. We suspect that in many cases such a network will have fewer expressways built to Interstate standards, and more lower-speed expressways and high-type arterials, than are now common in the US.

Current trends present a mixed picture as to how the relative advantages of different highway designs are likely to change over time. Intractable congestion and general growth of travel, along with limited capital budgets, seem to dictate increasing traffic but probably some peak spreading, thus moving highway parameters toward the lower right in Figs. 5 and 6, with uncertain implications for the comparison. If congestion pricing became widespread, that would curtail traffic while tending to make it more evenly distributed, thus moving parameters toward the lower left and making current practice relatively more attractive.

Aside from the advantages quantified here, it seems likely that the more modest highway designs suggested by these comparisons will also have more pleasing environmental and aesthetic impacts. Highways with slower free-flow speeds can fit better into existing geographical landforms and urban landscapes, permitting more curvature and grades and so requiring less earth-moving and smaller structures such as bridges and retaining walls. Tire noise and nitrogen oxides emissions are likely to be lower. Neighborhood disruption due to land condemnation and construction should be less. These advantages depend on reductions in speeds commensurate with the highway design, implying an important interaction between policies toward highway design and those toward speed control.

Acknowledgments We are grateful to the University of California Transportation Center for financial support, and to three anonymous referees and many colleagues for comments on earlier drafts, especially Pablo Durango-Cohen, Robin Lindsey, Robert Noland, Robert Poole, Peter Samuel, Ian Savage, and Erik Verhoef. Of course, all responsibility for facts and opinions expressed in the paper lies with the authors.

Appendix: Speeds and capacities from the highway capacity manual (2000)

This appendix briefly discusses the HCM's methodology for calculating speeds and capacities for expressways, which the HCM calls "freeways" (based on HCM ch. 13, 23), and arterials (based on HCM, ch. 10, 12, 15, 16, 21). A more detailed explanation of the equations and parameter values are presented in the online version of this Appendix (see Online Resource).

Expressways/Freeways

Capacity varies by free-flow speed, and equation 23-1 in the HCM is used to estimate free-flow speed (FFS) of a basic freeway segment:

$$\mathrm{FFS} = \mathrm{BFFS} - f_{LW} - f_{LC} - f_N - f_{ID} \tag{11}$$

A brief description of each parameter and the parameter values used in the paper are given in Table 5. As shown in Appendix N of the highway performance monitoring system (HPMS) field manual (FHWA 2002), the relationship between base capacity (*BaseCap*, measured in passenger-car equivalents per hour per lane) and free-flow speed is:

$$BaseCap = \begin{cases} 1,700 + 10FFS & \text{for FFS} < 70 \\ 2,400 & \text{for } FFS \geq 70 \end{cases} \tag{12}$$

Equation 23-2 in the HCM is used to convert hourly volume V, which is typically in vehicles per hour, to v_p, which is in passenger-car equivalents per hour per lane (pce/h/ln) and is used later on to estimate speed:

$$\mathrm{v}_p = V/(PHF \times N \times f_{HV} \times f_p) \tag{13}$$

See Table 5 for the parameter values used in this paper. Equation (3) can also be used to calculate capacity in terms of vehicles per hour for all lanes, which we call V_K in this paper, by replacing $V = V_K$ and $v_p = BaseCap$.

We use the HCM speed-flow diagrams in Exhibit 23-3 to calculate average passenger-car speed S (min/h) as a function of the flow rate v_p (pce/h/ln).

Table 5 Parameter values for expressways

Parameter	Description	Value
BFFS	Base free-flow speed (min/h)	70 for urban freeways (default)
f_{LW}	Adjustment for lane width	0 when lane width is 12 ft, 6.6 when lane width is 10 ft
f_{LC}	Adjustment for right shoulder lateral width	0 because right shoulders are wider than 6 ft
f_N	Adjustment for no. of lanes	4.5 for two lanes, 3.0 for three lanes
f_{ID}	Adjustment for interchange density	0 with 0.5 interchanges per mile
PHF	Peak-hour factor	0.92 (default)
N	Number of lanes	2 for the "regular" road, 3 for the "narrow" road
f_{HV}	Adjustment for heavy vehicles	0.98 when percentage of heavy vehicles (trucks and buses) is 5% in urban settings (default)
f_p	Adjustment for driver population	1 for drivers familiar with the road (default)

Note: See Exhibit 13-5, Exhibits 23-4 to 23-7, and equation 23-3 of the HCM

Urban arterials

The high-type unsignalized arterial analyzed in the comparison between freeways and arterials in Sect. 4 is an example of a "multilane highway" in the HCM's terminology. The capacity and free-flow speed of this arterial are calculated using the procedures outlined in Chapters 12 and 21 of the HCM (which are very similar to the expressway calculations). However, we make the following modification to the HCM speed-flow function since the HCM function results in the high-type arterial having a higher speed at capacity than the expressway. For free-flow speeds between 55 and 60 min/h, that speed-flow function (Exhibit 21-3 of the HCM) is:

$$S_a = \mathrm{FFS} - \left[\left(\frac{3}{10}\mathrm{FFS} - 10\right)\left(\frac{v_p - 1,400}{28\mathrm{FFS} - 880}\right)^{1.31}\right] \tag{14}$$

