Standard Highway Segments:

Source: IHCP (pulled from HCM and other sourced) due to it being established/accepted

Table C-2: Capacities at Free Flow Speed (No Work Zone)

Passenger Car Equivalents per Hour per Lane (PCEs/HR/ln)

Free Flow Speed (MPH)	45	55	60	65	70	75
Volume at Capacity (PCEs/HR/ln)	1900	2250	2300	2350	2400	2400

Table C-3: Model Parameters for Working Hour Capacities

Passenger Car Equivalents per Hour per Lane (PCEs/HR/ln)

Work Zone Type	*Lane Width*		
	12'-11'	10.0' to 10.9'	9.0' to 9.9'
2 lanes reduced to 1 lane	1550	1400	1325
3 lanes reduced to 2 lanes	1600	1450	1375
3 lanes reduced to 1 lane	1475	1350	1275
4 lanes reduced to 3 lanes	1650	1500	1425
4 lanes reduced to 2 lanes	1550	1400	1325
4 lanes reduced to 1 lane	1425	1325	1250

- Base work zone capacities used in the model may be increased by 10% for hours where traffic is restricted but no work is taking place.
- Base work zone capacities used in the model may be increased by 10% for locations designated as "Urban" in the 2017 IHCP Tables provided in Appendix B. In urban areas where traffic is restricted but no work is taking place capacity may be increased by a total of 20%.
- When traffic is to be placed on the left (inside) shoulder, if the shoulder has a rumble strip, the base work zone capacity for that lane will be 15% less than the base work zone capacity listed in table C-3.

Highway Ramps (Interstate to Interstate):

Source: HCM Exhibit 13-10

Context: Limiting factor for most ramps will likely be the intersection, but C-Ds and Interstate to Interstate ramps are not limited in this way and can thus rely on this data

Exhibit 13-10
Capacity of Ramp Roadways (pc/h)

Ramp FFS S_{FR} (mi/h)	Capacity of Ramp Roadway	
	Single-Lane Ramps	Two-Lane Ramps
>50	2,200	4,400
>40–50	2,100	4,200
>30–40	2,000	4,000
≥20–30	1,900	3,800
<20	1,800	3,600

Note: Capacity of a ramp roadway does not ensure an equal capacity at its freeway or other high-speed junction. Junction capacity must be checked against criteria in Exhibit 13-8 and Exhibit 13-9.

Signalized Intersections:

Source: HCM 2010 Exhibit 18-28

Context: The Base Saturation Flow Rate is defined on pg 18-14 of the HCM as follows:

The saturation flow rate represents the maximum rate of flow for a traffic lane as measured at the stop line during the green indication. The base saturation flow rate represents the saturation flow rate for a traffic lane that 12 ft wide and has no heavy vehicles, a flat grade, no parking, no buses that stop at the intersection, even lane utilization, and no turning vehicles. Typically one base rate is selected to represent all signalized intersections in the jurisdiction (or area) within which the subject intersection is located. It has units of passenger cars per hour per lane (pc/h/ln)."

Exhibit 18-28
Default Values: Automobile Mode with Fully or Semiactuated Signal Control

Data Category	Input Data Element	Default Values
Traffic characteristics	Right-turn-on-red flow rate	0.0 veh/h
	Percent heavy vehicles	3%
	Intersection peak hour factor	If analysis period is 0.25 h and hourly data are used: Total entering volume ≥ 1,000 veh/h: 0.92 Total entering volume < 1,000 veh/h: 0.90 Otherwise: 1.00
	Platoon ratio	See discussion
	Upstream filtering adjustment factor	1.0
	Base saturation flow rate	Metropolitan area with population ≥ 250,000: 1,900 pc/h/ln Otherwise: 1,750 pc/h/ln
	Lane utilization adjustment factor	See discussion

Justification: The saturation flow rate represents the potential rate under constant green with practically ideal conditions. Since the assumption is no heavy vehicles the units of pc/h/ln should align well with how heavy vehicles are accounted for with PCEs and allow us to use those rates without converting. Given the lost time for changing phases (all red, acceleration time, etc.) as well as the fact that the cycles will be shared with the other streets, a reasonable reduction that would provide a threshold that would be considered acceptable/conservative enough without greater scrutiny would be 66%. Therefore the resulting proposed capacities would be:

- For an urban location: a capacity of ~640 PCE/hr/ln
- For a rural location: a capacity of ~580 PCE/hr/ln

If the anticipated volumes exceed this or there are special circumstances that would potentially not allow these anticipated capacities, then an analysis of the effects of the additional traffic should be performed by the systems group at the TMC or otherwise justified using HCS or a microsimulation.

Roundabouts:

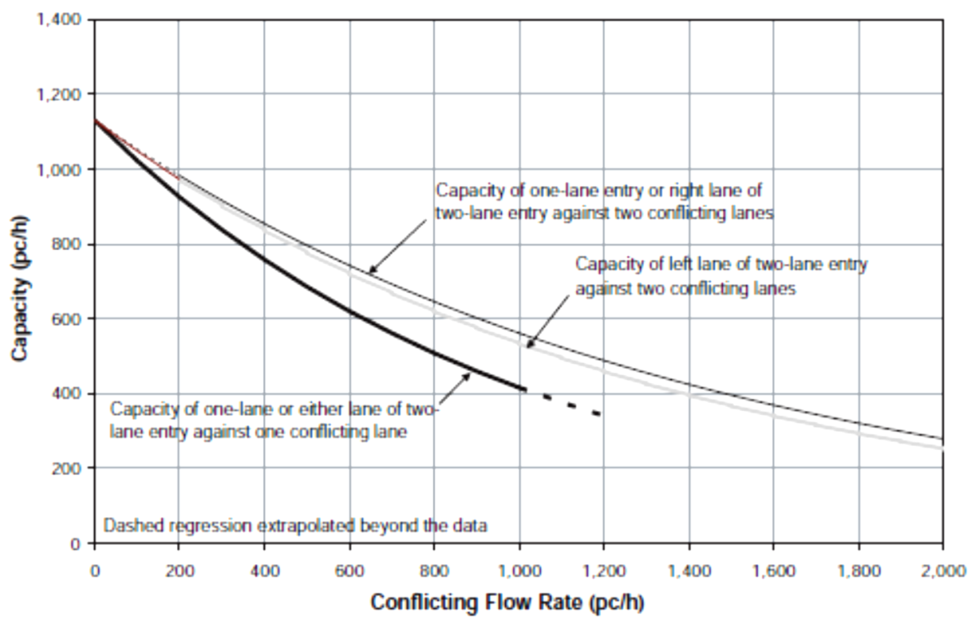
Source: NCHRP Report 672 (considered the current definitive reference for roundabouts)

Link: http://onlinepubs.trb.org/onlinepubs/nchrp/nchrp_rpt_672.pdf

Specific Page: 4-12, Exhibit 4-6

Comment: This doesn’t provide an exact value, but 600 PCE/hr seems reasonable unless the intersecting road is a very busy thoroughfare which leads to a lot of conflicting traffic opposing the ramp traffic. The analyst could reasonably go higher for locations with a high density of roundabouts (like Hamilton County) due to familiarity with RABs and willingness to accept smaller gaps.

Exhibit 4-6
Entry Lane Capacity



TWSC Intersections:

Source: HCM 2010 Chapter 19

Comment: The capacity of a TWSC intersection is the most difficult to estimate because the capacity of the minor road movements is so dependent on the traffic on the 'major road' (in this case, the road intersecting the interstate that the ramp traffic must stop for. To come up with something reasonable that could be applied in a more general case, the potential capacity (assuming there is no upstream signal affecting traffic at the interchange) was examined. Equation 19-32 offers a relatively straightforward answer: $c_p = v_c * \frac{e^{-v_c t_c/3600}}{1-e^{-v_c t_f/3600}}$

where: c_p is the potential capacity (vph)

v_c is the conflicting traffic volume (vph)

t_c is the critical headway for the minor movement (s)

t_f is the follow-up headway for the minor movement (s)

The default values for t_c and t_f used were 7.5 s (from Exhibit 19-10, LT from Minor Rd, 4 lanes on Major Rd, 1 stage movement with no median refuge) and 3.5 s (from Exhibit 19-11, LT from Minor Rd, 4 lanes on Major Rd), respectively. This should cover most locations at interchanges with this type of intersections and provide reasonably conservative results.