The high-type arterial's speed at capacity (which we shall denote as S_a^{cap}) can be calculated from this equation by setting flow rate v_p equal to capacity. Denoting the expressway's speed at capacity as S_e^{cap}, we then define a modified speed-flow function for the high-type arterial so that $S_a^{\mathrm{cap}} = S_e^{\mathrm{cap}}$, essentially by increasing the rate at which speed falls with traffic volume. Specifically we use:

$$\tilde{S}_a = \left(\frac{S_a - S_a^{\mathrm{cap}}}{\mathrm{FFS} - S_a^{\mathrm{cap}}}\right)(\mathrm{FFS} - S_e^{\mathrm{cap}}) + S_e^{\mathrm{cap}} \tag{15}$$

where S_a is given by equation (14).

For the signalized urban arterials in Sect. 3 (which the HCM calls "urban streets"; see Chapters 10, 15 and 16), we focus on high-speed principal arterials (design category 1). These arterials have speed limits of 45–55 min/h and a default free-flow speed of 50 min/h (Exhibits 10-4 and 10-5 of the HCM). Using the procedure recommended by Zegeer et al. (2008, pp. 66–73), if we assume the speed limits on the "regular" and "narrow" arterials in Sect. 3 are 55 and 45 min/h respectively, this gives us free-flow speeds of 51.5 and 46.8 min/h.

A vehicle's travel time on an urban street (ignoring queuing due to volumes exceeding capacity, computed separately in the text) consists of running time plus "control delay" at a signalized intersection. Based on Exhibit 15-3 of the HCM, running time for an urban arterial longer than one mile is calculated as the length divided by the free-flow speed.

The formula for calculating control delay (Eq.16–9 in the HCM) is the sum of three components: (1) uniform control delay, which assumes uniform arrivals; (2) incremental delay, which takes into account random arrivals and oversaturated conditions (volume exceeding capacity); and (3) initial queue delay, which considers the additional time required to clear an existing initial queue left over from the previous green period.[12] Because the initial queue limits entry flow to the road's capacity, the initial queue occurs only once at the entry to the road (prior to the first signal) since the traffic volume arriving at each intersection is never greater than the intersection's capacity. As a result, the control delay in this paper consists only of uniform control delay and incremental delay. Using Eqs. 16–9, 16–11 and 16–12 of the HCM, the control delay at a signal is:

[12] The delay calculation in the bottleneck queuing model at the entrance of the road is very similar to the HCM's control delay. In the bottleneck model described in Section 2 of this paper, the uniform control delay is zero (because there is no signal) and the term containing k in Eq. (16) is negligible because of the large traffic volume. The remaining components in Eq. (16) plus the HCM initial queue delay (both converted to hours) give precisely the same result as the bottleneck model.

Table 6 Parameter values for urban arterials

Parameter	Description	Value
C	Cycle length (s)	100 (default for non-CBD areas)
g	Effective green time (s)	70 (see notes in Table 1 of paper)
X	Volume-to-capacity ratio	Varies, equal to V/V_K in paper's terminology
PF	Progression adjustment factor	1 for signals more than 3,200 feet apart and for Arrival Type 3
T	Duration of analysis period (hrs)	Varies depending on peak or off-peak hours
k	Incremental delay factor	$k = 0.92(X - 0.5) + 0.04$, where $0.04 \leq k \leq 0.5$ assuming signals are actuated with snappy intersection operation
I	Upstream filtering factor	1 since upstream signal is more than a mile away

Note: See Exhibits 10-16, 16-12, and 16-13 of the HCM

$$d = \frac{0.5C(1 - g/C)^2}{1 - [\min(1, X)(g/C)]} \cdot PF + 900T\left[(X - 1) + \sqrt{(X - 1)^2 + \frac{8kIX}{cT}}\right] \quad (16)$$

Table 6 provides a description of the parameters and the values used in this paper.

The arterial's capacity, $V_{K,}$ is based on the saturation flow rates of the through and shared right-turn/through lane groups, along with the fraction of time the signal is green and the proportion of traffic at each intersection that is making left turns. Saturation flow means the highest flow rate that can pass through the intersection while the light is green and is calculated based on equations 16–4 and 16–6 of the HCM. Saturation flow rates depend on the number of lanes, lane width, proportion of vehicles turning right, and other factors. For the most part, we use the default values recommended by the HCM and we assume that 7.5 % of the total traffic volume will be vehicles turning left, and similarly for vehicles turning right. Complete details are available in the online version of this Appendix (see Online Resource).

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