	Base Critical Headway, $t_{c,base}$ (s)		
Vehicle Movement	Two Lanes	Four Lanes	Six Lanes
Left turn from major	4.1	4.1	5.3
U-turn from major	N/A	6.4 (wide) 6.9 (narrow)	5.6
Right turn from minor	6.2	6.9	7.1
Through traffic on minor	1-stage: 6.5 2-stage, Stage I: 5.5 2-stage, Stage II: 5.5	1-stage:6.5 2-stage, Stage I: 5.5 2-stage, Stage II: 5.5	1-stage: 6.5* 2-stage, Stage I: 5.5* 2-stage, Stage II: 5.5*
Left turn from minor	1-stage: 7.1 2-stage, Stage I: 6.1 2-stage, Stage II: 6.1	1-stage: 7.5 2-stage, Stage I: 6.5 2-stage, Stage II: 6.5	1-stage: 6.4 2-stage, Stage I: 7.3 2-stage, Stage II: 6.7

* Use caution; values estimated.

Exhibit 19-10
Base Critical Headways for TWSC Intersections

Exhibit 19-11
Base Follow-Up Headways for TWSC Intersections

	Base Follow-Up Headway, $t_{f,base}$ (s)		
Vehicle Movement	Two Lanes	Four Lanes	Six Lanes
Left turn from major	2.2	2.2	3.1
U-turn from major	N/A	2.5 (wide) 3.1 (narrow)	2.3
Right turn from minor	3.3	3.3	3.9
Through traffic on minor	4.0	4.0	4.0
Left turn from minor	3.5	3.5	3.8

Based on the equation, the capacity for a range of major road volumes was calculated:

v_c (veh/h)	c_p (veh/h)	HCS 1	HCS 2
1	1027	1029	1029
100	876	907	880
200	746	798	750
300	635	702	638
400	540	618	541
500	458	542	457
600	389	475	386
700	330	415	324
800	280	362	271
900	237	316	225
1000	200	275	186
1100	169	239	154
1200	143	207	126
1300	121	179	102
1400	102	155	82
1500	86	133	65
1600	72	115	51
1700	61	98	40
1800	51	84	30
1900	43	72	22
2000	36	61	15

The resulting capacities decrease from the theoretical maximum of 1027 vph (if there is virtually no cross-traffic on the major road) as the volume of traffic on the major road increases, which is expected. To check the validity/consistency of these results, HCS was used. The two sets of data for HCS 1 and HCS 2 had different distributions of traffic.

HCS 1 assumed parameters:

- Single shared lane NB (minor road)
- 2 lanes each on EB and WB with shared LT on EB and shared RT on WB
- All minor road traffic was LT from NB
- 50/50 split between directions on major road with all WB traffic going through and 20% of EB traffic turning left and 80% through.
- Default values otherwise

HCS 2 assumed parameters:

- Single shared lane NB (minor road)
- 2 lanes each on EB and WB with shared LT on EB and shared RT on WB
- All minor road traffic was LT from NB
- *50/50 split between directions on major road with all WB traffic going through and 50% of EB traffic turning left and 50% through.*
- Default values otherwise

As shown in the table, the results for HCS 1 always yield capacities for the minor movement greater than the base potential capacity. HCS 2 yields very similar results at lower volumes until it begins to deviate at about v = 700 vph. In this case the number of left turn vehicles from the major road allows less time for the minor road left turns. However, unless the interstate in the same direction as the exit ramp (in this case NB) is the destination, this is unlikely to be a factor and the calculated capacities should be sufficiently conservative.

AWSC Intersections:

Source: HCM 2010 Chapter 20

Context: To simplify the possible variety of possible values I tried to distill the methodology to a representative intersection that would provide guidance on the capacity of an approach on a ramp (since we mainly want to make sure that queues don't back-up to the interstate). To accomplish this I made the following assumptions and calculations:

1. To calculate roughly what the capacity would be for a single approach, I focused on the Saturation Headway. My understanding is that the saturation headway represents the minimum amount of time that would be required per vehicle to pass through the intersection. Therefore: $\frac{1\,veh}{h} * \frac{3600\,sec}{hr} = C$ where h is the saturation headway (eq 20-27 of the 2010 HCM) and C is the capacity of a given approach.
2. Eq. 20-27 requires the base saturation headway and the saturation headway adjustment. The adjustment requires knowledge of proportions of turning movements and heavy vehicles. I plan on accounting for this by rounding up to the nearest 0.5 seconds. The base saturation headway requires designations of groups and cases as shown in Exhibit 20-14. I will describe my determination of which case and group I selected below.
3. Exhibit 20-10 describes the Geometry Groups. Most ramps that will terminate at an AWSC intersection will likely have 2 lanes at most. Since the standard ramps I'm considering would be in a standard diamond configuration the result would be a 4-leg intersection with 0 lanes in the opposing approach. Most of these intersections will have no more than 2 lanes per approach for the conflicting approaches. This combination will result in the model intersection being in Geometry Group 5.

Exhibit 20-10
Geometry Groups

Intersection Configuration	Number of Lanes: Subject Approach	Opposing Approach	Conflicting Approaches[a]	Geometry Group
Four leg or T	1	0 or 1	1	1
Four leg or T	1	0 or 1	2	2
Four leg or T	1	2	1	3a/4a
T	1	2	2	3b
Four leg	1	2	2	4b
Four leg or T	1	0 or 1	3	5
	1	3	1	
	2	0, 1, or 2	1 or 2	
	3	0 or 1	1	
	3	0 or 1	2 or 3	
	3	2 or 3	1	
Four leg or T	1	3	2	6
	1	2	3	
	1	3	3	
	2	3	1, 2, or 3	
	2	0, 1, 2 or 3	3	
	3	2 or 3	2 or 3	

Note: [a] If the number of lanes on conflicting approaches is different, the higher of the two should be used.

4. Similar to the reasoning in point 3, Exhibit 20-7 is the appropriate reference for the Degree-of-Conflict Case. With no opposing and both a conflicting left and right, the Case would 4.

Degree-of-Conflict Case	Approaches with Vehicles: Opposing	Conflicting Left	Conflicting Right	Number of Opposing and Conflicting Vehicles
1				0
2	x			1, 2
3		x		1, 2
			x	
4	x	x		2, 3, 4
	x		x	
		x	x	
5	x	x	x	3, 4, 5, 6

Exhibit 20-7
Degree-of-Conflict Cases for Two-Lane Approaches

5. Base to Exhibit 20-14, the max number of vehicles on the conflicting approaches would be 4. Combined with Case 4 and Group 5, the resulting base saturation headway is 9.0 seconds.

Case	No. of Veh.	Base Saturation Headway (s): Group 1	Group 2	Group 3a	Group 3b	Group 4a	Group 4b	Group 5	Group 6
1	0	3.9	3.9	4.0	4.3	4.0	4.5	4.5	4.5
2	1	4.7	4.7	4.8	5.1	4.8	5.3	5.0	6.0
	2							6.2	6.8
	≥3								7.4
3	1	5.8	5.8	5.9	6.2	5.9	6.4	6.4	6.6
	2							7.2	7.3
	≥3								7.8
4	2	7.0	7.0	7.1	7.4	7.1	7.6	7.6	8.1
	3							7.8	8.7
	4							9.0	9.6
	≥5								12.3
5	3	9.6	9.6	9.7	10.0	9.7	10.2	9.7	10.0
	4							9.7	11.1
	5							10.0	11.4
	≥6							11.5	13.3

Exhibit 20-14
Saturation Headway Values by Case and Geometry Group

6. To account for the headway adjustment given above, an additional 0.5 seconds is added, resulting in a saturation headway of 9.5 seconds.
7. The resulting capacity would be $C = \frac{1\ veh}{9.5\ sec} * \frac{3600\ sec}{hr} \cong 380\ vph$
8. For the purposes of analysis for an IHCP exception, the standard units are PCE/hr. The conversion of Veh/hr to PCE/hr is 1 for all smaller vehicles and 2 for all larger vehicles. Therefore, the capacity would be equal or greater once expressed as PCE/hr. Since we don’t have a constant proportion of trucks, the exact conversion is unknown, but using a capacity of 380 PCE/hr would be sufficiently conservative.